Ministry of Transportation & Infrastructure
Westside Road Underpass No. 7558
Forensic Investigation of Failure of
MSE Wall Facing

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Executive Summary

During the morning of November 20th, 2011, precast concrete facing panels of a mechanically stabilized earth (MSE) wall under the Westside Road Underpass #7558 south of Kelowna collapsed onto Highway 97. At the time of the collapse, the underpass, with its abutments supported on the MSE backfill, had been in service for approximately 3 weeks.

Buckland & Taylor Ltd. (B&T) was retained by the BC Ministry of Transportation and Infrastructure to complete a forensic investigation of the facing panel collapse and to prepare this forensic report.

The MSE walls used on the project are the MSE PLUS™ system, designed and supplied by SSL Construction Products of Scotts Valley, California. The walls consist of rectangular precast concrete facing panels anchored by horizontal reinforcing grids. The grids are comprised of galvanized welded wires extending into compacted backfill.

The reinforcing grids for the three MSE walls built at the Westside interchange, Walls A, B and C, were supplied with two deformed wire sizes: D20 and D11. Grids are connected to the panels through 90° bends in wires that hook onto loops embedded in the panels.

Wall B facing panels, in front of the west abutment, collapsed spilling some backfill. Failed wires connecting the precast facing panels all ruptured at their 90° bends. Following the collapse, Wall C (at the east abutment) was deconstructed and a majority of wires under the abutment were found to be broken at their bend locations. The wire ends at the breaks, in both Walls B and C, typically contained rust indicating that the wires had been broken for some time, estimated to be in the order of three months.

Deformed wires were used instead of smooth wires specified on the MSE wall design and shop drawings developed by the MSE wall supplier. In addition, the grid wires were bent to a tighter radius than specified under applicable ASTM standards and as a result, the wires were excessively cold worked during the bending process. This cold working embrittled the wires at their bends and these were further embrittled during the galvanizing process. Consequently, the wires were susceptible to brittle fracture, particularly at cold temperatures, in the region of the 90° bends. There is a lack of evidence that tests for embrittlement were performed on the galvanized wires as mandated by ASTM A143.

The method used to place and compact backfill during construction of the MSE walls created an approximately 900 mm wide zone of poorly to lightly compacted backfill immediately behind the precast concrete facing panels. The fill within this zone underwent differential settlement with respect to the facing panels during construction that strained the grids and likely ruptured wires.
at the bend locations mostly around the mid-height region of the wall. Part of both east and west abutment footings rest on this poorly to lightly compacted zone. As a result, the abutments settled under progressively applied loads during construction and the top few rows of reinforcing grid connections ruptured at their 90° bends.

The global site settlements were reviewed and found to be within generally acceptable limits for MSE walls. The MSE wall design was also reviewed and found to be generally adequate with the exception of an overstress of the grids at the end of the 100 year service life when material loss due to corrosion is greatest. In addition, the facing panel connection detail did not meet standards because of the tight wire radius.

The cause of the facing panel collapse was the embrittlement of the steel wires at their 90° bends. The embrittlement resulted from an overly tight radius used to fabricate the wire bends and from the galvanizing process whereby the wires were further embrittled because of the high heat applied during the process.

As a result of being embrittled, the wires either partially or completely fractured:

• During the initial bending, where some wires were cracked;
• During backfill placement and compaction, where the methodology did not comply with AASHTO requirements and led to additional loads and demands being placed on the wires at the connections; and
• During settlement of the abutments relative to the facing panels during the course of construction, where wires ruptured in the topmost region of the wall at their connections. The abutment settled because the backfill within 900 mm of the facing panels was only poorly to lightly compacted and as such, could not provide the necessary bearing capacity nor soil stiffness to properly support the abutment footing.

The rupture of most of the wires connecting facing panels under the abutment resulted in a loss of support for the panels which made the wall unstable and on the verge of a collapse.

The eventual failure of Wall B facing panels was likely triggered by the cold temperature at the time of the collapse which lowered the fracture toughness of remaining connected wires at their bends.

The portion of facing panels along Wall C under the east abutment was on the verge of collapsing before being deconstructed because a majority of wires connecting the facing panels were broken making this wall unstable.
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1 Glossary of Terms

Mechanically Stabilized Earth (MSE)

MSE "walls" consist of a soil mass, with artificial reinforcement placed in discrete layers in the soil mass, connected to wall facing panels. The reinforcement used in MSE systems in British Columbia is typically galvanized steel grids or steel strips. A facing system, frequently of architecturally surfaced precast concrete panels, is used to prevent soil ravelling from the vertical reinforced soil face and to facilitate construction. The precast panels are normally keyed along the edges to interlock them.

Abutment

The term abutment in this report refers to a cast in place reinforced concrete element supporting the ends of the bridge girders, acting as a spread footing and supported on top of the MSE walls.

Leveling Strip

A level concrete pad cast against the ground that serves to set the first row of MSE wall facing panels at the correct elevation. The strip can be stepped when the ground profile along the wall is variable.

Mesh

A mesh consists of two planes of wires spaced some distance apart with one plane of wires perpendicular to the other. Wires are typically welded at crossing points. The term mesh as it applies to this report is interchangeable with the term grid. This type of reinforcement is also commonly referred to as welded wire mesh (WWM), welded wire reinforcement (WWR) or welded wire fabric (WWF), all of which describe the same product.

Grid

The term grid in this report is used interchangeably with mesh. The term grid as it applies to the soil reinforcement in the MSE wall system discussed in this report is implied to mean galvanized grid as all steel grids supplied to the project, whether comprised of smooth wires or deformed wires must be galvanized.
Reinforcement
A general term that applies to a specific material such as a grid or a steel strip used to reinforce the backfill in the MSE wall system and to anchor the wall facing panels.

Smooth Wire
Plain, undeformed steel wire used in the construction of grids.

Deformed Wire
Deformed wire is steel wire that has been mechanically pressed to form ridges along the length of the wire. The ridges increase the bond between the wire and the surrounding material - in this case backfill.

Mud Slab
A mud slab is a relatively thin layer of concrete, generally 75 to 100 mm, placed on top of the MSE backfill which serves as a solid platform for constructing the abutment footing. The mud slab is not a structural component of the bridge.

Embrittlement
Embrittlement is a loss of ductility of a material, making it brittle and therefore susceptible to a sudden non-ductile failure.

Strain Age Embrittlement
Strain age embrittlement refers to a loss in ductility accompanied by an increase in hardness and strength that occurs when low carbon steel is aged following plastic deformation. Aging refers to a change in material property manifested by exposure to a particular environment for some undefined interval of time.

Cold Working
Cold working refers to the altering of the shape or size of a metal by plastic deformation. Hardness and tensile strength increase with the degree of cold work while ductility and impact resistance are lowered.
2 Introduction

During the morning of 2011 November 20th, the precast concrete facing panels from a segment of a mechanically stabilized earth (MSE) wall collapsed onto Highway 97 under the Westside Road Underpass #7558 south of Kelowna. The collapse is shown in Figure 1.

Buckland & Taylor Ltd. (B&T) has been retained by the BC Ministry of Transportation and Infrastructure (Ministry) to complete a forensic investigation into the failure and to prepare and submit a detailed forensic report. The report presented herein, details the findings and conclusions regarding the cause of the failure.

Figure 1: Collapsed Facing Panels and Spilled MSE Wall Backfill
3 Project Description

The Westside Road Interchange Project is a partnership between Westbank First Nation and the Province of British Columbia. The new interchange is located approximately 1.6 km west of Okanagan Lake and replaces the original traffic intersection at Westside Road and Highway 97 with a grade-separated interchange at Westside Road and an overpass at Nancee Way. At the time of collapse, the project was still under construction with the Westside Road works being essentially complete. The construction works associated with Nancee Way had not yet commenced.

The owner for the construction contract is Westbank First Nation (WFN) and the owner's engineering team consists of:

- Urban Systems Ltd. (USL) - Lead Consultant and Roadway design;
- CWMM Consulting Engineers (CWMM) - Sub-Consultant Structural design;
- Golder Associates (Golder) – Sub-Consultant Geotechnical; and
- EBA Engineering Consultants (EBA) - Sub-Consultant Material Testing (QA).

The prime construction contractor for the permanent works is Ledcor CMI Ltd. (Ledcor). The subcontractors relevant to the works addressed in this report are:

- SSL Construction Products (SSL); and
- BC General Contracting (BCG).

At the new Westside road interchange, Highway 97 is considered to be oriented north-south and Westside Road is considered to be oriented east-west. Earth fill embankments have been raised to elevate Westside Road on the east and west approaches to meet the Underpass grade. At the time of the collapse, there were three MSE walls constructed on site and shown in Figure 2 (the MSE walls associated with Nancee Way had not been constructed yet). Appendix A includes locations and sections of the MSE walls and these are generally described as follows:

- MSE wall to support the Highway 97 Southbound off ramp embankment and northwest side of Westside Road (also referred to as Wall A);
- MSE wall to support the Westside Road Underpass west abutment and the Highway 97 Southbound off ramp embankment (also referred to as Wall B); and
- MSE wall to support the Westside Road Underpass east abutment (also referred to as Wall C).

**Figure 2: Site Plan**

The new Westside Road Underpass #7558 over Highway 97 is a two span bridge consisting of precast concrete box stringers with a concrete deck overlay. The box stringers are supported on laminated elastomeric bearings with dowels in foam filled holes at the perched abutments on top of the mechanically stabilized earth (MSE) walls. The bridge superstructure is fixed to the center pier and is partially free to expand at both abutments. The superstructure accommodates two 3.6 m wide lanes and a 1.5 m wide shoulder in each direction, a central 1.5 m wide raised island and a 2.14 m wide sidewalk along the north side separated from traffic by a concrete parapet.
Shoring towers were installed under both spans of the Underpass superstructure near the abutments shortly after the collapse of west abutment MSE wall facing panels (Wall B). Over the next two months, the collapsed facing panels at Wall B were removed along with the spilled backfill and Wall C was deconstructed. Some remaining Wall B facing panels were also deconstructed and Walls B and C were then repaired.
4 MSE Walls

4.1 Design Approach

The Owner for the Westside Interchange Project specified the use of MSE walls for the project and provided the Contractor with a general layout of the walls as well as a detailed design of the bridge. The Owner, through CWMM, specified the bridge loadings at the abutments. The Owner also specified a maximum overall soil bearing pressure on the subgrade under the MSE walls.

As is typical for MSE walls, the Contractor procured them from a list included in the Contract Documents which was based on the Ministry’s Recognized Products List. The Owner's geotechnical engineers provided allowable overall soil bearing pressures on the subgrade and internal/external design soil parameters. The wall supplier then produced a design and supplied the proprietary MSE wall materials. The MSE walls were required to meet Ministry and AASHTO standards (refer to Appendix F and Appendix G).

4.2 Westside Road MSE Walls

The MSE walls chosen by the Contractor for the Westside Road project are the MSE PLUS™ system designed and supplied by SSL Construction Products of Scotts Valley, California. They consist of rectangular precast concrete facing panels connected to galvanized steel reinforcing grids (the reinforcing grids and the back faces of the precast facing panels are shown in Figure 4).

The facing panels are connected to the grids through bends in the wires, with the bent wires inserted between pairs of steel loops protruding from the back side of each facing panel. The wires are then locked in place by a steel pin slid between the loops and the inside of the wire bend. An example of this connection is shown in Figure 5 with deformed wires. The connection detail is similar regardless of the wire type (smooth or deformed) and regardless of the wire size used in the reinforcing grids supplied to the project.
Figure 4: SSL MSE PLUS™ Wall under Construction (Wall A)

Figure 5: Grid to Panel Connection (Wall B)
5 Site Observations

5.1 Wall B Post Collapse

B&T staff arrived on site (2011 November 22) shortly after the collapse of Wall B facing panels. Figure 1 shows the state of the wall shortly after the collapse and Figure 6 shows the exposed reinforced backfill after the Contractor had removed the collapsed wall panels and loose fill.

Notable observations regarding the collapse are as follows:

• An approximate half to one metre wide column of MSE wall backfill spilled with the collapsed facing panels;

• The Underpass did not appear to be damaged as a result of the wall failure;

• The abutment spread footing was supported on the remaining reinforced backfill without the facing panels, but was partially undermined by the collapse;

• The collapse was limited to the section of wall (facing panels and some of the backfill) immediately beneath the Underpass;
• Where the facing panels collapsed, virtually all of the grid wires were broken at their 90° bends at the connections to the facing panels;

• Most of the failure surfaces of the broken grid wires were significantly corroded as shown in Figure 7;

• Most of the failure surfaces of the grid wires contained black deposits on the broken faces as shown in Figure 7; and

• All grid wires that failed were deformed 12.8 mm (1/2 in.) diameter wire (denoted D20).

Figure 7: Broken Wire Fracture Face (Wall B Sample)

5.2 Deconstruction of Wall C

Following the collapse of Wall B facing panels, the Contractor performed some exploratory excavation along the top of Wall C in front of the east abutment. The majority of wires were broken and the breaks occurred at the same locations as found in Wall B namely at the wire bends (see Figure 8). The Contractor then dismantled Wall C one panel at a time.
B&T staff observed most of the dismantling of Wall C and recorded a number of conditions that appeared to be relevant to the collapse of the Wall B facing panels.

![Image](image.png)

**Figure 8: Broken Wires at East Abutment (Wall C)**

The recorded observations are summarized in point form below.

- The majority of wires were broken wires at the 90° bends pictured in Figure 8 and Figure 9, particularly in the upper half of the wall;
- Rust and black deposits, similar to those seen on fracture faces of broken Wall B wires (Figure 7), were present on the fracture faces of the Wall C wires;
- In addition to the broken wires, many of the remaining wires in Wall C contained partial depth cracks at the 90° bend locations;
- The bridge east abutment that rests on top of Wall C had settled relative to the facing panels;
- The backfill immediately behind the facing panels was often suspended on a reinforcing grid with voids below; and
- The mud slab under the abutment, shown in Figure 9, settled below the level of the top wire to facing panel connections.
Figure 9: East Abutment Mud Slab Settlement and Broken Wires (Wall C)
6 Failure Investigation

Wall B grid wires failed at the connection to the loops embedded in the rear face of the precast facing panels. With little lateral support, the panels collapsed onto Highway 97 spilling some of the backfill.

An investigation was completed to:

• Observe the deconstruction of Walls B and C;
• Collect existing and obtain new survey data;
• Determine the metallurgical properties of the steel grid wires;
• Independently check the design of the MSE walls and sizing of the grids;
• Independently check the geotechnical parameters (anticipated settlements, soil pressures);
• Review construction procedures to confirm whether these had any impact on the failure; and
• Review backfill material to confirm its influence on the anticipated behavior of the MSE wall system.

Levelton Consultants Ltd. were engaged to perform metallurgical testing on behalf of B&T on wire samples taken from the Westside Interchange MSE walls. A summary of Levelton's findings follows (a more detailed report is included in Appendix M).

6.1 Metallurgical Testing

6.1.1 Specified Wire Type

According to the construction contract, the supplier of the MSE wall system must complete the design and take responsibility for the design of the MSE walls and their details. SSL design drawing RW-03, Section 5.2 specifies that cold drawn steel rods (wires) must be used for the soil reinforcing grids.

According to SSL design drawings, two sizes of longitudinal wires were required for the various reinforcing grids. These are a 12.8 mm (0.505 in.) diameter wire (0.20 in$^2$ cross sectional area) designated W20 and a 9.5 mm (0.374 in) diameter wire (0.11 in$^2$ cross sectional area) designated W11.
While galvanized smooth wire mesh (classified as "W" wires) was specified on both the SSL design drawings and the SSL shop drawings for the reinforcing grids, the type of wire actually furnished to the project consisted of deformed wires (classified as "D" wire).

The MSE wall designer used the higher strength properties of D wire in the MSE wall design calculations provided but the design and shop drawings nonetheless specified the W wire.

6.1.2 Visual Microscopic Examination

Levelton visually examined 5 broken wire samples retrieved from the collapsed wall under a microscope. The following observations are highlighted:

- The fracture plane was consistent with a brittle type fracture;
- Rust was visible along a portion of the fracture surface;
- A black deposit (referred to as "smut" in Levelton Report) was also visible along a portion of the fracture surface; and
- D20 wire (Levelton sample # 4-3) contained a crack in the steel that pre-existed the galvanizing process.

In addition to the microscopic examination, broken wires were found in stockpiled grid reinforcement on site as illustrated by Figure 10.

The rust contained on the fracture faces of a majority of broken wires indicates that the fracture had occurred several months prior to the facing panel collapse. The presence of black smut is believed to result from the pickling process prior to galvanizing albeit, this is not conclusive.
6.1.3 Wire Bend Radius

ASTM A82 for plain wires and A496 for deformed wires, provide quality control (QC) material minimum bend radius requirements. The bend radius requirements are more restrictive for D wire (deformed) than for W wire (smooth).

The SSL shop drawings for the MSE walls on this project show a bend detail for the longitudinal wires with an 11 mm inside bend radius for all wire sizes and with no adjustment for differing wire types (plain vs. deformed).

Table 1 below provides a summary of the bend radii applicable according to ASTM standards to the plain and deformed wires under discussion.

<table>
<thead>
<tr>
<th>Standard</th>
<th>Wire</th>
<th>Minimum Bend Radius</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSL Specified</td>
<td>All</td>
<td>11 mm</td>
</tr>
<tr>
<td>ASTM A82 (ASTM 1060)</td>
<td>W11</td>
<td>9.5 mm</td>
</tr>
<tr>
<td></td>
<td>W20</td>
<td>12.8 mm</td>
</tr>
<tr>
<td>ASTM A496 (ASTM 1060)</td>
<td>D11</td>
<td>19 mm</td>
</tr>
<tr>
<td></td>
<td>D20</td>
<td>25.6 mm</td>
</tr>
</tbody>
</table>
The 11 mm bend radius for deformed wires supplied to the project does not meet ASTM A496 QC material bend requirements.

We note that ASTM A1060 was in force at the time of the Contract signing and replaces ASTM A82 and A496. ASTM A1060 applies to galvanized steel welded wire reinforcement and contains no changes to the minimum bend radii shown in Table 1 above.

6.1.4 Chemical Analysis

ASTM standards do not specify a chemical composition for plain nor for deformed wires. Wires samples were tested and results compared to the requirements of CSA 30.18 criteria for avoidance of strain age embrittlement in hot dip galvanized reinforcing steel.

Notably a higher carbon content and the presence of grain refining elements reduces the likelihood of strain age embrittlement. The material tests conducted on samples collected from the site indicate that:

- wire steel material was low carbon ranging from 0.06 to 0.09 % wt; and
- negligible grain refining elements had been utilized to alloy the steel material.

According to CSA 30.18, the risk of strain age embrittlement is then significant since the material has low carbon content and is produced without grain refining elements.

6.1.5 Impact Testing

Wire samples taken from stockpiled reinforcing grids on site were impact tested as described in the Levelton Report in Appendix M. Notably, three out of three D20 wires tested fractured when struck with a hammer at a low test temperature of -22°C. The test was analogous to CAN/CSA G164 test for embrittlement. The test results indicate that the wire material loses ductility at low temperatures.

6.1.6 Tensile Testing

Deformed wires were tested for tensile strength and the wires met the tensile strength requirements of ASTM A496-97.

Tests were also conducted on simulated panel connections (with D20 wires) and on additional samples of D20 wires bent to 90°. The tests on these indicated that the wires failed at the 90° bends at loads that were 27% of the wire ultimate tensile capacity.
6.1.7 Galvanizing and Embrittlement

The supplied steel grid reinforcement was required to be galvanized after fabrication in accordance with ASTM A123-09 in order to meet the durability and service life requirements of the project.

Strain age embrittlement is a function of time and temperature and while it occurs very slowly at ambient temperatures, it occurs rapidly at the 450°C to 460°C temperatures of the galvanizing process. The high heat of the galvanizing process, combined with cold working by bending the wires to a radius less than that specified by ASTM A496 resulted in strain age embrittlement of the deformed bars at the bend locations.

ASTM A123-09 Section 5.2 references specification ASTM A143 Safeguarding Against Embrittlement of Hot-Dip Galvanized Steel Products and states that it "shall be complied with in both design and fabrication."

The following excerpts from ASTM A143 are highlighted:

• For cold temperature implications: Section 4.4 "Low temperatures increase the risk of brittle failure of all plain carbon steels including steel that has been galvanized."

• For severity of cold work and embrittlement: Section 6.1 "A cold bending radius of three times (3x) the section thickness … will ordinarily ensure satisfactory properties in the final product… embrittlement may occur if cold bending is especially severe."

• For design responsibility: Section 8.1 "Design of the product and selection of the proper steel to withstand normal galvanizing operations without embrittlement are the responsibility of the designer."

• For embrittlement testing requirements: Section 9 Testing for Embrittlement gives a variety of test options and notes that "the tests given in 9.2, 9.3, 9.4, or 9.5, or a combination thereof, shall apply."

The supplier's construction documentation attests to having galvanized the grids to ASTM A123 but there was no indication of having completed any embrittlement testing as required by ASTM A143.
6.2 Backfill Placement and Compaction

Owner’s field staff provided a description of the actual method used to construct the MSE walls which corresponded substantively to the method described on SSL drawings. This information is in addition to fairly extensive photographic records that have been gathered covering the construction period of the three MSE walls. Wall construction is also discussed under Section 7.2 of this report.

6.2.1 MSE Fill Placement

For the majority of the MSE backfill, maximum 250 mm thick lifts of granular backfill material were placed and compacted by heavy mechanized machinery to form the reinforced backfill core of the MSE wall system. Heavy compaction was not applied to the backfill within 900 mm of the precast panels as SSL drawings limit the compacting effort in this area to a light tamping. Plate tampers were used to compact backfill placed within 900 mm of the facing panels.

Although note D of ‘Erection of Subsequent Courses’ on SSL drawing RW-05 states that "Fill may be sloped downward toward the panels within 610 of the back face…", construction photographs indicate that the region of sloped fill extended to at least 900 mm from the facing panels.

Figure 11: MSE Wall Construction (Wall A)
The lift thickness of the backfill placed in the 900 mm zone appears to have been greater than 250 mm and up to 750 mm thick, matching the vertical spacing of grid layers (and their panel connections). The backfill reinforcing grids were installed and attached to the wall facing panels before the backfill was placed in the 900 mm zone as shown in Figure 11.

The larger aggregates in the backfill were retained on the grid in some areas, preventing backfill from sifting through creating voids in the backfill under the grids. Accordingly, the backfill compaction in the 900 mm zone behind the facing panels is light at best and poor in places where voids formed.

The following observations were consistent with the assumption of a poor to light compaction of the 900 mm zone of backfill immediately behind the wall facing panels:

• When the Wall B facing panels collapsed, this approximately 900 mm wide zone of reinforced backfill spilled along with the facing panels, but the well compacted fill zone remained standing (and in fact continued to provide support for the Underpass abutment);

• During the deconstruction of Wall C, the fill immediately behind the wall panels in the 900 mm wide zone would slough as the panels were removed while the heavily compacted fill beyond remained; and

• During the deconstruction of Wall C, voids were seen in the undisturbed soil behind the precast facing panels, usually immediately below the level of a reinforcing grid.

6.2.2 Grid Reinforcement Placement

Backfill was placed and compacted in keeping with SSL procedures shown on SSL drawings but the methodology did not follow the AASHTO Standard Specifications, Section 7.6.4.3. Construction (of MSE Walls) which states the following:

"At each level of soil reinforcement, the backfill material shall be roughly levelled to an elevation approximately 0.1 foot (30 mm) above the level of connection at the facing before placing the soil reinforcement."

The backfill in the 900 mm zone behind the wall facing panels was not placed before grid reinforcement was installed, as shown in Figure 11 and Figure 12, so a consistent level backfill surface was not provided as required by the AASHTO specification.
Construction photographs show that in many instances, the backfill beyond the 900 mm wide zone behind the facing panels was placed to a lower elevation than the panel connections which required the hooked ends of the reinforcing grid wires to be bent upwards in order to make the connection to the facing panels contrary to the AASHTO specification. Figure 11 and Figure 12 provide examples of grid reinforcement bent upwards in order to align with and connect to the facing panels.

Figure 12: MSE Wall Construction (Wall C)

6.3 Settlements

6.3.1 Abutment Settlement

The Underpass abutment spread footings rest on top of the backfill immediately behind the MSE wall facing panels. The toes of the west and east abutment spread footings are intended by design to be 818 mm and 308 mm respectively away from the facing panels (refer to Figure 13). Section views are included in Appendix A.
Figure 13: East Abutment Spread Footing

Prior to deconstruction of MSE Wall C, the east abutment was seen to have settled relative to the MSE wall facing panels. This relative settlement was substantiated by survey results, both taken prior to the deconstruction of the wall and from survey results taken during construction that were provided to B&T; and by field observations showing that the mud slab of the east abutment was lower than the level of the top layer of mesh (see Figure 9).

The overall settlement of the east and west abutments was derived from field surveys and is summarized in Table 2 below.

Table 2: Abutment Settlement

<table>
<thead>
<tr>
<th>Location</th>
<th>2011 Dec 09 Survey</th>
<th>2012 Jan 24 Survey</th>
</tr>
</thead>
<tbody>
<tr>
<td>North end of West Abutment</td>
<td>68 mm</td>
<td>74 mm</td>
</tr>
<tr>
<td>South end of West Abutment</td>
<td>49 mm</td>
<td>60 mm</td>
</tr>
<tr>
<td>North end of East Abutment</td>
<td>49 mm</td>
<td>58 mm</td>
</tr>
<tr>
<td>South end of East Abutment</td>
<td>48 mm</td>
<td>66 mm</td>
</tr>
</tbody>
</table>
The original construction survey information obtained for the purpose of setting deck screed elevations also revealed significant settlement of the west abutment during construction. These survey results are summarized in Table 3 below:

### Table 3: Abutment Settlement at Time of Setting Deck Screeds

<table>
<thead>
<tr>
<th>Location</th>
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</tr>
</thead>
<tbody>
<tr>
<td>North end of West Abutment</td>
<td>-7 mm</td>
</tr>
<tr>
<td>South end of West Abutment</td>
<td>32 mm</td>
</tr>
<tr>
<td>North end of East Abutment</td>
<td>61 mm</td>
</tr>
<tr>
<td>South end of East Abutment</td>
<td>17 mm</td>
</tr>
</tbody>
</table>

The abutments are positioned in close proximity to the facing panels and require firm support from the underlying backfill so that the necessary design bearing capacity can be developed. Since a portion of the 900 mm wide zone of light to poorly compacted backfill extended under both abutments, the backfill could not provide the necessary bearing capacity nor soil stiffness required to prevent excessive settlement.

### 6.3.2 Global Site Settlement

Surveys were completed in order to assess the extent of global site settlements. The following was surveyed:

- Highway 97 profile along the eastern and western edges and along the median;
- MSE wall leveling strips; and
- Center pier.

The top of Highway 97 pavement along the western edge of the highway was 9 to 30 mm below the design profile. The median profile of the highway to the north of the bridge was 22 to 40 mm below the design profile (excluding one anomalous survey point).

The pavement adjacent to the pier was 41 to 86 mm below the design profile while the median pavement to the south of the bridge was 52 to 80 mm below the design profile.
The Highway 97 pavement along the eastern edge of the pavement (actually, the edge of the abutting on ramp) was 0 to 57 mm below the design profile; four points along the south east edge of the pavement indicate the pavement elevation to be above the design profile. This latter anomaly is attributed to the approximation used to determine the alignment of the spiral portion of the control line curve that governs the geometry of this portion of the roadway.

The low pavement profile recorded at the center pier conflicts with the 10 to 12 mm surveyed settlement of the center pier and the surveyed elevation of the bridge at the pier. It is clear though from surveyed pier elevations and girder elevations, that the bridge pier underwent no significant settlement and as such, it appears that the Highway 97 median pavement elevations indicate that the road was constructed at or near the surveyed elevations, lower than required by design.

The MSE Wall B leveling strip contains a step in its profile. The leveling strip was surveyed and found to be 21 mm lower than the design elevation to the south of the step and 38 mm lower to the north. Since an abrupt change in the settlement at the step is highly unlikely, the difference in surveyed settlements is most likely a result of construction tolerances in setting the top of the leveling strip elevations.

The MSE Wall C leveling pad elevations were derived from a survey of the top of the MSE wall facing panels. The leveling pad elevations ranged from being 1 mm high to 34 mm below the design elevation.

The Highway 97 survey results are inconclusive in defining a site global settlement however the magnitude of any global settlements that could be inferred from the survey results would be well below settlements that are accommodated by MSE walls.

Additional component surveys are included in Appendix K.

### 6.4 Wire Break Mapping

The location of the wire breaks in MSE Wall B and Wall C were mapped (see Figure 14 and Figure 15 and Appendix L). Red wire locations indicate that a broken wire was recorded at the location while a green location indicates that the wire was found to be intact.
Figure 14: Wall B Wire Break Mapping (After Collapse)

Figure 15: Wall C Wire Break Mapping (During Deconstruction)
The pattern of wire breakage coincides with the location of the abutment spread footings. The wire break mapping coupled with the screed elevation survey data and survey data obtained during the course of the forensic investigation, indicate that the poorly to lightly compacted backfill in the 900 mm zone behind the facing panels progressively settled as the walls were constructed and loads were applied incrementally to the abutments. This is also confirmed by the geotechnical analyses completed by Naesgaard (see Appendix N).

The portion of well compacted backfill supporting abutment footings also settled as abutments were loaded as this well compacted backfill was not adequately confined by the loosely to poorly compacted backfill. This is further discussed in the Naesgaard report included in Appendix N.

The backfill settlement, relative to the facing panels, is greatest at the top of the wall directly beneath the spread footing, remains constant till about mid-height of the wall and then lessens progressively towards the base of the wall. This is evidenced by a greater number of wire breaks at the top to mid-height of the wall and fewer wire breaks at the base of the wall.

The settlement of the abutments, screed adjustments and post-construction settlements are discussed in more detail in Appendix K.

The wire break patterns indicate that many wires were broken prior to the commissioning of the Underpass, most likely breaking during construction. This is further supported by the rust deposits found on crack surfaces discussed earlier. Given the number of broken wires found in Wall C, its facing panels were on the verge of a collapse.

6.5 Earthquake

A small earthquake was recorded at 5:09 am on November 18th, 2011, centered about 180 km south of the bridge site with a magnitude of 4.3. Although felt in the Okanagan region, there were no reports of damage caused to infrastructure.

While this seismic event may have slightly jarred the MSE walls at the Westside Interchange, the fact that a majority of broken wires had rust on their fractured faces indicates that the wires were broken well before the earthquake occurred.

It is conceivable that the earthquake could have caused some damage to the remaining wires however there is no evidence to support such speculation.
7 Geotechnical Review

Naesgaard Geotechnical Ltd reviewed the design of MSE Walls and independently estimated settlements based on the as constructed conditions at the site. The design review was conducted to AASHTO Standard Specifications for Highway Bridges, 17th Edition - 2002 (‘AASHTO 2002’) and included the verification of the internal and external wall stability as well as the factor of safety of the tensile grid reinforcement. In addition, Naesgaard Geotechnical completed the following:

• A comparison of observed versus calculated settlements;
• Impact assessment of the poorly to loosely compacted backfill zone; and
• A review of the MSE wall construction process.

The analysis software FLAC was used to compute settlements and to investigate the influence of the poorly to loosely compacted backfill zone.

7.1 MSE Wall Design

7.1.1 Dead Load

The dead load of the bridge was found to be higher than anticipated from the bridge design because of the thicker deck that was placed to compensate for bridge settlements that had occurred prior to deck construction. An increase in the bridge's dead load of 15% was assumed for the purpose of conducting the geotechnical analyses.

7.1.2 External Stability Check

The SSL MSE wall design was checked for external stability. Both the foundation soil parameters provided in the Golder August 2009 Technical Memorandum and the Contract Special Provisions were used. MSE Walls B and C were found to meet the AASHTO 2002 requirement for external stability using both sets of soil parameters.

7.1.3 Internal Stability Check

The internal stability check assesses the factor of safety for wire tensile strength and the pull out resistance of the reinforcing grids from the backfill. The wire tensile strength was evaluated assuming wires to be in a new condition and after material losses occurred from corrosion at the end of the structure's design life. Based on the internal stability check:
• The factor of safety for the non-corroded wire tensile strength exceeds AASHTO 2002 requirements;
• The factor of safety after the design life of 100 years, assuming full corrosion losses, was slightly below AASHTO 2002 requirements; and
• The pull out resistance of the reinforcing grids meets the AASHTO 2002 requirements.

Accordingly, the size and length of the grid reinforcement meets AASHTO 2002 requirements with an exception as noted in the second bullet above.

7.1.4 Abutment Location

The centerlines of the east and west abutments are located 1108 mm and 1618 mm respectively from the inner face of the MSE wall facing panels. According to AASHTO 2002 clause 7.5.4 the centerlines of abutment bearings must be located in plan, at least 3.5 feet (1067 mm) from the outer face of the facing panels. AASHTO 2002 clause 7.5.4 also specifies that footings must be located a minimum of 6 inches (150 mm) from the back of the wall facing panels.

The Westside Road Underpass East and West Abutments have been located in accordance with AASHTO 2002 clause 7.5.4 requirements.

7.2 MSE Wall Construction

SSL specifies a construction procedure for its MSE walls shown on SSL drawing RW-05. The procedure includes the following requirements:

i. "do not place the first lift of backfill directly against the panels" (Note I);

ii. "compact the zone within 910 of the back of the wall … by using light mechanical tampers …" (Note I);

iii. The backfill between grid layers slopes down towards the back of the facing panels. When subsequent backfill lifts are placed, the backfill spills through the grid to fill the cavity between the lower backfill layers and the back face of the panels (Figure 4);

iv. "Fill may be sloped downward toward the panels within 610 of the back face..." (Note D of 'Erection of Subsequent Courses'); and

v. "compaction tests should not be taken within 910 of the panels" (Note K).
The prescribed construction methodology for the MSE walls result in a zone of backfill behind the MSE walls that can be at best, only lightly compacted.

### 7.2.1 Compaction Requirements

The Highway Innovative Technology Evaluation Center (HITEC) conducted an evaluation of the SSL MSE Plus system in 1999 and this is summarized in a document titled "Evaluation of the SSL MSE Plus retaining wall system". While the evaluated system had a somewhat different connection detail between the steel reinforcing grids and the facing panels, the review and commentary on backfill placement is valid for the current system.

HITEC was created in 1992 to speed the introduction of innovative technologies into the highway marketplace in the USA by a collaboration of The Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), the National Association of County Engineers, the American Public Works Association, and the Transportation Research Board (TRB)\(^1\).

HITEC emphasized that in the construction of MSE walls it is important to adequately compact the backfill below the grid reinforcement. In particular, this must occur beneath the panel connection in order to prevent additional bending forces in the grids. To quote from the publication:

"As with all MSE wall systems, it is important that the construction specifications under “Backfill Placement” be strictly enforced. Specifically the following must be enforced.

At each reinforcement level, the backfill shall be placed to the level of the connection. Backfill placement methods near the facing shall assure that no voids exist directly beneath the reinforcing elements.

Failure to strictly enforce this specification may introduce bending forces in the grid reinforcement and significantly increases the difficulty of maintaining the required overall verticality.”

---

\(^1\) http://www.fhwa.dot.gov/publications/publicroads/97novdec/p97nov47.cfm, Nov/Dec 1997 Vol. 61, No. 3, Three Years Later and Exceeding Expectations: Highway Innovative Technology Evaluation Center, by Peter Kissinger and Nicole Testa
The US Department of Transportation FHWA document FHWA-NHI-00-043 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines", Section 9.3, Construction Control: Reinforced Fill Placement, Compaction, p.330, also emphasizes the compaction of the area behind the facing panels to prevent:

“chimney-shaped vertical void immediately behind the facing elements”

Neither of these guidelines appears to have been followed, resulting in a zone of poorly to lightly compacted fill behind the MSE wall facing panels that lacked the capability to properly support the abutments. The lack of uniform placement and compaction of backfill is illustrated by Figure 16 where the grids are not supported by backfill in the zone behind the precast facing panels.

Figure 16: MSE Wall Construction

7.3 Calculated Settlements

The following settlements were computed based on the geotechnical parameters provided:

• Settlement of the west abutment (above Wall B);
• Settlement of the east abutment (above Wall C);
• Settlement of the unreinforced concrete leveling strip supporting the MSE Wall panels; and

• Settlement of the central pier of the Underpass.

Reported settlements are inferred from theoretical design elevations. As-built survey information has not generally been recorded with the exception of top of girder elevations taken to set deck screeds and some top of coping elevations taken for handrail detailing.

7.3.1 West Abutment

The predicted west abutment settlement at Wall B from FLAC analysis was in the range of 20 to 45 mm along with a tilt of 0.1 to -0.2%. A positive tilt is towards the pier, the stated tilt is relative to a plumb vertical line and a tilt of 100% is equal to 45°. The analysis accounted for the 900 mm zone of poorly to lightly compacted backfill behind the wall facing panels and only includes the settlement within the depth of the MSE Wall backfill.

Survey results indicate that the west abutment had settled 49 to 74 mm and tilted 0% to 0.4% towards the pier. The predicted abutment settlements do not account for wire breakage which would result in greater predicted settlements. As such, the predicted settlements appear to be in reasonable agreement with the observed settlements and movements.

7.3.2 East Abutment

Survey results indicate that the east abutment at Wall C had settled 48 to 66 mm and tilted 1.6% to 3.4% towards the pier.

No calculations or modelling was performed for the east abutment; however, settlements and rotations were extrapolated from the west abutment analysis. The resulting settlement estimate reasonably agrees with surveyed values considering that wire breakage was not modelled. The actual tilt is greater than that predicted and this is also attributed to wire breakage which would increase the extent of tilt expected as there would be less support under the toe of the footing.

7.3.3 Pier

Survey results indicate that the pier had settled 10 to 12 mm. The calculated settlement of the pier spread footing is about 3 to 4 times greater than that surveyed.
This implies that the site soil stratum appears to be stiffer than what was inferred from the oedometer tests for the analysis.

### 7.3.4 Leveling Strips

The top of the Wall B levelling strip was surveyed and found to be 21 to 38 mm lower than the design elevations. Calculated immediate (elastic) settlements were 30 mm whereas settlements calculated using consolidation theory and oedometer tests were 100 mm. The site soil strata are then stiffer than what was inferred from the consolidation theory and oedometer test results.

### 7.3.5 Site Settlement Tolerance

Properly designed and constructed MSE walls are flexible and accommodating structures that typically tolerate global site settlements in the range of 100 mm to 200 mm without failure along with corresponding differential settlements. This has been demonstrated by the resilient behaviour of MSE walls during major earthquakes (refer Appendix N Naesgaard Geotechnical Report). Both theoretically computed and actually surveyed site settlements are at or below the 100 mm value.

Differential settlements along the length of the wall are inferred from the levelling strip surveys and the magnitudes of these do not pose a concern.

### 7.4 Compaction Influence

The impact of the poorly to lightly compacted zone of backfill (termed "loose zone" in the Naesgaard Report) behind the precast facing panels was investigated. The investigation consisted of FLAC analyses on a model assuming full compaction of all backfill - the baseline case and, analyses on a model that assumed a 1.5 meter wide zone of poorly compacted fill. The results of the analyses indicate that with the presence of a poorly compacted zone:

- Abutment settlements are higher – 53 mm predicted by analysis;
- Grid layers displaced downwards up to 100 mm within the "loose zone" relative to the facing panels;
- Grid vertical displacements are greatest from top of wall to mid height; and
- Grid displacement pattern is consistent with wire break mapping.
The abutments do not settle to the same extent as the grid layers as the back of the
abutment footing is supported on well compacted backfill and the abutment is
restrained from tilting further by the bridge deck to which it is pinned with dowels.
8 Additional Aspects

The following additional aspects of the Westside Road MSE Wall design and construction have been reviewed and are discussed in more detail in the Appendices:

• Summary of the construction timeline and consequent applications of dead and live loads to the MSE Walls supporting the bridge abutments (Appendix E);

• Information supplied within the Contract Special Provisions for the purpose of the design of the MSE Walls (Appendix F) including provided loads, allowable bearing pressures and soil parameters, corrosion protection requirements, Bridge End Fill specifications for placement and compaction of the MSE Wall backfill, and requirements for site presence of the MSE Wall supplier;

• Details of the MSE Wall construction method (Appendix H);

• Review of samples of the construction documentation and QC and quality assurance (QA) records (Appendix J);

• Summary and discussion of the survey results (Appendix K);

• Mapping and discussion of the wire breaks recorded for the MSE walls during their deconstruction (Appendix L);

• Independent materials testing and investigations by Levelton Consultants, Ltd. The focus of this section is on the steel reinforcing grid (Appendix M); and

• Independent geotechnical review by Naesgaard Geotechnical, Ltd. This addresses the parameters provided for the design of the MSE walls, their internal and external stability and settlements (Appendix N).
9 Summary & Conclusions

9.1 Summary

• Deformed wires (denoted D) were supplied to the project contrary to the specified smooth wires (denoted W).

• Wire bend radii were significantly less than those specified by ASTM standards resulting in greater cold working and resulting embrittlement of the wires at their bends.

• Because the wires were cold worked at their bends, cracks formed at the bends in some D20 wires during the bending process.

• The galvanizing process resulted in accelerated embrittlement of the cold worked wires and as a result, the ductility and fracture toughness of the material decreased.

• There is a lack of evidence that testing was conducted on the galvanized grid wires for embrittlement as required by ASTM A143.

• Steel is typically more brittle at lower temperatures and the wall failure occurred after the coldest nights in Kelowna following the wall construction.

• Granular backfill material was placed in such a way as to create a 900 mm wide zone of poorly to lightly compacted backfill directly behind precast wall facing panels.

• The reinforcing grid layers were placed lower in many cases than required to vertically align their ends with the panel connections.

• A light compaction effort was applied to a 900 mm wide zone of backfill behind the precast facing panels that in many cases depressed or further depressed and stressed the wire mesh at its connections to the panels.

• The poorly to lightly compacted backfill zone did not provide the necessary bearing capacity for the abutment footings and the abutments settled under load relative to the facing panels breaking several rows of grid wires at the 90° bends at the panel connections near the top of the wall.
• Rust deposits on wire fracture faces indicate that the majority of wires were broken either during wall construction, including backfill placement and compaction, or soon thereafter as loads were applied to the abutments during construction.

• Global site settlements are within tolerable limits for MSE wall systems and differential settlements along the wall are tolerable by MSE wall systems.

• The MSE wall design is generally adequate (with the exception of an undersized of the wires for the 100 year corrosion loss design case) but the grid connection is inadequate because of the sub-standard wire bend.

9.2 Conclusions

The cause of the collapse of MSE Wall B is the embrittlement of the supplied deformed wires in the reinforcing grids at their bends. Embrittlement resulted from both the wire bending and galvanizing processes.

The rupture of a majority of wires at the wire bends occurred during construction notably because:

• Some D20 wires contained cracks that originated during wire bending;

• The grid connections were subject to additional strain due to improper placement of the grid layers (not vertically aligned with connections);

• The grid connections were subject to additional strain due to inadequate backfill placement within a 900 mm wide zone behind the facing panels; and

• The abutment settled as it was loaded during construction breaking several rows of wire connections under the abutment.

As a result of the rupture of a majority of wires, the facing panels lost most of their lateral support and were unstable. The panels were likely on the verge of a collapse when the bridge was put into service.

The collapse of Wall B facing panels resulted from the fracture of remaining wires holding the panels likely due to the cold nighttime temperature just prior to the failure which reduced the fracture toughness of the wires particularly at their connections to the facing panels.

Wall C facing panels under the east abutment were on the verge of collapsing before the wall was deconstructed as the majority of wires in the upper portion of the wall under the abutment were broken.
10 Limitations

This report has been prepared for the sole use of the Ministry and shall not be relied upon by any other party for any purpose. This report addresses issues related to solely the SSL MSE Plus™ system. Any concerns the Ministry may have regarding other wall systems must be assessed on their own merits.

All other parties having an interest in any SSL MSE Plus™ retaining wall, including those on the Westside Interchange Project, must undertake their own inspections and investigations and reach their own conclusions.
Appendix A
Site Plan & MSE Wall Sections
WEST ABUTMENT AND MSE WALL B
CROSS SECTION AT L200G1 LINE
SCALE 1:30

EAST ABUTMENT AND MSE WALL C
CROSS SECTION AT L200G1 LINE
SCALE 1:30

FIGURE A2: MSE WALL SECTIONS

NOTE: ALL ELEVATIONS TAKEN FROM DESIGN DRAWINGS AND DO NOT REFLECT POST-CONSTRUCTION SURVEY WORK.
* ABUTMENT LOCATION AS CALCULATED FROM CMWN DWG 7758.3, SECTION 1.
Appendix B
CWMM Drawings
<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Description</th>
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<th>Index Pk.</th>
<th>Index Shear</th>
<th>Index Unit Weight</th>
<th>Index Void Ratio</th>
<th>Index Pore Ratio</th>
<th>Index Eot</th>
<th>Index Swell</th>
<th>Index Unit Weight</th>
<th>Index Void Ratio</th>
<th>Index Pore Ratio</th>
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</thead>
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</tbody>
</table>

MATERIALS CLASSIFICATION LEGEND

- **SYMBOL**: SK
- **SOIL TYPE**: CLAY
- **DESCRIPTION**: SOIL CLASSIFIED AS CLAY, HIGHLY FLOCCULATED
- **INDEX MATERIALS**: DENSITY, UNIT WEIGHT,VOID RATIO, PORE RATIO, EOT, SELL
Appendix C
SSL Drawings
C.1 Wall A
TYPICAL FRACURED RIB
ARCHITECTURAL PANEL DETAIL

SECTION A-A

NOT TO SCALE

SECTION B-B

NOT TO SCALE

JOINT DETAIL

NOT TO SCALE

NOTES:
1. ACTUAL THICKNESS TO BE SIMILAR TO WHAT IS SHOWN.
2. PANEL REINFORCEMENT, ANCHORS, LUGS, ETC., NOT SHOWN FOR CLARITY.
1.0 INTRODUCTION

The framed wall panels shall be made of a dry structured wall system with a thickness of 8 in. (200 mm) and should consist of oriented strand board (OSB) panels and a thermally insulated concrete core. The panels shall be firmly attached to the structural frame of the building to ensure stability and prevent movement.

2.0 WALL COMPONENTS

The framed wall panels shall consist of the following components:

- Oriented Strand Board (OSB) panels
- Thermally insulated concrete core
- Structural frame

The wall panels shall be designed to meet fire resistance requirements and be capable of withstanding the forces of wind and seismic activity.

3.0 GENERAL REQUIREMENTS

The framing of the wall panels shall be designed to meet the following requirements:

- Structural stability
- Fire resistance
- Weather protection

The framing shall be designed to withstand the forces of wind and seismic activity and be capable of withstanding the forces of fire and other natural disasters.

4.0 DESIGN REQUIREMENTS

The design of the framed wall panels shall meet the following requirements:

- Structural integrity
- Fire resistance
- Weather protection

The design shall be based on the provisions of the building code and the manufacturer's instructions.

5.0.1 CASTING PANELS

The panels shall be cast in a single piece, and the concrete shall be placed in the formwork before the reinforcement is placed. The panels shall be demolded and cured before being transported to the project site.

5.0.2 TOLERANCES

The tolerances for the framed wall panels shall be as follows:

- Height: ±0.3 in. (±7.5 mm)
- Width: ±0.3 in. (±7.5 mm)
- Thickness: ±0.3 in. (±7.5 mm)

The tolerances shall be measured at the project site and shall be in accordance with the specifications provided by the manufacturer.

6.0 MATERIALS

The materials used in the construction of the framed wall panels shall meet the following requirements:

- Concrete: Type I or II
- Reinforcing steel: ASTM A615 or A706

The materials shall be selected and tested in accordance with the provisions of the building code.

7.0 FINISHES

The finishes for the framed wall panels shall be selected based on the design requirements and the project specifications.

The finishes shall be applied in accordance with the provisions of the building code and the manufacturer's instructions.

8.0 QUALITY CONTROL

The quality control procedures for the framed wall panels shall be designed to ensure the quality of the finished product.

The quality control procedures shall be in accordance with the provisions of the building code and the manufacturer's instructions.

9.0 TESTING

The testing procedures for the framed wall panels shall be designed to ensure the quality of the finished product.

The testing procedures shall be in accordance with the provisions of the building code and the manufacturer's instructions.
6.0 DIRECTION SEQUENCE

The first step in the process of placing and finishing the concrete is to establish a direction sequence for the placement of the panels. This sequence will be determined in consultation with the client and may vary depending on the project requirements. The sequence should be based on factors such as the ease of人流流动, equipment access, and the desired appearance of the finished surface.

CONSTRUCTION OF THE CONCRETE PLACING AREA

The placing area should be properly prepared to ensure a smooth and efficient placement process. This includes setting up the necessary equipment, such as pumps and hose reels, and ensuring that the area is free from obstructions and debris. The placing area should be marked with醒目的标志 to indicate the area for concrete placement.

PLACEMENT OF THE PANELS

The panels are typically placed in a specific sequence to ensure a smooth and uniform surface. The sequence may vary depending on the project requirements and the desired appearance of the finished surface. The panels are usually placed in a sequence that allows for easy人流流动 and equipment access.

FINISHING THE CONCRETE

After the panels are placed, the concrete is finished to achieve the desired appearance. This may include smoothing, grading, and finishing with the appropriate tools and techniques.

7.0 CONCRETE FINISHING

The concrete is finished using a combination of hand tools and power tools. The finishing process includes smoothing, grading, and finishing with the appropriate tools and techniques to achieve the desired appearance.

8.0 CLEAN-UP

Once the concrete is finished, the area is cleaned up to ensure a safe and tidy work environment. This includes removing any debris and equipment, and ensuring that the area is ready for the next step in the construction process.

9.0 QUALITY ASSURANCE

The quality of the concrete work is monitored throughout the process to ensure that the project meets the required standards. This includes regular inspections and testing of the concrete to ensure it meets the specified requirements.

10.0 ARCHIVAL GENERATION

After the project is completed, a final report is generated to document the entire process. This report includes details about the project, the materials used, and the methods employed to ensure that the project is accurately documented for future reference.
### ALL DIMENSIONS ARE IN MILLIMETERS

<table>
<thead>
<tr>
<th>Plate</th>
<th>Dimension</th>
<th>Dimension</th>
<th>CM</th>
<th>Panel</th>
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<td>20</td>
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<td>T105</td>
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<td>T113</td>
<td>763</td>
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</tbody>
</table>

**NOTE:**
Top panels above 1000 mm need 1 horizontal band.
Top panels 1000 mm and below need 0 horizontal bands.
Top panels 1050 mm and below need 1 vertical band.
Top panels 1100 mm and below need 2 vertical bands.
Top panels 1150 mm and below need 3 vertical bands.
Top panels 1200 mm and above need 3 vertical bands.

**CONSTRUCTION:**
The bands shall be fixed to the frame in the same order as the panels are listed. Each panel shall be denoted with a "1" if the panel number is one or a "2" if the panel number is two, and so on. All panels above 2000 mm shall be denoted with a "X." The number of bands per panel shall be as indicated. If a panel is not used, no band will be required. Any panel with no bands will be denoted with a "X." In the case of panels needed, they are needed. If panels are needed, there will be a "0" in the column.

**DIAGRAM:**
The drawing shows the panel arrangement and the position of the bands. The bands are shown in black, and the panels are shown in white. The dimensions and panel numbers are indicated on the drawing for reference.

**TABLE:**
The table lists the dimensions and panel numbers for each plate. The dimensions are given in millimeters, and the panels are listed in sequence. The table is used to determine the number of bands required for each panel.

### TYPE "T" PANELS WITH BOLTS

**WITH 1-2 EMBEDS**

<table>
<thead>
<tr>
<th>Plate</th>
<th>Dimension</th>
<th>Dimension</th>
<th>CM</th>
<th>Panel</th>
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**NOTE:**
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Top panels 1000 mm and below need 0 horizontal bands.
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Top panels 1150 mm and below need 3 vertical bands.
Top panels 1200 mm and above need 3 vertical bands.

**CONSTRUCTION:**
The bands shall be fixed to the frame in the same order as the panels are listed. Each panel shall be denoted with a "1" if the panel number is one or a "2" if the panel number is two, and so on. All panels above 2000 mm shall be denoted with a "X." The number of bands per panel shall be as indicated. If a panel is not used, no band will be required. Any panel with no bands will be denoted with a "X." In the case of panels needed, they are needed. If panels are needed, there will be a "0" in the column.

**DIAGRAM:**
The drawing shows the panel arrangement and the position of the bands. The bands are shown in black, and the panels are shown in white. The dimensions and panel numbers are indicated on the drawing for reference.

**TABLE:**
The table lists the dimensions and panel numbers for each plate. The dimensions are given in millimeters, and the panels are listed in sequence. The table is used to determine the number of bands required for each panel.
TYPE "L" PANELS
MODIFIED FROM A PANELS
SHOWN FROM BACK FACE

<table>
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<tr>
<th>No.</th>
<th>Name</th>
<th>Description</th>
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<tbody>
<tr>
<td>2</td>
<td>LOOP CHOKES</td>
<td>CONNECTION PANELS</td>
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<tr>
<td>4</td>
<td>MOUNTING BRK</td>
<td>1/2&quot; - GAGE 80</td>
</tr>
<tr>
<td>6</td>
<td>MOUNTING BRK</td>
<td>1/2&quot; - GAGE 80</td>
</tr>
<tr>
<td>6</td>
<td>MOUNTING BRK</td>
<td>1/2&quot; - GAGE 80</td>
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</table>

NOTE: 1/2" BAR IS EQUIVALENT TO A BAR. THE FABRICATOR HAS THE OPTION TO USE ALTERNATIVE BAR CONFIGURATIONS AS LONG AS THE TOTAL VERTICAL AND BAR CONFIGURATIONS MUST BE APPROVED AND APPROVED BY DG, RECARO LASO.
DIMENSION "A"

LIFTING

HOLES

2" CLEAR
FOR NIPPLE BAR

TYPE "RX" PANELS
MODIFIED FROM X PANELS
SHOWN FROM BACK FACE

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<tr>
<th>No.</th>
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<th>Description</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Horizontal Bar</td>
<td>1 9/16&quot; DIA., 1/2&quot; THREAD</td>
</tr>
<tr>
<td>2</td>
<td>Horizontal Bar</td>
<td>1 9/16&quot; DIA., 1/2&quot; THREAD</td>
</tr>
<tr>
<td>3</td>
<td>Loop Endless</td>
<td>Connection Varies</td>
</tr>
</tbody>
</table>

NOTE: 1 9/16" BAR IS EQUIVALENT TO 8# BAR. THE DESIGNER HAS THE OPTION TO USE ALTERNATIVE BAR CONFIGURATIONS AS LONG AS THE CROSS SECTION AREA IS EQUIVALENT IN ACCORDANCE WITH THE SPECIFIED CLEARANCES AND VERTICAL CLEARANCES. PROPER ALT.

ALL DIMENSIONS IN MILLIMETERS

1" X 6"-10" X 9"-12" X 14"

1" X 6"-10" X 9"-12" X 14"
C.2 Wall B
1.0 INTRODUCTION

A. stitcher enclosing earth wall shall be driven into a suitable structure with a thickness of not less than 0.15 m for a height of not less than 0.5 m above the ground. The stitcher enclosing earth wall shall be constructed to resist the forces of inclined earth acting on it. The stitcher enclosing earth wall shall be constructed to resist the forces of inclined earth acting on it.

2.0 WALL COMPONENTS

A. The stitcher enclosing earth wall shall be constructed of concrete and shall be reinforced with steel bars. The concrete shall have a minimum cover of 25 mm for all reinforcement. The steel bars shall be placed in the concrete in such a manner as to provide the required strength and stability.

3.0 GENERAL REQUIREMENTS

A. The stitcher enclosing earth wall shall be constructed to resist the forces of inclined earth acting on it. The concrete shall have a minimum cover of 25 mm for all reinforcement. The steel bars shall be placed in the concrete in such a manner as to provide the required strength and stability.
6.0 ERECTION SEQUENCE

1. Lay out the area where the RIM BIOPLANK is to be placed should be graded level. At least 12 inches below the finished area and ensure that the area is free from debris and obstructions. Set the concrete forms and place the RIM BIOPLANKs in the forms.

2. All RIM BIOPLANKs should be mounted on the concrete forms in the same way that they were in the factory. The concrete forms should be filled with concrete and allowed to cure.

3. Once the concrete has cured, remove the RIM BIOPLANKs from the forms and place them on the location where they are to be used. The RIM BIOPLANKs should be placed on a level surface and allowed to cure for at least 24 hours.

4. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.

5. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.

6. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.

7. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.

8. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.

9. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.

10. Once the RIM BIOPLANKs have cured, they can be used in the erection of the building. The RIM BIOPLANKs should be placed in the areas where they are to be used and allowed to cure for at least 24 hours.
TYPE "T" PANELS WITH DOWELs
WITH 1-2 EMBEDs

<table>
<thead>
<tr>
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<th>Nom.</th>
<th>Description</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>HORIZONTAL</td>
<td>1/2&quot; BAR - GRATE 8</td>
</tr>
<tr>
<td>2</td>
<td>VERTICAL</td>
<td>1/2&quot; BAR - GRATE 8</td>
</tr>
<tr>
<td>3</td>
<td>CONNECTOR</td>
<td>1/2&quot; BAR - GRATE 8</td>
</tr>
</tbody>
</table>

ALL DIMENSIONS ARE IN MILLIMETERS

NOTE:
- The designer has the option to use alternative bend configurations as long as the bend is equivalent. If equivalent, bends at horizontal and vertical connections. Proposed alternative bend configurations must be reviewed and approved by all before use.

ALL DIMENSIONS ARE IN MILLIMETERS (IN.)

1" CLEAR
FOR REBAR MAX

LIFTING HEIGHTS
B 300 O.C.

COMPLIES WITH CSA O80.4-2006
TYPE "TREX" PANEL
SHOWN FROM BACK FACE

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>1</td>
<td>VERTICAL BAR - GRUB BS</td>
</tr>
<tr>
<td>2</td>
<td>HORIZONTAL BAR - GRUB BS</td>
</tr>
<tr>
<td>3</td>
<td>LOOP DOWELS - CONNECTION DOWEL</td>
</tr>
</tbody>
</table>

NOTES: 1. W/B BAR IS EQUIVALENT TO #6 BAR. THE PRECASTER HAS THE OPTION TO USE ALTERNATIVE REBAR CONFIGURATION AS LONG AS THE CROSS SECTION WHERE IS EQUIVALENT TO BOTH THE HORIZONTAL AND VERTICAL DIRECTIONS. NO PRECISE ALTERNATIVE REBAR CONFIGURATIONS MUST BE RENEWAL AND APPROVED BY E.I. BEFORE USE.

ALL DIMENSIONS IN MILLIMETERS ± 1/8
C.3 Wall C
TYPICAL FRACTURED RIB
ARCHITECTURAL PANEL DETAIL

SECTION A-A

SECTION B-B

NOTES:
1. ACTUAL FINISH TO BE SIMILAR TO WHAT IS SHOWN.
2. PANEL MATERIALITY, PANEL DETAILS, ETC. NOT SHOWN FOR CLARITY.
2.0 WALL COMPONENTS

2.1 WALL PLATE - Furring Strips that make up the wall plate shall be 2 feet x 8 feet.对接接 

2.2 OLD OR NEW - The old and new building should be as a whole.对接接 

2.3 ALL PLACING, DIVIDING, SPLITTING, OR SOLING SHOULD BE PERFORMED IN A MANNER THAT WHOSE REASONS BECAUSE OF THE ACCEPTANCE OF THE WORKING CONDITIONS.对接接 

2.4 GENERAL REQUIREMENTS

2.4.1 The design of the wall plate shall be determined by the general conditions of the wall.对接接 

2.4.2 The thickness of the wall shall be determined by the load of the wall.对接接 

2.4.3 The reinforcing elements shall be designed to have adequate corrosion resistance to the environment.对接接 

2.5 MOLDING FLANGE

2.5.1 The flange shall be designed to have adequate corrosion resistance to the environment.对接接 

2.6 SUPPORTING ELEMENTS

2.6.1 The supporting elements shall be designed to have adequate corrosion resistance to the environment.对接接 

2.7 MATERIALS

2.7.1 The materials shall be selected to have adequate corrosion resistance to the environment.对接接 

3.0 DESIGN REQUIREMENTS

3.1 GENERAL

3.1.1 The design shall be based on the requirements of the wall.对接接 

3.1.2 The design shall be based on the load of the wall.对接接 

3.1.3 The design shall be based on the requirements of the wall.对接接 

3.2 CONCRETE PLATE

3.2.1 The plate shall be designed to have adequate corrosion resistance to the environment.对接接 

3.2.2 The plate shall be designed to have adequate corrosion resistance to the environment.对接接 

3.3 MOLDING FLANGE

3.3.1 The flange shall be designed to have adequate corrosion resistance to the environment.对接接 

3.4 SUPPORTING ELEMENTS

3.4.1 The supporting elements shall be designed to have adequate corrosion resistance to the environment.对接接 

3.5 MATERIALS

3.5.1 The materials shall be selected to have adequate corrosion resistance to the environment.对接接 

4.0 FABRICATORS OF SAFETY

4.1.1 The fabricators of safety shall be determined by the requirements of the wall.对接接 

4.1.2 The fabricators of safety shall be determined by the requirements of the wall.对接接 

4.2.1 The fabricators of safety shall be determined by the requirements of the wall.对接接 

5.0 CEMENT OF PANELS

5.1.1 The panels shall be cast in place on a floor supported on a firm foundation.对接接 

5.2.1 The panels shall be cast in place.对接接 

5.3.1 The panels shall be cast in place.对接接 

6.0 SUPPORTING ELEMENTS

6.1.1 The supporting elements shall be designed to have adequate corrosion resistance to the environment.对接接 

6.2.1 The supporting elements shall be designed to have adequate corrosion resistance to the environment.对接接 

6.3.1 The supporting elements shall be designed to have adequate corrosion resistance to the environment.对接接 

7.0 DESIGN REQUIREMENTS

7.1 GENERAL

7.1.1 The design shall be based on the requirements of the wall.对接接 

7.1.2 The design shall be based on the load of the wall.对接接 

7.1.3 The design shall be based on the requirements of the wall.对接接 

7.2 CONCRETE PLATE

7.2.1 The plate shall be designed to have adequate corrosion resistance to the environment.对接接 

7.3 MOLDING FLANGE

7.3.1 The flange shall be designed to have adequate corrosion resistance to the environment.对接接 

7.4 SUPPORTING ELEMENTS

7.4.1 The supporting elements shall be designed to have adequate corrosion resistance to the environment.对接接 

7.5 MATERIALS

7.5.1 The materials shall be selected to have adequate corrosion resistance to the environment.对接接
Appendix D
Photos - MSE Walls Post Collapse
D.1 Photo Summary - Wall A

Figure D.1: Wall A Supporting Southbound Off Ramp (view looking to the North towards Kelowna).

Figure D.2: Tallest Portion of Wall A supporting the Intersection of Southbound Hwy 97 Off Ramp (on left) and Westside Road (on right). The base of Wall A contains four layers of D20 mesh in this area.
Figure D.3: Wall A Supporting North Side of Westside Road. Photo looking East Towards the Central Portion of Wall A.
D.2 Photo Summary - Wall B

Figure D.4: East Abutment (Wall C) on Left with Longer Span, Central Pier, and West Abutment (Wall B) on Right.

Figure D.5: Central Pier, Precast Concrete Box Stringers, Wall B and Coping, North End of West Abutment.
Figure D.6: Collapse extent under West Abutment at Wall B.

Figure D.7: Mesh Broken at Bends; Lightly Compacted Fill Spilled from behind Panels; Abutment Partially undermined; and, Heavily Compacted Fill Still Standing Vertically and Supporting Abutment.
Figure D.8: Depth of Lightly Compacted Fill Spilled from Behind Panels.

Figure D.9: Hook End of Mesh Reinforcement Retained with Panel Embeds and Locking Pins after Facing Panel Collapse and Mesh Failure at the 90° Bends.
D.3 Photo Summary - Wall C

Figure D.10: Wall C Shortly after Collapse of Wall B.

Figure D.11: Wall C Buttressed with Lock Blocks and Shoring in Place under the East Span
Figure D.12: Settlement of the East Abutment (right) Relative to the Wall C Coping (left), later Confirmed by Survey. Note the Broken Wires in the Background Exposed during Initial Hydrovac Works for Preliminary Examination.

Figure D.13: Downward Vertical Displacement of Reinforcing Mesh at South End of Wall C Abutment
Figure D.14: Wall C during Deconstruction. Note Panel Connection Aligns with Mud Slab, Reinforcing Displaced approximately 75mm Vertically and 25mm Horizontally. Break is at the 90° Bend.
Appendix E
Construction Timeline

The project timeline for the works was provided by the owner's field staff. The timeline supplied the start and finish dates for the various stages in the construction of the MSE walls and bridge structures.

The important events within that timeline are set out in Figure E.1. Relevant photos of the construction process are set out in Appendix I. Of note are the following events:

- The owner's field staff informed us that the SSL representative was on site 2010 November 5-6 to provide guidance during the first two days of the construction of Wall C but did not attend for Walls B and A. The Ledcor Project Superintendent was then designated as the onsite supervisor for the balance of the works. Refer Appendix F Review of Contract Special Provisions for MSE Wall supplier site presence requirements.

- Pier and Wall C excavations began at the end of 2010 October.

- Pier completed 2010 December 17 (Pier and Wall C nearing completion, shown in Figure I.15).

- The Wall C abutment spread footing was completed 2010 December 09 with east abutment completed 2011 January 24.

- The precast girders were placed on the east span during 2011 February 01-02 (Girders in place on Pier and Wall C shown in Figure I.16).

- Wall A was constructed between 2011 March 01 and 2011 August 22 (refer Figure I.1 to Figure I.5).

- Wall B excavations began 2011 June 03.

- The Wall B abutment spread footing was completed 2011 June 30 with the west abutment completed on 2011 August 02 (Wall B nearing completion shown in Figure I.9 and concrete pouring stage for west abutment spread footing and coping shown in Figure I.10).

- The precast girders were placed on the west span during 2011 July 18-19.

- Overlay concrete to final profile was complete on the Westside Rd Overpass on 2011 September 30.
- Overpass opened to traffic on 2011 October 30.
- MSE Wall B facing panels collapsed 2011 November 20 (refer Figure D.6 to Figure D.9).

The construction timeline finish dates indicate when the associated dead and live loads for a stage of construction were imposed. Wall B experienced the facing panel failure, but Wall C experienced higher loading from the bridge structure and for longer, as noted below:

- Wall C was completed approximately 6 months before Wall B.
- The abutment spread footing dead load was applied at Wall C approximately 8 months before Wall B.
- The dead load from the precast girders in the east span was in place on Wall C for about 5-1/2 months longer than the west span girder loads at Wall B.
- The dead load and live load from the overpass applied to the Wall C abutment is larger than that applied to Wall B as the span on the east side is 46% longer (25.57 m vs. 17.55 m).

The precast girders at Wall B were in place for a total of 4 months, the overlay for approximately 7 weeks and the presence of live load from traffic for approximately 3 weeks prior to the facing panel collapse.
## CONSTRUCTION EVENTS

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<tr>
<th>Event Description</th>
<th>Start Date</th>
<th>Finish Date</th>
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<tbody>
<tr>
<td>Initial Site Mobilization</td>
<td>6-Aug-10</td>
<td>15-Aug-10</td>
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<tr>
<td>Excavate Pier Foundation</td>
<td>27-Oct-10</td>
<td>29-Oct-10</td>
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<tr>
<td>Foundation Excavation</td>
<td>29-Oct-10</td>
<td>3-Nov-10</td>
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<tr>
<td>SSL Rep on site to train Ledcor (*)</td>
<td>5-Nov-10</td>
<td>6-Nov-10</td>
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<tr>
<td>SSL</td>
<td>3-Nov-10</td>
<td>1-Dec-10</td>
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<td>East Abutment Spread footing</td>
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<td>Precast Girders, east span</td>
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<td>Foundation Excavation</td>
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<td>Construction of Levelling Strip</td>
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<td>MSE Wall Installation</td>
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<td>West Abutment Spread footing</td>
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<td>Open Westside Rd Interchange</td>
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<td>MSE Wall B Collapse</td>
<td>20-Nov-11</td>
<td>30-Oct-11</td>
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</tbody>
</table>

### NOTES

(*) The SSL Representative was on site November 05 and 06 2010 to provide guidance to Ledcor/BC General during the first couple days of MSE Wall C construction. They [SSL] did not attend for Walls B and A; Ledcor were designated to supervise.** - Per owner's engineering field staff, email 2012 Jan 5

Begin application of dead load (DL) from the bridge superstructure to the MSE Walls

Application of traffic live load (LL) to the MSE Walls

MSE Wall B (West Abutment) collapse date

---

**FIGURE E.1: PROJECT TIMELINE**
Appendix F

The Contract section relevant to the design and construction of the MSE Walls is Schedule 3, Special Provisions, Section 6 - Structures, specifically Subsections 6.04 Bridge End Fill (this is also the granular backfill material used for the construction of the MSE walls) and 6.14 Mechanically Stabilized Earth (MSE) Wall Systems. For reference, Section 6 of the Contract is provided in full in Appendix G.

This appendix discusses:

- The governing design codes and standards set out in the contract and the design drawings;
- The specifications within the relevant ASTM standards for the reinforcing mesh;
- The provided design values;
- The corrosion protection requirements (galvanizing) especially relating to steel embrittlement;
- The specifications relevant to the backfill used in the MSE walls; and
- The construction oversight requirements.

Section 6.14 Mechanically Stabilized Earth (MSE) Wall Systems is reviewed first. The Design Criteria are tabled in Section 6.14.b and are addressed in the order they are tabled.

F.1 Design Criteria

F.1.1 Design Codes

The governing Design Codes are set out in Item 2 of the Design Criteria Table (p. 66 of 88), as follows:

2. Design Codes.

"CAN/CSA S6-06 in conjunction with the BC MoT Supplement to CHBDC S6-06, including the limit states approach. For items not covered by CAN/CSA-S6-06 or the BC MoT Supplement to CHBDC S6-06, or unless noted otherwise, AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002 ['AASHTO 2002'] including interim revisions shall be the governing code. In the case of any discrepancies between the specified
design codes and these MoT specifications and/or Drawings, these MoT specifications and/or Drawings shall govern."

MSE Walls are specifically addressed in BC MoT Supplement to CHBDC S6-06, Section 6.12 MSE Structures, which states:


AASHTO 2002 has two parts, Division I-Design and Division II-Construction. MSE Walls are addressed in Division I in Section 5.8 (MSE Wall Design) and in Division II in Section 7.6.4 (MSE Walls).

F.1.2 Relevant ASTM Standards - Soil Reinforcement

AASHTO 2002 Division I and II require conformance to four particular ASTM standards relevant to the soil reinforcement. These mandate the use of cold drawn smooth steel wire to ASTM A82, fabricated in accordance with ASTM A185 (welded wire reinforcement) and galvanized in accordance with ASTM A123 (hot dip galvanized coatings on steel products) and ASTM A641 (galvanized carbon steel wire). The SSL Drawings call out the same standards in Drawing RW-04 (Dwg RW-03 for Wall C), Section 5.2 Materials: Reinforcing Elements (omitting for ASTM A641).

The ASTM Standards noted above are material (smooth wire), fabrication (welding of wire into mesh, bending of the mesh) and process (hot dip galvanizing) standards. These are not design standards. However, they provide quality control bend tests for the wire material to determine conformance to the standard. The ASTM A82, Clause 5.2, Table 3 bend test for smooth wire greater than W7 (W11 and W20 are the SSL Drawing sizes) is to "Bend around a pin the diameter that is equal to twice the diameter of the specimen".

The supplied reinforcing mesh used deformed wire to ASTM A496, fabricated in accordance with ASTM A497 and galvanized in accordance with ASTM A123 (Refer Appendix J for mesh fabricator and galvanizer documents). This does not agree with the SSL Drawings and does not agree with AASHTO 2002. However, if deformed wire was to be used, at the minimum it should be used in accordance with the ASTM specifications covering the material, its fabrication and processing. The ASTM A496, Clause 8.2, Table 5 bend test for deformed wire greater than D6 (D11 and D20 are the SSL supplied sizes) is to "Bend around a pin the diameter that is equal to four times the diameter of the specimen".
Table F.1: SSL Specified (Smooth) and Supplied (Deformed) Material and ASTM Bend Test Diameters

<table>
<thead>
<tr>
<th>Wire</th>
<th>Material Relevance</th>
<th>Material Specification</th>
<th>Wire Diameter (mm)</th>
<th>Actual Bend Diameter (mm)</th>
<th>Bend Test Diameter* (mm)</th>
<th>Ratio Actual / Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>W11</td>
<td>Specified on SSL Drawings</td>
<td>ASTM A82 (plain)</td>
<td>9.50</td>
<td>22.0</td>
<td>19.0</td>
<td>1.16</td>
</tr>
<tr>
<td>W20</td>
<td></td>
<td>ASTM A82 (plain)</td>
<td>12.8</td>
<td>22.0</td>
<td>25.6</td>
<td>0.86</td>
</tr>
<tr>
<td>D11</td>
<td>Supplied</td>
<td>ASTM A496 (deformed)</td>
<td>9.51</td>
<td>22.0</td>
<td>38.0</td>
<td>0.58</td>
</tr>
<tr>
<td>D20</td>
<td></td>
<td>ASTM A496 (deformed)</td>
<td>12.8</td>
<td>22.0</td>
<td>51.2</td>
<td>0.43</td>
</tr>
</tbody>
</table>

* Bend test diameter is 2 times wire diameter for plain and 4 times wire diameter for deformed. The advent of ASTM 1060/1064 did not alter the above requirements.

Note that ASTM A1060 (galvanized steel welded wire reinforcement, plain and deformed) was in force at the time of the Contract signing and Clause 5.1, Note 2 says "Specifications A82/A82M, A185/A185M, A496/A496M and A 497/A497M have been replaced by Specification A 1064/A1064M". ASTM A1064 (steel wire and welded wire reinforcement, plain and deformed) and ASTM A1060 are therefore the governing specifications. However, no change was made to the bend test diameters of the previous specifications.

F.2 Provided Bridge Loads

The following loads are set out in Items 3 and 4 of the Design Criteria Table (p. 67 of 88):

3. Bridge Loads (un-factored)

4. Fence and Traffic Loads (un-factored)

Item 3 includes Design Loads from the Superstructure, from the Bridge Abutment, Horizontal Loads and Traffic Surcharge. The values were independently checked and are reasonably consistent with application of CAN/CSA S6-06 and the design drawings. Only the un-factored horizontal braking load was inconsistent at approximately 10% lower than the independent check.
Item 4 includes Design Horizontal Loads and Traffic Surcharge. The values were independently checked and are reasonably consistent with application of CAN/CSA S6-06 and the design drawings.

F.3 Allowable Bearing Pressures and Soil Parameters

Items 5 and 6 of the Design Criteria (pp. 68 of 88) were independently checked by B&T’s subcontractor, Naesgaard Geotechnical, Ltd (Naesgaard). Refer Appendix N, Naesgaard Geotechnical Report for a more detailed treatment.

5. Allowable Bearing Pressure and Internal/External Design Soil Parameters

6. Minimum Soil Reinforcement Length

Item 5 gives an "allowable bearing pressure under the MSE wall of 200 kPa when founded on compacted granular backfill". It also gives parameters for the backfill in the reinforced soil zones, the retained earth and the foundation soil.

The Ledcor Request for Information (RFI) 001 queried the 200 kPa value and the response clarified that the "200 kPa pressure provided in the geotechnical report was solely intended for the actual bridge footings and the MSE strip footings and not for the MSE reinforcement abutment fill zone footing. Using the soil parameters provided in the geotechnical report… the calculated allowable bearing pressure is 317 kPa."

The SSL MSE wall design was checked for external stability using two sets of foundation soil parameters, 1) parameters provided in the Golder 2009 report and, 2) simplified soil model and parameters contained in the Contract Special Provisions. The MSE wall design meets the AASHTO 2002 requirement for external stability for both sets of parameters.

Item 6 reiterates requirements of AASHTO 2002, Division I, Section 5.8.1.

F.4 Corrosion Protection Requirements

Item 7 of the Design Criteria addresses corrosion protection (pp. 69 of 88):

7. Corrosion Protection.

This section states "All soil reinforcement shall be galvanized in accordance with AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002, clause 5.8.6.1” and the service life is set at 100 years. The above clause specifies reinforcement galvanizing to ASTM A123 (strip type) or ASTM A641 (bar mat or grid type).
The current versions of ASTM A123 (2009), Section 1.2 states "This specification covers both unfabricated and fabricated products, for example… wire work fabricated from uncoated steel wire." ASTM A641 focuses on "galvanized carbon steel wire in coils for general use" with specific cases limited to "wire for nails and staples". As the fabrication of welded mesh starts with uncoated steel wire rather than a galvanized spooled wire product, ASTM A123 is the more relevant code. It is one of the two specifications referenced in AASHTO 2002 and the specification used by the galvanizer. Further discussion will be confined to the requirements of ASTM A123.

**F.4.1 Canadian Galvanizing Standard**

The Special Provisions, Section 6.14.d Quality Control requires test reports confirming galvanizing to CAN/CSA G164 for steel components. Section 6.14.g Supply and Installation, Metal Components, also requires "All steel components, including MSE reinforcing, shall be galvanized in accordance with CAN/CSA-G164. The Contractor shall provide 610 g/m² per side minimum mass of zinc coating to all metal fabrications."

A zinc deposition of 610 g/m² per side is the same value required by AASHTO 2002, so that is not in conflict. ASTM A-123/A-123M-02 is the standard noted by the reinforcing grid galvanizer; the Canadian standard is not referenced.

**F.4.2 Galvanizing and Steel Embrittlement**

It is of note that ASTM A123-09, Clause 5.2, Fabrication states that "the design and fabrication of the product to be galvanized are the responsibility of the designer and the fabricator. Practices A143… provide guidance for steel fabrication for optimum hot dip galvanizing and shall be complied with in both design and fabrication".

ASTM A143 (Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement) details factors in embrittlement (Clause 4), cold working and the need for thermal treatment to relieve severe cold work (Clause 6), and testing for embrittlement (Clause 9).

CAN/CSA G164-M92 (confirmed 2003) also includes requirements to test for embrittlement (Clause 6.5.1.1.1), "Strain age embrittlement tests on galvanized and ungalvanized material shall be made by clamping the samples in a vise and striking them a sharp blow with a 1 kg hammer. Fracture of the basis metal shall be considered a failure."
The connection between high degrees of cold work and embrittlement is clearly made. The material standard bend tests noted earlier in Section F.1.2 of this appendix (which are a test of material ductility and tolerance for cold work) are also clearly set out for both smooth and deformed wire, as are non-complex test methods and the responsibilities of the various parties to safeguard against embrittlement. Documentation was supplied stating that galvanizing was done in accordance with ASTM A123 (refer Appendix J Construction Documentation) but no evidence of testing for embrittlement to either ASTM A143 or CAN/CSA G164 was included. Refer to Appendix M Levelton Materials Report for further discussion of the effects of excessive cold working prior to galvanizing and embrittlement tests.

F.5 Bridge End Fill Specifications, Placement and Compaction

Special Provisions Section 6.04 Bridge End Fill (p. 60 of 88) states, "Processed blast rock has been placed in stockpile on site for use on this Project." This material was created during the already completed Phase I of the works begun in Fall 2009; construction of the MSE Walls and overpass structures is part of the current Phase II works awarded to Ledcor in Spring 2010.

Note that the material is referred to as Bridge End Fill; however, it is also the granular backfill material used for the construction of the MSE Walls and referred to as backfill in the SSL Drawings.

F.5.1 BC MoT Standard Specifications (SS)

Section 6.04 also states, "Bridge end fill shall be constructed as shown on the Drawings and in accordance with placement and compaction requirements of SS 201.40 Bridge End Fill. The Contractor shall perform all quality control and testing required to verify that the completed bridge end fill meets the specified requirements." This information is reiterated at 6.14.b (p. 66 of 88).

BC MoT 2009 Standard Specification (SS) for Highway Construction is the source for SS 201.40 which states "SS201.40 Bridge End Fill - Material for bridge end fill shall be in accordance with SS 202.04 and SS 202.05. Construction of bridge end fill shall be in accordance with SS 202.23."

- SS 202.04 addresses Aggregate Quality (Tests to Table 202-A and Properties to Table 202-B).
- SS 202.05 addresses Aggregate Gradation (Table 202-C).
SS 202.23 Bridge End Fill addresses lift thicknesses and density requirements. "The bridge end fill is to be constructed in successive horizontal layers not exceeding 150 mm in loose thickness (*). Each layer shall be compacted to a minimum 100% of the laboratory density obtained by the current ASTM test method D 698 (Standard Proctor). The determination of field density shall follow a method approved by the Quality Manager."

(" Note that the Golder Associates Technical Memorandum of August 21, 2009, Section 6.4 Structural and MSE Backfill notes that the "specified granular materials should be placed in loose horizontal layers not exceeding 300 mm thickness" and supports the decision taken to increase the maximum lift thickness to 250 mm.

The compaction machinery and testing is detailed in Appendix H MSE Wall Construction Process. 100% of Standard Proctor was the compaction requirement away from the facing panels. As discussed in the body of the report and the Appendix N Naesgaard Geotechnical Report, the issue relating to compaction is in the zone immediately behind the facing panels where SSL Dwg RW-05 (Dwg RW-04 for Wall C), "Erection Sequence and Details", Step I stipulates use of "light mechanical tampers" and Step K stipulates "compaction testing should not be taken within 910 [mm] of the [rear face of] the panels".

F.5.2 SSL Drawing Specifications

The SSL Drawings address the backfill material and placement on Dwg RW-04 and -05 (Dwgs RW-03, 04 for Wall C).

The relevant sections of Drawing RW-04 (Dwg RW-03 for Wall C) are:

- Section 2.0 Wall Components, states "The backfill material shall conform to project special provisions."

- Section 4.3 Backfill Material, addresses the backfill material angle of shearing resistance for internal stability of the wall, plasticity index, presence of deleterious components and angle of friction.

- Section 5.3 Select Granular Backfill, specifies gradation limits based on sieving in accordance with AASHTO T-27 and includes the requirement for 100% to pass a 102 mm (4") sieve. The electrochemical requirements for the soil are also tabled with the relevant AASHTO test criteria.

Drawing RW-05 (Dwgs RW-04 for Wall C) contains the erection sequence procedure and figures.
The concerns with the erection sequence detailed in Dwg RW-05 (Dwg RW-04 for Wall C) are discussed in the main body of this report and also in Appendix N Naesgaard Geotechnical Report.

F.5.3 AASHTO 2002 Requirements

AASHTO 2002, Division II-Construction, Clause 7.6.4.3 makes the following statements:

- Backfill to conform to Clause 7.3.6.3. Gradation and electrochemical properties are set out within this clause and are in agreement with those on the SSL Dwgs.
- "Placement and compaction shall be accomplished without distortion or displacement of the… soil reinforcement."
- "At each level of soil reinforcement, the backfill material shall be roughly leveled to an elevation of 0.1 foot [30 mm] above the level of the connection at the facing before placing the reinforcement."

Note that the Drawing RW-05 (Dwg RW-04 for Wall C) erection sequence is not in agreement with the last two bullet points.

The electrochemical properties for the backfill in the MSE walls are listed on the SSL Dwgs, in AASHTO 2002 Clause 7.6.4.3 and in the Contract Special Provisions and the same values are used in all three locations.

F.5.4 BC MoT Standard Specifications and AASHTO 2002 Gradation

The BC MoT SS gradation requirements for Bridge End Fill (used in the Special Provisions to also address the MSE wall backfill) in Table 202-C are more restrictive than those set out in AASHTO 2002 and the SSL Dwgs (refer Figure F.1).

AASHTO 2002 and the SSL Dwgs allow for a maximum sieve size of 4" (102 mm) and provide no restrictions between the 0.425 mm (No. 40) sieve size and the 4" maximum particle size. The BC MoT SS maximum particle size is 3" (75 mm) and upper and lower bounds are provided throughout the range of sieve sizes.

The Appendix J Construction Documentation shows that although a wider tolerance for gradation was allowed by AASHTO 2002 and the SSL Dwgs, the material was crushed and placed using the limits set by the BC MoT SS, with an allowance for a maximum 4" (102 mm) particles size. The EBA plots for the owner's engineering team and the Interior Testing Services (ITS) plots for Ledcor/BC General both include the MoT SS upper and lower bounds. The EBA plots note "Material plotted to
MoT 2009 SS, Table 202-C Bridge End Fill gradation requirements" with a "Description: 100 mm-" and the ITS plots cite "Specification MoTH - Bridge End Fill, Section 202-C". Although the latter cites "Sample Description: 3” minus", plots show sieving was done to 100 mm.

Figure F.1: Comparison of the relevant BC MoT SS and AASHTO 2002 Gradation Limits

The sample of gradation plots provided for the crushing and placing activities indicate that the material showed compliance (refer discussion in Appendix J Construction Documentation) with the criteria applied by the owner’s engineering team above.

Interaction between the large particle size and the size of the mesh openings at the abutments is discussed in the body of this report and Appendix H MSE Wall Construction Process.

F.6 MSE Wall Supplier Site Representative & Presence

Special Provisions Section 6.14.d Quality Control notes the following requirements for the monitoring program (p. 72-73 of 88):
"The work shall be inspected and supervised by the proprietor's (SSL) site representative who shall be:

- a Professional Engineer registered in the province of British Columbia with a minimum of five (5) years experience in the design and construction of the applicable proprietary structure; or

- who is a technician, with a minimum of five (5) years experience in the design and construction of the specific proprietary structure and who is designated in writing by and working under the direct supervision of a Professional Engineer registered in the province of British Columbia with a minimum of five (5) years experience in the design and construction of the applicable propriety structure."

Additionally, this section states:

"The proprietor's site representative must be on site to inspect and supervise the installation of the proprietary structure on the following basis:

- "… After foundation excavation is complete and prior to placement of any MSE wall material (including structural backfill and sub fills), and

- Continuously during the installation of the MSE walls."

The owner's field staff informed us that the SSL representative was on site 2010 November 05-06 to provide guidance during the first two days of the construction of Wall C but did not attend for Walls B and A (refer E-1 Construction Timeline). The Ledcor Project Superintendent was then designated as the onsite supervisor for the balance of the works.

It is not clear whether the criteria noted in the Special Provisions for the experience with this MSE Wall system by the wall supplier's site representative was met. Confirmation should be sought on that point as well as for the supervising British Columbia licensed P. Eng.
Appendix G
Contract Special Provisions - Section 6
Westbank First Nation and Ministry of Transportation and Infrastructure

Major Works

CONTRACT

PROJECT NO. 22402-0000

HIGHWAY NO. 97 WESTSIDE ROAD INTERCHANGE

Date of Contract:
Contractor:
Name:
Address:
City:
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HIGHWAY No. 97, WESTSIDE ROAD INTERCHANGE

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SECTION 6 – STRUCTURES

6.01 General

This Section of the Special Provisions shall apply to the following structures:

- Westside Road Underpass No. 7758
- Nancee Way Overpass No. 7759

Payment for work will be made at the prices bid for the Items appearing in the Schedule of Approximate Quantities and Unit Prices under the appropriate structure headings.

Any work called for which is not listed as an Item in the Schedule of Approximate Quantities and Unit Prices will not be paid for separately. The cost of such work shall be included in the prices bid for the Items in the Schedule of Approximate Quantities and Unit Prices.

The Construction Project Manager may require an acceptable declaration from the Contractor transferring ownership of materials to the WFN.

Progress payments will be made monthly, on the basis of progress estimates prepared by the Construction Project Manager. Unless more particularly specified in these Special Provisions, each progress estimate will assess the contract value of materials supplied and work done.

6.02 Structure Identification Numbers

The Construction Project Manager will make available the numeral forms for the Contractor’s use when the Contractor is required to imprint an identification number on the structure. The Contractor shall return the numeral forms to the Construction Project Manager in good clean condition upon completion of the Work.

6.03 Foundation Excavation and Backfill

Foundation excavation includes the material to be removed for the construction of the Westside Road Underpass pier.

a) Excavation

All materials shall be removed as necessary for the construction of foundations or other works. Foundation excavations shall not be larger than is reasonably necessary. Excavations and adjacent highways and other facilities shall be protected as necessary by barricades, and/or shoring.

Excavations shall be constructed in compliance with the applicable Workers Compensation Act, Occupational Health and Safety Regulations, BC.

b) Description of Material Types

“Solid rock” shall include all material of sufficient hardness to require breaking up by continuous drilling and blasting before removal, and boulders 1.5 cubic metres volume or more.

“Other materials” shall include all other solid materials which must be excavated.

No distinction shall be made between wet and dry excavation.
c) **Preparation of Foundations**

For excavations in material other than rock, care shall be taken to not disturb the bottom of the excavation. If the bottom of the excavation is disturbed, the Contractor shall remove and dispose of all disturbed material and shall replace it with material meeting the material, placement and compaction requirements of SS 201.40 "Bridge End Fill".

Where concrete is to be placed on rock, the rock surfaces shall be clean and free from any loose material.

Where, in the opinion of the Construction Project Manager, the bottom of an excavation is not competent, the Construction Project Manager may direct the Contractor to excavate deeper. The Construction Project Manager may direct replacement of the incompetent material with material meeting the requirements SS 201.40 "Bridge End Fill" or with a concrete fill or sub-footing.

Unless underwater concreting is approved by the Construction Project Manager, excavations for concrete structures shall be dewatered, if necessary, so that concrete is placed in the dry.

d) **Backfilling**

After the structures are sufficiently built, excavations shall be backfilled to the final ground contours as shown on the Drawings, or as directed by the Construction Project Manager.

e) **Measurement and Payment**

Payment for foundation excavation and backfill will be made at the Unit Price bid for "Foundation Excavation and Backfill" in Schedule 7. Payment shall include, excavation, shoring, barricades, backfilling to the final ground contours with suitable material, compaction of the material, compaction tests and quality control. No payment will be made for removal and replacement of material disturbed by the Contractor below the required depth of excavation. Payment shall also cover restoration of roadway gravel and pavement damaged by the Contractor's operations.

Where there is excess excavated material, payment shall cover loading, hauling and disposal of such material.

Excavated material which the Construction Project Manager deems unsuitable for backfill will be replaced with bridge end fill.

In the case of excavation for pier footings, no payment will be made for material removed outside vertical planes 600 mm outside the edges of the structure. Volumes shall be above the bottom of the excavations as shown on the Drawings or directed by the Construction Project Manager.

Progress payments will be made on the following basis:

<table>
<thead>
<tr>
<th></th>
<th>Solid Rock</th>
<th>Other Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation</td>
<td>80%</td>
<td>60%</td>
</tr>
<tr>
<td>Backfilling or Disposal</td>
<td>20%</td>
<td>40%</td>
</tr>
</tbody>
</table>
6.04 Bridge End Fill

Processed blast rock has been placed in stockpile on site for use on this Project. The stockpiled material is deemed to meet the specification for bridge end fill and is to be used for Bridge End Fill. There will be no charge to the Contractor for use of this stockpiled material.

Bridge end fill shall be constructed as shown on the Drawings and in accordance with the placement and compaction requirements of SS 201.40 Bridge End Fill. The Contractor shall perform all quality control and testing required to verify that the completed bridge end fill meets the specified requirements. Sampling and testing shall include, but may not necessarily be limited to, insitu density testing.

Drainage course material shall be installed as shown on the Drawings. The gradation of drainage course material shall be as follows:

<table>
<thead>
<tr>
<th>SIEVE SIZE mm</th>
<th>% PASSING BY MASS OF TOTAL SAMPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>0 - 100</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
</tr>
</tbody>
</table>

Payment for bridge end fill will be made at the Unit Price bid for “Bridge End Fill” in Schedule 7. Payment for bridge end fill shall include loading, hauling, placing, watering if required, and compaction of the material. Payment will cover all compaction tests. The volume will be measured in place to the neat lines shown on the Drawings. Payment will also cover the supply and placement of gravel drainage courses, geotextile and drain pipes as shown on the Drawings.

6.05 Formwork and Falsework

Formwork and falsework shall be in accordance with SS 211.

Formwork for parapets and bridge deck overlays will be considered as deck formwork. All other formwork required will be considered as substructure formwork.

Payment for formwork will be made in accordance with the SS 211.21 and at the applicable Unit Price bid for “Formwork” in Schedule 7. Payment shall also include quality control, submissions and any falsework and bracings as required. No payment will be made under this Item for formwork required as part of another Item.

6.06 Reinforcing Steel

Reinforcing steel shall be supplied and installed in accordance with SS 412 unless otherwise specified on the Drawings. Welding of reinforcing steel shall not be permitted. Reinforcing steel shall comply with CAN/CSA G30.18, 400R as specified on the Drawings.

Payment for reinforcing steel shall be in accordance with SS 412.91 and at the applicable Unit Price bid for “Reinforcing Steel” in Schedule 7. Reinforcing steel for parapets and bridge deck overlays will be considered as deck reinforcing steel. All other reinforcing steel required will be considered as substructure reinforcing steel. No payment will be made under this Item for reinforcing steel required as part of another Item, such as reinforcing for MSE wall copings.
6.07 Cast-in-Place Concrete

All concrete work shall be in accordance with SS 211, 413, and 933, unless otherwise modified by this clause. The Contractor shall be responsible for the design and quality control for all concrete used on this project.

All concrete materials and admixtures for concrete shall conform to the requirements of SS 211.04, unless otherwise specified in these Special Provisions.

The Contractor shall be responsible for and shall provide the Construction Project Manager with current certified results for all of the applicable tests as outlined in Table 211-D of the SS 211 “Required Aggregate Testing for Normal Density Coarse and Fine Aggregate (Per Individual Product and Aggregate Source)”.

Concrete mixes shall meet the requirements given in the following table:

<table>
<thead>
<tr>
<th>Classification</th>
<th>Minimum Compressive Strength at 28 days (MPa)</th>
<th>Nominal Maximum Size of Coarse Aggregate (mm)</th>
<th>Air Content (%)</th>
<th>Slump (mm)</th>
<th>Maximum W/C Ratio by Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Deck Concrete:</strong> Deck Overlay and Parapet (Standard Mix) (3)</td>
<td>35</td>
<td>28(^{(1)})</td>
<td>5 ± 1</td>
<td>50 ± 20(^{(2)})</td>
<td>0.38</td>
</tr>
<tr>
<td><strong>Substructure Concrete:</strong> Abutments, Pier, Working Floors (4)</td>
<td>30</td>
<td>28</td>
<td>5 ± 1</td>
<td>50 ± 20</td>
<td>0.45</td>
</tr>
<tr>
<td><strong>Keyways between Box Stringers:</strong></td>
<td>35</td>
<td>14</td>
<td>5 ± 1</td>
<td>20 ± 10</td>
<td>0.38</td>
</tr>
<tr>
<td><strong>MSE Wall Concrete:</strong> Precast Facing Panels, Copings, and Leveling Pads (4)</td>
<td>30</td>
<td>20</td>
<td>5 ± 1</td>
<td>50 ± 20</td>
<td>0.45</td>
</tr>
</tbody>
</table>

**Notes:**

\(^{(1)}\) The maximum proportion of aggregate passing the 5 mm screen shall be 37% of the total mass of aggregate.

\(^{(2)}\) Superplasticizer shall not be used.

\(^{(3)}\) No silica fume or flyash shall be used.

\(^{(4)}\) Application rate for flyash shall not exceed 15% by mass of Portland Cement.

The gradation of the 28 mm nominal size aggregate shall conform to Table 211-B of the SS 211 unless noted otherwise in this clause.
Concrete Surface Finishes shall meet the requirements given in the following table:

<table>
<thead>
<tr>
<th>Buried Surfaces</th>
<th>Class 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top and inner surfaces of parapets</td>
<td>Class 3</td>
</tr>
<tr>
<td>Outer surfaces of parapet on north side of Westside Road Underpass (adjacent to sidewalk)</td>
<td>Class 3</td>
</tr>
<tr>
<td>Outer surfaces of all other parapets and outer edges of deck</td>
<td>Class 2</td>
</tr>
<tr>
<td>Abutments, Pier and MSE Wall Copings</td>
<td>Class 2</td>
</tr>
<tr>
<td>MSE Wall Precast Facing Panels</td>
<td>Class 3 or Textured</td>
</tr>
<tr>
<td>Bearing seats</td>
<td>Troweled finish</td>
</tr>
<tr>
<td>Top of deck overlay</td>
<td>Transverse Tining</td>
</tr>
<tr>
<td>Sidewalk</td>
<td>Transverse Broom Finish</td>
</tr>
</tbody>
</table>

a) **Parapet Concrete**

Parapets shall not be extruded. Exposed corners shall be chamfered and surfaces finished as noted in these Special Provisions or on the Drawings.

b) **Payment**

Payment for concrete will be made in accordance with SS 211.21.02 at the applicable Unit Price bid for "Concrete" in Schedule 7. No payment will be made under this Item for concrete supplied as part of another Item, such as precast concrete, MSE wall copings or leveling pads. Payment for concrete for shear keys between box stringers and stringer dowel grouts will be included in the Lump Sum Price bid for erection of prestressed concrete stringers and will not be considered under this Item.

Payment for concrete will also include the preparation of all construction joints and control joints irrespective of their location, and the supply and installation of joint fillers, joint waterproofing membranes, and joint sealant as indicated on the Drawings.

For the purpose of establishing payment quantities, concrete used in bridge deck overlays and parapets will be considered as deck concrete. Payment for deck concrete will also include the supply and installation of the anchor bolt assembly for the signal pole in the parapet on the south side of the Westside Road Underpass. Concrete used in abutments and piers, including working floors, shall be considered as substructure concrete. Payment for substructure concrete will also include the supply and installation of the anchor bolt assemblies for the sign poles on the pier at the Westside Road Underpass.

### 6.08 Prestressed Concrete Stringers

a) **Supply and Manufacture**

Supply, manufacture and quality control of prestressed concrete stringers shall be in accordance with SS 415 - "Manufacture and Erection of Precast and Prestressed Concrete Members".
b) **Shipping and Erection**

Shipping and erection of prestressed concrete box stringers shall be in accordance with SS 415 - "Manufacture and Erection of Precast and Prestressed Concrete Members".

c) **Payment**

Payment for supply, manufacture and quality control of prestressed concrete stringers will be in accordance with SS 415.91, and at the Lump Sum Price bid for “Supply and Manufacture” in Schedule 7.

Payment for shipping and erection of prestressed concrete box stringers will be in accordance with SS 415.92, and at the Lump Sum Price bid for “Shipping and Erection” in Schedule 7. Payment shall also include the supply and placement of shear key concrete and supply and placement of stringer dowel grout.

### 6.09 Deck Joints

Deck joints shall be fabricated and installed by the Contractor as shown on the Drawings and in accordance with SS 422. Steel components shall be hot-dipped galvanized in accordance with CAN/CSA G164, after fabrication.

Each joint seal shall be supplied in a single length, without splices. Before the joint seal is installed, the joint seal and armouring shall be thoroughly cleaned and all moisture removed from the joint seal and armouring.

The seal shall be installed in accordance with the manufacturer's recommendations.

Compression seals shall be installed almost fully compressed and shall be 5 mm below the surface of the deck, unless shown otherwise on the Drawings.

Payment will be made at the Unit Price bid for “Deck Joint” in Schedule 7. Payment shall include quality control, all necessary material, shop drawings, the supply, fabrication, galvanizing of steel components, as required, and installation of the steelwork, anchors and elastomeric seals.

The lengths of joints shall be measured in place along the centrelines of the upper surfaces, and the upturns into the parapets.

### 6.10 Bearing Assemblies

The Contractor shall supply and install bearing assemblies in accordance with the Appendix “Supply, Fabrication and installation of Bearing Assemblies” and as shown on the Drawings.

Bearing assemblies shall include elastomeric bearing pads and galvanized dowels as shown on the drawings.

Payment will be made at the Lump Sum Price bid for “Bearing Assemblies” in Schedule 7. Payment shall include quality control, all necessary materials, submission of shop drawings, test results and certificates of compliance, supply, fabrication, and installation of bearing assemblies.
6.11 Bicycle Parapet Railing
Steel bicycle parapet railing and steel components of the bicycle parapet railing system shall be supplied, fabricated and installed as shown on the Drawings and in accordance with SS 422. All steelwork shall be galvanized after fabrication.

Railing shall be adjusted to produce uniform height and smooth alignment.

Payment will be made at the Unit Price bid for “Bicycle Parapet Railing” in Schedule 7. Payment shall include quality control, shop drawings and supply, fabrication, galvanizing and installation of anchor bolts and railing, including final alignment adjustments and the supply and placing of grout and shims if required.

The length of railing shall be taken as the out-to-out length of metalwork.

6.12 Standard Steel Bicycle Fence
Standard steel bicycle fence shall be supplied, fabricated and installed as shown on the Drawings and in accordance with SS 422. All steelwork shall be galvanized after fabrication.

Fences shall be adjusted to produce uniform height and smooth alignment.

Payment will be made at the Unit Price bid for “Standard Steel Bicycle Fence” in Schedule 7. Payment shall include quality control, shop drawings and supply, fabrication, galvanizing and installation of anchor bolts and railing, including final alignment adjustments and the supply and placing of grout and shims if required.

The length of fence shall be taken as the out-to-out length of metalwork.

6.13 Silane Surface Treatment
Silane surface treatment shall conform to SS 418. The tops and both faces of parapets shall be treated with a silane solution.

Payment will be made at the Unit Price bid for “Silane Surface Treatment” in Schedule 7, and in accordance with SS 418.08.

6.14 Mechanically Stabilized Earth (MSE) Wall Systems
a) General
Mechanically Stabilized Earth (MSE) wall systems shall be designed, supplied and installed under this Contract as proprietary structures.

MSE wall systems to be covered under this Special Provision shall include the following structures:

• At Westside Road Underpass, the MSE walls include MSE Wall A, MSE Wall B, and MSE Wall C as shown on the Drawings.
• At Nancee Way Overpass, the MSE walls include South MSE Wall and North MSE Wall as shown on the Drawings.

The Contractor shall ensure that the proprietary structures are designed, fabricated and constructed in accordance with the Contract Documents and that the structures meet all of the Project requirements.
Drawings have been included in the Contract Document Package and show the general arrangement of the MSE walls. As shown on the Drawings, the proprietary structures shall consist of:

- All MSE wall systems including facing panels, soil reinforcing and granular fill materials;
- All cast-in-place concrete copings and leveling pads;
- All pedestrian fences and attachments to the tops of the wall copings;
- All related drainage materials including piping, geotextiles and drain rock.

The Mechanically Stabilized Earth walls supplied for this Project must be one of the following systems:

<table>
<thead>
<tr>
<th>Wall System</th>
<th>Name of Proprietor and Address</th>
<th>Phone/Fax/Contact</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atlantic Wire Wall</td>
<td>Atlantic Industries Ltd. 4155 Crozier Road Armstrong, BC V0E 1B6</td>
<td>Ph: 1-250-546-9479 Fax: 1-250-546-9411</td>
</tr>
<tr>
<td>Hilfiker</td>
<td>Hilfiker Retaining Walls 1902 Hilfiker lane Eureka, CA 95503-5711</td>
<td>Ph: 1-800-762-8962 Fax: 1-707-443-2891</td>
</tr>
<tr>
<td>ISOGRID</td>
<td>Surespan Precast Suite 216 545 Clyde Avenue West Vancouver, B.C. V7T 1C5</td>
<td>Ph: 1-604-925-3377 Fax: 1-604-925-3394</td>
</tr>
<tr>
<td>Reinfoced Earth</td>
<td>Reinforced Earth Company Ltd. 101-2145 West Broadway Street Vancouver, B.C. V6K 4L3</td>
<td>Ph: 1-604-714-0766 Fax: 1-604-714-0767</td>
</tr>
<tr>
<td>SSL MSE Plus</td>
<td>SSL, Construction Products 4740-E Scotts Valley Drive Scotts Valley, CA 95066</td>
<td>Ph: 1-831-430-9300 Fax: 1-831-430-9340</td>
</tr>
<tr>
<td>VSL Retained Earth</td>
<td>Foster Geotechnical 1660 Hotel Circle North, Suite 304 San Diego, CA 92108</td>
<td>Ph: 1-619-688-2400 Fax: 1-619-688-2499</td>
</tr>
</tbody>
</table>

1 Accepted systems are those MSE walls with precast reinforced concrete facing panels and as per MoT Recognized Products List in the category titled “Bridge Abutments and Retaining Walls”.

2 Connection details with longitudinal wire size of W11 and W20 only.

b) Project Requirements
The MSE walls must be designed and constructed to satisfy the following:

- The MSE walls shall include precast reinforced concrete facing panels.
- Soil reinforcement shall be inextensible galvanized steel strips or galvanized steel wire grids. The reinforcement length shall be uniform throughout the entire height of the wall.
- The MSE walls must be supplied from one of the pre-qualified proprietors listed above.
• The structures shall conform to the general arrangement shown on the Drawings. All references to horizontal and vertical alignments shall be maintained as shown. All details, including minimum embedment depths, cast in place concrete copings, railings, reinforced backfill and drainage conduits etc., shall be incorporated in the final design.

• The cast in place concrete coping shall conform to the coping detail as shown on the Drawings. All coping concrete shall meet the requirements of SP 6.07.

• The pedestrian sidewalk fence shall be anchored along the top of the MSE wall copings.

• Backfill in the reinforced zones shall be constructed as shown on the drawings and in accordance with the material placement and compaction requirements of SS201.40 Bridge End Fill.

Design Criteria:

<table>
<thead>
<tr>
<th></th>
<th>Service Life</th>
<th>75 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.</td>
<td>Design Codes</td>
<td>CAN/CSA-S6-06 in conjunction with the BC MoT Supplement to CHBDC S6-06, including the limit states approach. For items not covered by CAN/CSA-S6-06 or the BC MoT Supplement to CHBDC S6-06, or unless noted otherwise, AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002 including interim revisions shall be the governing code. In the case of any discrepancies between the specified design codes and these MoT specifications and/or Drawings, these MoT specifications and/or Drawings shall govern.</td>
</tr>
</tbody>
</table>
3. **Bridge Loads (un-factored)**

MSE walls below bridge abutment footings shall be designed for the following loads:

**Design Loads from Bridge Superstructure:**
(at centreline of bearings, kN/m)

<table>
<thead>
<tr>
<th></th>
<th>Westside Rd. U/P</th>
<th>Nancee Way O/P</th>
</tr>
</thead>
<tbody>
<tr>
<td>East Abut</td>
<td></td>
<td></td>
</tr>
<tr>
<td>West Abut</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Each Abut</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL</td>
<td>195</td>
<td>138</td>
</tr>
<tr>
<td>LL</td>
<td>85</td>
<td>62</td>
</tr>
</tbody>
</table>

**Design Loads from Bridge Abutment:**

- Self Weight = 24 kN/m

**Design Horizontal Loads:**
(at top of stringers, normal to abutment)
- $H = 6.9$ kN/m (braking)

(at 1.05m above the top of coping)
- $P = 1.5$ kN/m (pedestrian fence)

**Traffic Surcharge:**
- 17 kPa

Also:
Granular fill loads (horizontal and vertical earth pressures) on C.I.P. Abutment Walls and seismic loads (Not included in above loads, to be included by the proprietor’s wall designer).

4. **Fence and Traffic Loads (un-factored)**

MSE retaining walls shall be designed for the following loads:

**Design Horizontal Loads:**
(at 1.05m above the top of coping)
- $P = 1.5$ kN/m (pedestrian fence)

**Traffic Surcharge:**
- 17 kPa

Also:
Seismic loads (Not included in above loads, to be included by the proprietor’s wall designer).
### Westside Road Underpass:

The allowable bearing pressure under the MSE wall is 200 kPa when founded on compacted granular backfill. The backfill used in the reinforced soil zone can be assumed to have a unit weight ($\gamma$) = 22 kN/m$^3$ and a friction angle ($\phi$) = 34°. The retained earth can be assumed to have a unit weight ($\gamma$) = 21 kN/m$^3$ and a friction angle ($\phi$) = 31°. The foundation soil will be layered silty clay and clayey silt which can be assumed to have a unit weight ($\gamma$) = 18 kN/m$^3$ and a friction angle ($\phi$) = 28°.

### Nancee Way Overpass:

The allowable bearing pressure under the MSE wall is 200 kPa when founded on compacted granular backfill. The backfill used in the reinforced soil zone can be assumed to have a unit weight ($\gamma$) = 22 kN/m$^3$ and a friction angle ($\phi$) = 34°. The retained earth can be assumed to have a unit weight ($\gamma$) = 21 kN/m$^3$ and a friction angle ($\phi$) = 31°. The foundation soil will be sand to silty sand which can be assumed to have a unit weight ($\gamma$) = 20 kN/m$^3$ and a friction angle ($\phi$) = 34°.

### Minimum Soil Reinforcement Length

The minimum soil reinforcement length provided for the MSE walls shall be the greater of:

- 2400 mm or
- 70% of the wall height, where:
  - For retaining walls the wall height shall be taken as the distance from the top of the leveling pad to the location where finished grade intersects the back of the wall face.
  - For walls below abutment footings the wall height shall be taken as the distance from the top of the leveling pad to the bridge road surface. The minimum soil reinforcement length shall apply below the abutment footing and for a distance on either side of the footing determined by assuming that the footing load disperses with depth at a slope of 2V:1H.

Reinforcement length shall be increased as required for surcharges and other external loads.
### 7. Corrosion and Protection

All soil reinforcement shall be galvanized in accordance with AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002, clause 5.8.6.1. Corrosion rate shall be computed for each exposed surface as follows:

Galvanization loss is equal to:
- 15 micrometres/year for first 2 years
- 4 micrometres/year for subsequent years

Carbon steel loss is equal to:
- 12 micrometres/year after zinc depletion

Service Life = 100 Years

### 8. Aesthetics

The wall coping shall be cast in place concrete finished to a Class 2 finish in accordance with the SS 211. The coping shall be formed to suit the alignment of the top of the MSE wall and finished to the elevations shown on the Drawings.

The wall system shall include precast concrete facing panels which shall be supplied with the following finishes:

MSE Wall A - Class 3 finish in accordance with the SS 211 or alternatively, a suitable textured concrete surface may be used with approval from the Construction Project Manager.

All other MSE Walls - Suitable textured concrete surface consistent with the MSE wall finish at the nearby Campbell Road Overpass and meeting with approval from the Construction Project Manager.

*Note: The Contractor shall be responsible for confirmation of internal and external wall stability. The Construction Project Manager will be responsible for determining the global stability of the wall system. Therefore, as part of its design review process, the Construction Project Manager may impose specific limits on some parameters such as; footing location, size and shape, pressure distribution on the footing and backfill, stress/strain/deformation parameters and drainage assumptions.*

c) Proprietary Structure Design Report

The Contractor shall prepare and submit a Proprietary Structure Design Report consisting of all information necessary for the project specific design, fabrication, transportation, quality control and installation of the proprietary structures, including but not limited to the following:

- Transmittal / covering letter;
- Complete set of design calculations;
- Design methodology, design assumptions, applicable design codes and standards;
- Geotechnical constraints;
- Force diagrams showing the magnitude, location and direction of all forces for each load case;
• Diagrams showing all foundation loads including load inclination angle, effective width of footing and bearing pressure for each load case;
• Service life calculation in accordance with the design criteria for corrosion rates;
• Wall drainage details;
• Complete material and construction specifications;
• Fabrication and installation drawings (as specified below);
• Fabrication location and schedule;
• Delivery details and schedule;
• Detailed backfill procedures including a determination of allowable deflections and procedures for monitoring deflections during construction;
• Details for temporary construction bracings if required;
• Concrete quality control procedures; and
• Assurance of Professional Design and Commitment to Field Reviews as specified herein.

Fabrication and installation drawings shall conform to the following:

(i) Format:
• Full size reproducible drawings, approximately 560 mm by 865 mm (one set).
• Lettering for notes and dimensions to be at least 2.5 mm, for headings 4 mm.
• Drawings to be legible when half-sized.

(ii) Contents:
• Plan views, showing information required to lay out the structure.
• Elevation views, with elevations and dimensions required to construct the work.
• Details and typical sections of all proprietary and associated components.
• Notes listing design codes, design criteria, unfactored applied loads, and material and construction specifications.
• Soil reinforcement types and lengths.
• Limits of excavations and backfill required by the wall designer.
• Component layout, coping, movement joints, typical details.
• Equipment to be used and any temporary support and bracings if required for installation including details of any excavation protection works.
• Identification of the materials and other work required under this Item, identifying, where applicable;
  o materials and work which will be provided by the proprietary structure supplier and are included in their price quotation to the Contractor, and
  o materials and other work which will not be provided by the proprietary structure supplier, and which the Contractor will have to supply and install.
The Contractor shall submit four (4) complete copies of the Proprietary Structure Design Report to the Construction Project Manager a minimum of six (6) weeks prior to commencement of the wall component fabrication work. The Proprietary Structure Design Report shall be sealed by the proprietor's design engineer, who shall be a Professional Engineer, registered in the province of British Columbia, with experience in the design and construction of similar structures.

The Construction Project Manager will review the submissions for general compliance with the Contract requirements and complete an assessment of the global stability of the proposed wall system.

If modifications are required, the Construction Project Manager will return one marked up copy of the Proprietary Structure Design Report to the Contractor. The Contractor shall make the appropriate revisions and resubmit four (4) complete copies of the revised Proprietary Structure Design Report to the Construction Project Manager for further review.

If no exceptions are taken, the Construction Project Manager will return one set of the reviewed Proprietary Structure Design Report to the Contractor. The Contractor shall forward an additional two (2) copies of the reviewed report to the Construction Project Manager.

The Contractor shall ensure that submissions are complete and in compliance with the requirements of the Contract. Incomplete or non-compliant Proprietary Structure Design Report submissions may, at the sole discretion of the Construction Project Manager, be returned to the Contractor for resubmission without review. Review of resubmissions by the Construction Project Manager may require an additional six (6) weeks to complete. Any delays to the Contractor's work schedule caused by review of resubmissions shall be the sole responsibility of the Contractor to remedy.

Any additional costs to the Construction Project Manager for additional review of resubmissions will be back charged to the Contractor. Such costs will be recovered by the Construction Project Manager via deductions to payments on the progress estimate. The Construction Project Manager will provide review of the original submission plus one resubmission at no cost to the Contractor.

Any work done or materials ordered prior to the completion of the review process shall be at the Contractor's risk. Installation of the MSE walls will not be allowed to proceed until the Construction Project Manager has completed its review of the Proprietary Structure Design Report and has received a Letter of Assurance for the design signed and sealed by the proprietor's engineer.

The Construction Project Manager's review of the Proprietary Structure Design Report will not relieve the Contractor from its obligation to perform the work in accordance with the Contract.
d) **Quality Control**

The Contractor shall implement a quality control program to the following requirements:

<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>REQUIREMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast-in-placeConcrete</td>
<td>The Contractor shall provide concrete mix designs and strength test results to the Construction Project Manager in accordance with SP 6.07.</td>
</tr>
<tr>
<td>Precast Concrete</td>
<td>The Contractor shall provide concrete mix designs, aggregate test results and strength test results to the Construction Project Manager and perform Quality Control as required in SS 415.</td>
</tr>
</tbody>
</table>
| Steel                    | The Contractor shall provide copies of the following to the Construction Project Manager:  
                            - mill certificates giving chemical and physical properties;  
                            - test reports confirming galvanizing to CAN/CSA G164                                          |
| Reinforced Backfills      | The Contractor shall provide tests results to the Construction Project Manager to verify that the reinforced backfills meet all placement and compaction criteria as specified in the Contract Documents and are in compliance with the requirements of the proprietary structure designer. Tests on reinforced backfill materials shall include but may not necessarily be limited to in-situ density tests. |

The completed structure shall meet the following tolerances:

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Dimensions</td>
<td>±25 mm</td>
</tr>
<tr>
<td>Elevations</td>
<td>±25 mm</td>
</tr>
<tr>
<td>Variations from plumb</td>
<td>1:100 but not more that 30 mm</td>
</tr>
<tr>
<td>Line on coping</td>
<td>such that no abrupt deviations are visible</td>
</tr>
</tbody>
</table>

A minimum of two (2) weeks prior to installation, the Contractor shall submit a detailed program for monitoring construction for both erection safety and structural integrity of the completed installation. The program must be approved and endorsed by the proprietary structure supplier and the MSE wall designer, who must indicate that the proprietor’s site representative will comply with the procedure.

The monitoring program shall include but may not be limited to:
- Stages at which monitoring is to be performed;
- Method, frequency and location of measurement and testing;
- Allowable deflections or movement;
- Procedures to be followed if allowable deflection or tolerances are exceeded and, if procedures include a stoppage of work, details of corrective action to be taken as a prerequisite to resuming work;
- Availability of onsite documentation for review and reference including: Letters of Assurance, WCB Inspection Reports, Site Erection Procedures and Quality Control Reports for off-site precast concrete or fabrication shops;
- Identification and relevant experience of the proprietor’s designated site representative(s); and
• A detailed schedule updated at the start of each work week, indicating when inspections will be performed and when the designated site representative(s) will be on site.

The work shall be inspected and supervised by the proprietor’s site representative who shall be a Professional Engineer registered in the province of British Columbia with a minimum of five (5) years experience in the design and construction of the applicable proprietary structure or, who is a technician, with a minimum of five (5) years experience in the design and construction of the specific proprietary structure and who is designated in writing by and working under the direct supervision of a Professional Engineer registered in the province of British Columbia with a minimum of five (5) years experience in the design and construction of the applicable proprietary structure.

The qualifications of the proprietor’s site representative(s) shall be subject to review by the Construction Project Manager.

The proprietor’s site representative must be on site to inspect and supervise the installation of the proprietary structure on the following basis:

• As required by the Professional Engineer who has sealed the Proprietary Structure Design Report, and;
• After foundation excavation is complete and prior to placement of any MSE wall material (including structural backfill and sub fills), and;
• Continuously during the installation of the MSE walls.

The proprietor’s site representative shall record all changes, additions and deletions to the accepted Fabrication and Installation Drawings to reflect the "as constructed" installation. The Contractor’s proprietor shall modify the accepted Fabrication and Installation Drawings, using the same format as the accepted submission, and noting the revision as the “As-Built Installation”.

e) Letter of Assurance and Record Drawings

Prior to commencing installation of the proprietary structure the Contractor shall provide to the Construction Project Manager an Assurance of Professional Design and Commitment to Field Reviews form as referenced in Technical Circular T06/09. The form shall be submitted with the Proprietary Structure Design Report and be signed and sealed by the proprietor’s Professional Engineer responsible for design of the proprietary structure.

After Construction, the Contractor shall provide a complete set of structure Record Drawings as well an Assurance of Professional Design – Post Construction, also as referenced in Technical Circular T06-09, completed by the proprietor stating that the proprietor has reviewed the MSE wall backfill materials, in-situ test results and has monitored the installation of the structural components and certifies that the in-situ structure is in accordance with all installation requirements. The Record Drawings and Assurance of Professional Design – Post Construction shall be sealed by the Professional Engineer who performed or was responsible for supervising the site inspections. The forms are available at:

f) Quality Assurance

The Construction Project Manager will implement a quality assurance program, by auditing the Contractor’s quality control program and by inspection and testing at its discretion. Quality assurance by the Construction Project Manager shall not relieve the Contractor from its obligation to perform the Work in accordance with the Contract.

The Construction Project Manager may reject any component of the work which, in its opinion, does not comply with the requirements of the Contract. The Contractor shall bear the cost of re-inspection of any rejected component of the Work.

The Contractor shall notify the Construction Project Manager at least fourteen (14) days prior to fabrication or installation of each component of the Work. The Contractor shall allow the Construction Project Manager access to all components the Work and shall supply such information and assistance as is required. The Contractor shall provide samples of any materials as requested by the Construction Project Manager.

g) Supply and Installation

The Contractor shall perform the Work in accordance with the Construction Project Manager reviewed Proprietary Structure Design Report, the Contractor’s quality control and construction monitoring program and all other Contract requirements.

- Cast-in-Place Concrete Components:
  All cast-in-place concrete, formwork and reinforcing steel supplied under this Item shall meet the requirements of SP 6.07, SP 6.05 and SP 6.06 respectively.

- Precast Concrete Components:
  SS 415 shall apply to supply and installation of precast concrete components, as modified by this clause.
  Concrete for precast components shall be supplied in accordance with SP 6.07.

- Metal Components:
  SS 422 shall apply to metal components, as modified by this clause.
  All steel components, including MSE reinforcing, shall be galvanized in accordance with CAN/CSA-G164. The Contractor shall provide 610 g/m$^2$ per side minimum mass of zinc coating to all metal fabrications.

- Backfill:
  Backfill is defined as the material within the reinforced zone that extends from the back face of the MSE wall precast panels to a vertical plane intersecting the ends of the longitudinal MSE wall soil reinforcement between the underside of the MSE leveling pad and to the level where finished grade meets the backside of the wall coping, except in the immediate vicinity of the bridge abutments, where the top boundary is the underside of abutment footing.
Backfill material shall meet the requirements for bridge end fill material as specified in SS201.40 and also the requirements of the Proprietary Structure Design Report. When steel is used in contact with water or soil, the following electrochemical requirements shall also be met:

- pH of 5 to 10;
- Resistivity not less than 3,000 ohm centimeters;
- Chlorides not greater than 100 PPM; and
- Sulfates not greater than 200 PPM.

Backfill shall be placed concurrently with roadway embankment or adjacent fill material such that the vertical difference between the two does not exceed 300mm.

Processed blast rock has been placed in stockpile on Site for use on this Project. The stockpiled material is deemed to meet the specification for Bridge End Fill and may be used, at the Contractor’s discretion, for backfill of MSE Structures. There will be no charge to the Contractor for use of this stockpiled material, however the Contractor’s use of this material for this purpose will be considered confirmation of the Contractor’s acceptance of this material for this purpose.

- Drainage Pipe:
  The Contractor shall supply and install all perforated PVC drainage pipes, drain rock and geotextiles as shown on the Drawings.

- Pedestrian Fence:
  Pedestrian fence shall be supplied fabricated and installed as shown on the Drawings and in accordance with SS 741 and Figure SP741-07.01 – Welded Fence. Fence anchorage shall be by epoxy grouting in a drilled hole.
  The pedestrian fence shall be adjusted to produce uniform height and smooth alignment.

h) Payment

Payment for the Mechanically Stabilized Earth Walls will be made at the Lump Sum Price bids as follows:

- The Lump Sum Price bid for MSE walls at Westside Road Underpass shall include MSE Wall A, MSE Wall B and MSE Wall C.
- The Lump Sum Price bid for MSE walls at Nancee Way Overpass shall include South MSE Wall and North MSE Wall.

Payment shall be for all works associated with the design, supply, installation, quality control, record drawings for and certification of the Mechanically Stabilized Earth Walls within the payment envelopes as shown on the Drawings. Payment shall include:

- Submission of an acceptable Proprietary Structure Design Report including complete Fabrication and Installation Drawings;
- Design, supply and installation of cast-in-place concrete leveling pad for the first row of pre-cast panels;
- Design, supply and installation of all components of the MSE Walls including, but not necessarily limited to, the precast concrete face panels, galvanized steel soil reinforcement, connection pins and filter fabric;
• Design, loading, hauling, placement, compaction, watering and insitu testing of all structural backfill materials;

• Design, supply and installation of all reinforced, cast-in-place concrete copings at the top of the uppermost row of precast MSE Wall panels;

• Supply and installation of all pedestrian fences;

• Design, supply and installation of all required drainage works behind the MSE Walls and extending to the outlets;

• Design and implementation of a quality control program for the design fabrication and installation of the MSE Walls;

• Inspection and field reviews by proprietor’s design engineer of record for the structure; and

• Submission of structure record drawings and Letters of Assurance for the completed works.

Payment for foundation excavation and preparation for MSE wall construction including stripping, highway demolition, excavation and structural sub-fill, as well as payment for electrical components and installations, will not be covered under this Item. Payment for these Items will be made under the appropriate Items as set out in Schedule 7. Progress payments will be made on the following basis:

10% for accepted Proprietary Structure Design Report
30% for delivery of wall components to site
50% for installation components and backfill
10% for accepted Record Drawings and Letter of Certification
Appendix H

MSE Wall Construction Process

We have received a description from the Owner's field staff of the method used to construct the MSE walls. This information is in addition to photographic records that have been supplied covering the construction period of the three MSE walls. This information is set out in detail later in Appendix H with photos in Appendix I.

The construction procedure was as per SSL Dwg RW-05 (reference made to SSL Dwgs numbers applies to Walls A and B; decrement by one for the Wall C drawing number).

With respect to the forensic investigation, the important elements within the wall construction process are as follows:

i. Backfill placed through the reinforcing mesh after its connection to the precast facing panels in order to fill the void that existed underneath per the SSL Erection Sequence and Details;

ii. Relatively large and sometimes oblong particles size (although within specification) relative to the mesh openings (especially in conjunction with item one);

iii. Mesh installed at elevations below the level of the panel embeds; and,

iv. Loading from the subsequent light compaction, further lifts of backfill and the overpass.

H.1 Backfill Placement behind the Facing Panels

Lifts of backfill material a maximum of 250 mm thick were placed and subjected to heavy compaction machinery subsequent to each lift (the BC MoT Standard Specifications 2009 (SS) 202.23 requirement for 150 mm maximum lift heights had been modified to 250 mm with the approval of the owners engineer team). However, the backfill was not placed directly against the rear face of the precast panels. The SSL Erection Sequence Drawing RW-05 allows the fill to be "sloped downward toward the panels". Numerous photos document this as the case on site. The sloping downward effect generally existed over an approximately 1 meter wide strip immediately behind the panels.
When the main body of the fill was brought to the level of the next layer of reinforcing mesh, the mesh was then placed and connected to the embedded loops in the rear surface of the precast panels, bridging over this area of missing fill. During the next lifts, backfill material was poured through the mesh to fill the gap under and between mesh panels.

H.2 Backfill Size and Shape versus Mesh Openings

An interaction between the large and sometimes oblong particles with the relatively small opening sizes in the reinforcing mesh, in combination with the backfill placement method noted above that required the backfill material to pass through the mesh openings to fill the void below, created an opportunity for the backfill material to hang up on the mesh.

The SSL Drawings and AASHTO 2002 backfill gradation specification is for 100% passing a 102 mm sieve. The larger particles were sometimes observed to be elongated, exceeding the 102 mm in one dimension. These particles, being 4" or less in two dimensions, would have passed a dynamic sieving test and met the requirements of the gradation specification.
The D20 steel reinforcing mesh is a rectangular grid. It was constructed with a nominal arrangement of 203 mm centres between longitudinal wires. The centre to centre distance between the D11 cross wires varied. On Wall B, the four layers of mesh under the abutment (the most highly loaded area), used 152 mm cross-wire centres, giving a minimum opening dimension of 139 mm between cross wires. In the similar location on Wall C, 305 mm cross wire centres were used giving a minimum opening dimension of 194 mm between longitudinal wires.

In practice, the sizes of the mesh openings appeared to be sufficiently small relative to the maximum particle sizes to allow the material to interlock and hang up on the mesh. This condition was observed on site during the deconstruction of Wall C (East abutment). This interlock would also allow for voids to be maintained beneath the mesh.
H.3 Reinforcing Mesh Elevation with Respect to Panel Embeds

AASHTO 2002, Section 7.6.4.3. Construction (of MSE Walls) states "At each level of soil reinforcement, the backfill material shall be roughly leveled to an elevation approximately 0.1 foot (30 mm) above the level of connection at the facing before placing the soil reinforcement."

Based on the construction photographic evidence, mismatches in elevation between the reinforcing mesh and the facing panel loop embeds occurred. The top of the layer of backfill supporting the reinforcing mesh was finished low with respect to the level of the facing panel embedded loops, resulting in the mesh arcing upwards up to 100-150 mm in places to make the connection to the precast panel.

This conclusion is also supported by the survey results (Appendix K Survey Results). During deconstruction of Walls B and C, a vertical mismatch was seen on site between the mesh position in the backfill and the panel embed level. This mismatch frequently exceeded the relative settlement calculated between the backfill soil mass.
and the facing panels. The difference between the observed and calculated mismatch is the original difference in elevation between the reinforcing placement level and the panel embed level.

Note that further vertical displacement of an element that is already in a draped position generates greater tensions at an end connection than the same displacement applied to an element starting in a level or above level position. Placement of the mesh in line with the AASHTO clause would have resulted in lower tension in the reinforcing mesh when backfill settlement occurred compared to mesh that was already in a draped position.

H.4 Compaction and Loading Effects behind the Facing Panels

SSL Drawing RW-05 mandated that only light compaction equipment be used and no compaction testing be done in the 610 to 910 mm zone immediately behind the facing panels. Reviewing the construction process, the light compaction would have been applied to a layering of:

• Bottom layer: a 250 mm depth, or possibly deeper (as much as 760 mm at the panel face, the typical distance between mesh layers) of un-compacted backfill extending approximately 1m behind the wall facing panels;
• Middle layer: soil reinforcing mesh, possibly with voids under it due to the hanging-up effect and/or with an elevation mismatch to the panel embeds; and
• Top layer: further backfill material covering the mesh.

Note that the end of the mesh is pinned to the rear of the precast panels and is restricted from displacement downward. The reinforcing mesh in the zone behind the panels is insufficiently supported, as no compaction occurred in the zone behind the panels prior to the placement and connection of the mesh. Thus the mesh would be forced to drape (further) with subsequent compaction, applying tension to the connection. The backfill layer on top of the mesh is also capable of hanging up. This would prevent backfill material above the mesh from moving through it to fill voids below. The effect of the light compaction applied to the layering above was thus two-fold. The first was that the layering would act to maintain inadequate compaction in the zone behind the panels, as noted here. The second was adverse loading to the inadequately supported, embrittled reinforcing mesh at the connection to the wall panels, further discussed in the next section.
Inadequate compaction in this zone allowed for settlement upon further loading. Further loading came from the subsequent lifts of backfill, the weight from the abutment spread footings, and the overpass dead and live loads. The extent of material spilled from behind the facing panels supports the construction method leaving a considerable quantity of loose material in this zone.

H.5 Effect on Reinforcing Mesh Connection at the Panels

The reinforcing connection to the panel uses steel loops embedded (embeds) in the precast panels, a 90° bend in the reinforcing mesh, a bearing wire on the short leg of the reinforcing mesh and a 16 mm pin.

Figure H.4: Mesh Connections at Panels, Broken and Unbroken
To make the connection between the reinforcing mesh and the precast panel, the 90° bend in the mesh is first placed between the two embeds in the rear surface of the precast facing panel. The cross wire on the short, upward pointing leg of the reinforcing mesh is positioned above the embeds to restrain the mesh from downward movement and a 16 mm pin is passed through the embeds locking the mesh to the panel.

To prevent an opening effect on the 90° bend, the backfill material beneath the mesh needs to be well compacted and at a level suitable to offer support to the horizontal portion of the mesh. The addition of the next lift of backfill material on to and around the mesh connection likely serves to lock the short upward leg in a fixed position. Later downward movement of the soil mass (by further compaction or loading induced settlement) or outward movement of the panels (due to failure of adjacent reinforcing mesh) can open the 90° bend. Ductile steel would be relatively tolerant of this, but an embrittled bend lacks the ability to undergo further deformation without fracture.
H.6 MSE Wall Construction Process - Details

NOTES:

• OE is used for Owner's Engineers.
• SS is used for (BC MoT) Standard Specifications, 2009
• Reference to SSL Dwgs numbers is for Walls A and B; for the Wall C drawing number, decrement by one.

Table H.1: MSE Wall Construction Task

<table>
<thead>
<tr>
<th>ITEM</th>
<th>MSE WALL CONSTRUCTION TASK</th>
<th>PERFORMED BY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Manufacture of backfill (crushing process).</td>
<td>BC General.</td>
</tr>
<tr>
<td>2</td>
<td>Backfill handling from stockpile to current work site.</td>
<td>BC General.</td>
</tr>
<tr>
<td>3</td>
<td>Construction of concrete levelling strips.</td>
<td>Ledcor.</td>
</tr>
<tr>
<td>4</td>
<td>Excavator operation to place backfill.</td>
<td>BC General.</td>
</tr>
<tr>
<td>5</td>
<td>Heavy compaction operator.</td>
<td>BC General.</td>
</tr>
<tr>
<td>6</td>
<td>Light mechanical tamper operators.</td>
<td>BC General.</td>
</tr>
<tr>
<td>7</td>
<td>Mesh layout and pin connection to walls.</td>
<td>BC General.</td>
</tr>
<tr>
<td>8</td>
<td>Lifting precast panels into place.</td>
<td>BC General.</td>
</tr>
<tr>
<td>11</td>
<td>Bearing installation at spread footing.</td>
<td>Ledcor.</td>
</tr>
<tr>
<td>12</td>
<td>Lifting girders into place.</td>
<td>Ledcor.</td>
</tr>
<tr>
<td>13</td>
<td>Application of overlay onto girders to make up final profile.</td>
<td>Ledcor.</td>
</tr>
<tr>
<td>14</td>
<td>Small works (bracing precast panels, installation of filter fabric, etc).</td>
<td>BC General.</td>
</tr>
</tbody>
</table>

Table H.2: Backfill – Lift Thickness & Handling

<table>
<thead>
<tr>
<th>ITEM</th>
<th>BACKFILL - LIFT THICKNESS &amp; HANDLING</th>
<th>OE RESPONSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Material from Phase I was supplied in a stockpile location and per contract was processed blast rock meeting BC MoT SS 202.05, Table 202-C Aggregate Gradations for Bridge End Fill (for use as MSE wall backfill).</td>
<td>Correct.</td>
</tr>
<tr>
<td>2</td>
<td>Backfill handling method from stockpile to MSE wall site.</td>
<td>BC General loaded into Cat Haul trucks.</td>
</tr>
<tr>
<td>3</td>
<td>Noted backfill handling issues.</td>
<td>Excessive segregation from handling.</td>
</tr>
<tr>
<td>4</td>
<td>Lift heights in heavy compaction zone.</td>
<td>250 mm.</td>
</tr>
<tr>
<td>5</td>
<td>Depth of backfill added over last layer of mesh before additional heavy compacting.</td>
<td>250 mm.</td>
</tr>
</tbody>
</table>
Table H.3: MSE Wall Construction Detail / B&T Queries

<table>
<thead>
<tr>
<th>ITEM</th>
<th>MSE WALL CONSTRUCTION DETAIL / B&amp;T QUERIES</th>
<th>OE RESPONSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate for 152 deep x 305 wide unreinforced concrete levelling pad and 600 deep x 1200 wide structural sub-fill.</td>
<td>Depth varied depending upon Golder instructions.</td>
</tr>
<tr>
<td>2</td>
<td>Grade level, replacing unsuitable material with select backfill &amp; compact.</td>
<td>Used Bridge End Fill for backfill.</td>
</tr>
<tr>
<td>3</td>
<td>Construct levelling pads and cure minimum 12 hours.</td>
<td>Curing time increased 2 to 3 days.</td>
</tr>
<tr>
<td>4</td>
<td>Snap chalk line to strip footing for face of wall line.</td>
<td>Correct.</td>
</tr>
<tr>
<td>5</td>
<td>Place first panel, level and batter 16 in 1524 to fill side.</td>
<td>Batter adjusted to suit, typically 16 mm.</td>
</tr>
<tr>
<td>6</td>
<td>Place adjacent panels, checking level and match batter.</td>
<td>Correct.</td>
</tr>
<tr>
<td>7</td>
<td>Brace the full panels with approved bracing on the outside face.</td>
<td>Correct.</td>
</tr>
<tr>
<td>8</td>
<td>Clamp smaller panels to full panels (refer SSL clamp detail).</td>
<td>Correct.</td>
</tr>
<tr>
<td>9</td>
<td>Glue filter cloth to back face of panels over joints.</td>
<td>Correct.</td>
</tr>
<tr>
<td>10</td>
<td>Place backfill to the level of the first row of loop connectors. Do not place directly against back of first panel. Compact the zone within 900 mm of the back of the wall panel using light mechanical tampers.</td>
<td>Correct.</td>
</tr>
<tr>
<td></td>
<td>NB1: 152 mm levelling pad + 762 mm/2 (height to first embed) = 533 mm, confirm this was done in two x 250 mm compacted lifts.</td>
<td>NB1: Correct.</td>
</tr>
<tr>
<td>11</td>
<td>Describe the sequence for lifts with respect to front 600-900 mm gap behind panels and subsequent light compacting.</td>
<td>The front gap area was filled through the mesh openings; from the top; compaction between the layers of mesh only on the first layer above the mesh level.</td>
</tr>
<tr>
<td>12</td>
<td>Attach mesh to panels with the pin connectors.</td>
<td>Correct.</td>
</tr>
<tr>
<td></td>
<td>NB1: Ledcor QC method to determine that the height of compacted fill was correct.</td>
<td>NB1: Initially with laser level, mostly visual observation.</td>
</tr>
<tr>
<td></td>
<td>NB2: Ledcor QC method to determine that the compacted fill ready to receive a layer of mesh was level.</td>
<td>NB2: Laser level, carpenters level and mostly visual observation.</td>
</tr>
</tbody>
</table>
13. Place next lift(s) of BEF and compact "to top of smaller panel" (SSL Dwg RW-05, for Walls A, B). Given the 250 mm lift allowance, 1-2 more compacted lifts (not in contact with the back of the facing panels) would be applied, then the next row of panels fit on to the "smaller" panels before putting on the 3rd lift and compacting in preparation to lay out and attach the next layer of mesh.

   OE Response: The lift thicknesses were maintained using the offset from the panels until the next mesh level was reached.

14. Repeat above cycle with the SSL note that in subsequent lifts the "fill may be sloped downward toward the panels with in 610 mm (in lieu of 900 mm dim above for first lift) of the back face if necessary".

   OE Response: Correct.

15. Install end and top panels as required, continuing as above until all facing panels are installed.

   OE Response: Correct.

16. "Immediately following the placement of backfill material, lifts were compacted using the CAT CS-323C roller outside of the 610 mm or 910 mm zone as outlined in SSL Dwg RW-04 – 6.0 Erection Sequence. A minimum eight to ten passes with the roller were completed prior to density readings being taken. All in-field density tests recorded 100% compaction [according to nuclear densometer testing]."

   OE Response: Information received from Ledcor by e-mail on 2012 January 27.

17. On SSL Dwg RW-06, the 300 mm depth of "19 [mm] crushed rock under and behind spread footing for even compaction under footing" is up against the rear face of top panel and coping. State compaction method.

   OE Response: Smaller walk behind compactor; CS323 used where width would permit access.

Table H.4: Backfill Compaction – Contractor Quality Control

<table>
<thead>
<tr>
<th>ITEM</th>
<th>BACKFILL COMPACTION - CONTRACTOR QUALITY CONTROL</th>
<th>OE RESPONSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Field density testing method.</td>
<td>Nuclear Densometer.</td>
</tr>
<tr>
<td>2</td>
<td>Field density testing equipment used.</td>
<td>Troxler and Campbell Pacific Gauges.</td>
</tr>
<tr>
<td>3</td>
<td>In-situ density specification used.</td>
<td>MOT Standards.</td>
</tr>
<tr>
<td>4</td>
<td>Per BC MoT SS 202.23, &quot;Each layer shall be compacted to a minimum 100% of the laboratory density obtained by the current ASTM test method D 698&quot;. Confirm the use of this standard and that the target was minimum 100% as noted above.</td>
<td>Confirmed.</td>
</tr>
</tbody>
</table>
ITEM | BACKFILL COMPACTION - CONTRACTOR QUALITY CONTROL | OE RESPONSE
--- | --- | ---
5 | Firm used by Ledcor’s for above in-situ density testing. | Okanagan Testing Labs.
6 | Per ASTM D698, the inclusion of oversize particles (>3/4") requires replacement with smaller particles to complete a standard proctor test in the lab. State the approach taken at Westside Rd. | Industry standard using oversize correction to determine max unit weight.
7 | Frequency of density testing with regard to progressive addition of lifts & subsequent compacting. | Each lift.
8 | Quantity & spacing of density tests during a single density testing cycle for a lift. | Depend on the length of the section, but approximately every 10 metres.
9 | Provision of samples of the Ledcor in-situ density testing records. | Received by B&T, refer Appendix J.

Table H.5: Backfill Compaction – Owner’s Quality Assurance

ITEM | BACKFILL COMPACTION - OWNER’S QUALITY ASSURANCE | OE RESPONSE
--- | --- | ---
1 | Firm used by Owner’s Engineering team for quality assurance of backfill compaction (in-situ density testing). | EBA

Table H.6: Backfill Placement & Compaction – Equipment Used

ITEM | BACKFILL PLACEMENT & COMPACTION - EQUIPMENT USED | OE RESPONSE
--- | --- | ---
1 | Excavator used to place MSE Wall backfill. | Four excavators Hitachi 345; JD270D; Cat ECR 58; 200LC ZAXIS; Hitachi 225 USLC.
2 | Heavy compaction equipment used >900 mm from back of wall face. | Cat CS 323C; Bomag BW 213DH-3.
3 | Light compaction ("light mechanical tampers") equipment used within 900 mm of wall face. | Silver Fox 250 lb.
Appendix I
Photos - MSE Walls and Overpass Construction
I.1 Wall A

Figure I.1: Wall A Construction - Void Exists under the Mesh at Wall; Material Placement through Mesh; Some Backfill Segregation Visible

Figure I.2: Wall A Construction - Void Exists under Mesh at Wall; Mesh arcing up to make Panel Connection
Figure I.3: Light Plate Tamper used Adjacent to Wall Panels (Silver Fox, 250#)

Figure I.4: Mesh Spanning over Large Void in Fill
Figure I.5: Mesh Spanning over Void in Fill
I.2 Wall B

Figure I.6: Wall B Under Construction in Foreground – Wall C and Pier Completed & Supporting East Span Box Stringers in Background

Figure I.7: Taken from Pier Standing on Box Stringers looking at Southbound Off Ramp under Construction. Wall B in Foreground; Wall A Behind
Figure I.8: Placed and Compacted Fill Material about 500 mm above last Mesh Connection (Visible at toe of Sloped Fill) to Facing Panel

Figure I.9: Substantial Void area under Connected Mesh at Facing Panels
Figure I.10: Construction of West Abutment

Figure I.11: West Abutment with Bearings and Dowels in Place. Mud Slab Visible under Abutment
I.3 Wall C

Figure I.12: Void Area under Mesh at Connection to Facing Panels

Figure I.13: Void Areas under Mesh at Connection to Facing Panels
Figure I.14: Void area under Mesh and Upwards Arc of Mesh to meet Connection at Facing Panels

Figure I.15: Wall C and Pier under Construction
Figure I.16: Abutment and Pier Completed. Precast Box Girders in Place in East Span. Coping under Construction

Figure I.17: MSE Wall Completed with Coping. Abutment and Precast Box Girders in Place in East Span
Appendix J
Construction Documentation

This appendix includes samples of the Ledcor QC and Urban Systems QA documentation for the MSE wall construction process.

Appendix J1 – Backfill Electrochemical Test Results

The electrochemical results are included for completeness as these aspects of the backfill are not considered to have contributed to the MSE wall failure.

Backfill electrochemical testing results were performed by EBA Engineering Consultants, Ltd during 2010 April. The sample of backfill material was tested for resistivity, chloride ion content, pH and soluble sulfate ion content. The results were within specification limits set by the Contract, SSL Dwgs and AASHTO Standard Specifications except as noted below.

The soluble sulfate ion content in the EBA sample is 3.5 to 4x the maximum allowed per SSL Dwg RW-04 (Dwg RW-03 for Wall C) and the Contract. However, AASHTO Standard Specifications, Section 5.8.6.1.1 states that "if the resistivity is greater than or equal to 5,000 ohm-cm, the chlorides and sulfates requirements may be waived". As the backfill samples measured 78,000 ohm-cm the waiver would be allowed per AASHTO. Refer Appendix M Levelton Materials Report for further test results.

No test results for plasticity were obtained. The requirement for this is from SSL Dwg RW-04 (Dwg RW-03 for Wall C). The Contract, AASHTO Standard Specifications and BC MoT SS 202, Table 202-B (Aggregate Properties, Bridge End Fill) do not state a requirement for backfill plasticity.

Appendices J2-J4: Walls A,B,C Reinforcing Mesh Mill & Galvanizing Certifications

Mill certifications for the reinforcing mesh were supplied by Concrete Reinforcements, Inc. for Walls A, B and C. These certifications indicate conformance of the supplied D11 and D20 deformed wire to ASTM A-496/A-497, contrary to the requirement for W11 and W20 smooth "cold drawn steel rod conforming to ASTM A-82" as specified on SSL Drawing RW-04 (Dwg RW-03 for Wall C) and AASHTO Standard Specifications 7.6.4.2. The deformed wire noted above is only called out in the SSL drawings as an option for the precast concrete facing panel reinforcement.
The mill certifications show that the D20 wires in Wall A (Hwy 97 southbound off-ramp) and Wall B (West abutment) were from the same heat of material (No. 1190005607). The D20 bars in Wall C (East abutment) were from a separate heat of material (No. 10103008).

The mill certifications show that the D11 wires in Wall A (Hwy 97 southbound off-ramp) and Wall B (West abutment) were from the same heat of material (No. 1090004454). The D11 wires in Wall C (East abutment) were from a separate heat of material (No. 1090004142).

Table J.1: Summary of Material Heat Numbers and Usage in the MSE Walls

<table>
<thead>
<tr>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D11</td>
<td>ASTM A 82 (smooth)</td>
<td>ASTM A 496 (deformed)</td>
<td>1090004454</td>
<td>1090004454</td>
<td>1090004142</td>
</tr>
<tr>
<td>D20</td>
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<td>ASTM A 496 (deformed)</td>
<td>1190005607</td>
<td>1190005607</td>
<td>10103008</td>
</tr>
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</table>

Certification as supplied by AZZ Galvanizing, Arizona, state that the soil reinforcement mesh was hot dip galvanized in accordance with ASTM-A123/A123M-02 in accordance with SSL Drawing RW-04 (Dwg RW-03 for Wall C). No mention is made of the additional galvanizing specification, CAN/CSA G164, given in the Contract; refer Appendix F Review of Contract Special Provisions for discussion. No evidence of testing for embrittlement to either ASTM A143 (as referred to in ASTM A123) or CAN/CSA G164 was included.

Appendices J5-J6: Walls B,C Ledcor QC Checklists (in-situ density test records)

Samples of the Ledcor QC field density reports based on ASTM D6938 Nuclear Method, assess compaction against laboratory Standard Proctor to ASTM D698 using BC MOTH (Ministry of Transportation and Highways) Proctor Density Correction for oversize particles. The sample of documents obtained shows compaction test results generally meeting the 100% target with a few isolated areas at 98-99% and one location down to 96%. Comments from the owner’s field staff indicate that the 100% compaction requirement in the Special Provisions of the contract (referenced to SS202.23) was enforced prior to allowing any further construction to proceed.
Note that there is no requirement in the SSL process to verify compaction within 910 mm of the rear face of the precast panels. SSL Dwg RW-05 (Dwg RW-04 for Wall C) specifically notes that no compaction tests should be taken in this region. The near edge of the Wall B and Wall C spread footings are 818 mm and 308 mm, respectively, from the rear surface of the precast panels and therefore would exert some bearing pressure on this area that received compaction only from light mechanical tampers (refer SSL Dwg RW-04 [Dwg RW-03 for Wall C]; model used was Silver Fox 250#).

Appendix J7-J8: Backfill Gradation: Crusher Output & MSE Wall Construction

Samples of the material gradation curves generated by EBA Engineering Consultants Ltd. (EBA) for the material output from the crushing process that generated the backfill material (referred to as the Site H BEF stockpile) were supplied to B&T. Samples of the material gradation curves generated for Ledcor by Interior Testing Services Ltd (ITS) during the MSE Wall construction process and submitted to USL were also supplied to B&T.

Refer Appendix F Review of Contract Special Provisions for a comparison of the SSL Dwg RW-04 (Dwg RW-03 for Wall C) and BC MoT Standard Specification gradation requirements. The limits shown on the EBA and Interior Testing Services gradation plots are the standard 75 mm maximum (3" minus) used by the BC MoT Standard Specifications for Bridge End Fill.

Comments regarding these documents:

- The two sets of documents include all documents received by USL during the two stages of work. These include both instances that fall within the gradation curve limits and would have been accepted, as well as instances that fall outside the gradation curve limits and would have been rejected by USL;

- Neglecting for the allowance of 100 mm particles per SSL Dwg RW-04 (Dwg RW-03 for Wall C), the set of 19 crusher gradation plots show 3 instances falling outside the curve boundaries (16%). The 84 MSE wall construction gradation plots show 22 instances falling outside the curve boundaries (26%); and

- The 16% and 26% falling outside the MoT SS bounds would be fully within the greater tolerance allowed by the SSL drawings and AASHTO.
The owner's field staff noted that backfill handling issues during site works were resulting in some excessive segregation of material. Construction photo records support that some segregation was occurring during the MSE wall backfilling process. Segregation of material behind the facing panels would tend to aggravate the previously noted issue of aggregate hanging up on the mesh. However, in the absence of segregation, the system used (mesh size, backfill material, construction process) would still have allowed for aggregate hanging up on the mesh.
J.1 Electrochemical Test Results - Backfill


Resistivity of Aggregate
by Reinforced Earth Company (RECO) Method (modified)

**Project No.:** K13101409  
**Sample No.:** L-1  
**Project:** Westside Road Interchange Project  
**Date Sampled:** 7-Apr-10  
**Client:** Urban Systems Ltd.  
**Sampled By:** TJ  
**Date Tested:** 29-Apr-10  
**Attention:** Jim Tait  
**Tested By:** MC  
**Fax:** 250-763-5266  
**Office:** Edmonton  
**Email:** jtait@urban-systems.com  
**Ph:** 250-762-2517

**Description:** 100 mm (-) Crushed Gravel, trace sand, trace silt.  
**Source:** Site "H", BEF Material  
**Sample Location:** Loader assisted stockpile sample.  
**Supplier:**

<table>
<thead>
<tr>
<th>Resistivity of Aggregate (Wetted Condition)</th>
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<tbody>
<tr>
<td>Resistivity = ( R \times S / L )</td>
</tr>
<tr>
<td><strong>Box Used:</strong> Large Box</td>
</tr>
<tr>
<td>Where: ( L ) = Length of sample box (cm)</td>
</tr>
<tr>
<td>( h ) = height of sample in box</td>
</tr>
<tr>
<td>( S ) = Cross-sectional area of sample (cm(^2))</td>
</tr>
<tr>
<td>( R ) = Resistance (Ohms)</td>
</tr>
</tbody>
</table>

| L = 44.8 cm  |
| h = 10.1 cm  |
| S = 198.97 cm\(^2\)  |
| R = 17500 Ohms           |

\[ \text{Resistivity} = \frac{17500 \times 198.97}{44.8} \div 100 \]

\[ \text{Resistivity} = \boxed{780} \text{ Ohm} \cdot \text{m} \]

**Remarks:** Distilled Water Used

---

*Data presented herein is for the sole use of the stipulated client. EBA is not responsible, nor can be held liable, for use made of this report by any other party, with or without the knowledge of EBA. The testing services reported herein have been performed by an EBA technician to recognized industry standards, unless otherwise noted. No other warranty is made. These data do not include or represent any interpretation or opinion of specification compliance or material suitability. Should engineering interpretation be required, EBA will provide it upon written request.*
# Water Soluble Chloride Ion Content in Concrete

**Project:** Westside Road Interchange Project  
**Sample Number:** L-1  
**Project Number:** K13101409  
**Date Sampled:** 7-Apr-10  
**Client:** Urban Systems Ltd.  
**By:** TJ  
**Attention:** Jim Tait  
**Tested By:** EM  
**Supplier:**  
**Date Tested:** 29-Apr-10  
**Sample Location/Description:** Site "H", BEF Material - Loader assisted stockpile sample.

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<tr>
<th>Station</th>
<th>Offset</th>
<th>L-1</th>
<th>L-1 Rep</th>
<th>Offset</th>
<th>Depth (mm)</th>
<th>Beaker/Flask Number</th>
<th>Weight of Sample (g)</th>
<th>Measured Value (ppm)</th>
<th>Chloride Content (ppm)</th>
<th>Chloride Content %</th>
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**Remarks:**

---

**Checked By:**

---

For Internal Use Only
**pH CONTENT IN CONCRETE**

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<th>Westside Road Interchange Project</th>
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<td>K13101409</td>
<td>Date Sampled:</td>
<td>7-Apr-10</td>
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<tr>
<td>Client:</td>
<td>Urban Systems Ltd.</td>
<td>Sampled By:</td>
<td>TJ</td>
</tr>
<tr>
<td>Attention:</td>
<td>Jim Tait</td>
<td>Tested By:</td>
<td>EM</td>
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Sample Location/Description: Site "H", BEF Material - Loader assisted stockpile sample.

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<tr>
<td>Measured pH</td>
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<tbody>
<tr>
<td>Depth: (mm)</td>
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<td>Measured pH</td>
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<tr>
<td>Depth: (mm)</td>
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<td>Measured pH</td>
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Remarks: 

Checked By: [Signature]
## SOLUBLE SULPHATE ION CONTENT OF SOIL
(CSA Designation A23.2-2B & A23.2-3B)

**Project:** Westside Road Interchange Project

**Sample No.:** L-1

**Project No.:** K13101409  
**Date Tests:** 29-Apr-10  
**By:** EM

**Client:** Urban Systems Ltd.  
**Sample Source:** Site "H", BEF - April 7, 2010

<table>
<thead>
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<th>Sample Number</th>
<th>L-1</th>
<th>L-1 Rep</th>
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<tr>
<td>Borehole Number</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crucible Number</td>
<td>J</td>
<td>X</td>
</tr>
<tr>
<td>Mass of Crucible &amp; Precipitate (g)</td>
<td>12.6611</td>
<td>12.5394</td>
</tr>
<tr>
<td>Mass of Crucible (g)</td>
<td>12.6593</td>
<td>12.5374</td>
</tr>
<tr>
<td>Mass of Precipitate (g)</td>
<td>0.0018</td>
<td>0.002</td>
</tr>
<tr>
<td>Sulphate Content %</td>
<td>0.07</td>
<td>0.08</td>
</tr>
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</table>

**Degree of Exposure (Class):** Negligible

<table>
<thead>
<tr>
<th>Class of exposure</th>
<th>Degree of exposure</th>
<th>Water-soluble sulphate (SO₄)† in soil sample, %</th>
<th>Sulphate (SO₄) in groundwater samples, mg/l.‡</th>
<th>Water soluble sulphate (SO₄) in recycled aggregate sample, %</th>
<th>Cementing materials to be used§</th>
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<tbody>
<tr>
<td>S-1</td>
<td>Very severe</td>
<td>&gt; 2.0</td>
<td>&gt; 10 000</td>
<td>&gt; 2.0</td>
<td>HS or HSb</td>
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<tr>
<td>S-2</td>
<td>Severe</td>
<td>0.20–2.0</td>
<td>1500–10 000</td>
<td>0.60–2.0</td>
<td>HS or HSb</td>
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<tr>
<td>S-3</td>
<td>Moderate</td>
<td>0.10–0.20</td>
<td>150–1500</td>
<td>0.20–0.60</td>
<td>MS, MSb, LH, HS, or HSb</td>
</tr>
</tbody>
</table>

*For sea water exposure, see Clause 4.1.1.5.
† In accordance with CSA A23.2-2B.
‡ In accordance with CSA A23.2-2B.
§ Cementing material combinations with equivalent performance may be used (see Clauses 4.2.1.2, 4.2.1.3, and 4.2.1.4). Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates. Refer to Clause 4.1.1.6.3.

**Limitations:**

i) The degree of exposure class included herein are valid only if drainage and weeping systems meet the requirements of the site conditions.

ii) The degree exposure class should be re-verified if backfill soils for foundation walls originate from an unknown source.

**Remarks:**

For Internal Use Only

EBA Engineering Consultants Ltd.
J.2 Wall A Mill Certifications
March 22, 2011

Ledcor CMI, Ltd.
1200-1067 West Cordova St.
Vancouver, BC V6C-1C7
Attention: Amri Benjamin

Reference: Hwy 97- MSE Wall A
            Contract No. 22402-00
            SSL Job # 2310
            Material Certification

Dear Amri,

Please find attached certificates of compliance for the following MSE wall material:

Galvanized Welded Wire Reinforcement:
  • Fabricated by Concrete Reinforcements, Inc. according to ASTM A-496/A-497
  • Galvanized by Arizona Galvanizing according to ASTM A-123/A-123M-02

Miscellaneous Wall Components:
  • 4 oz geo-textile Fabric as supplied by WhiteCap Industries,
  • HDPE Bearing Pads, as supplied by Hayes Industries
  • OSI sealant, as supplied by WhiteCap Industries

The above listed material as supplied for the above referenced project complies with the standard specifications and project special provisions.

Should you have any questions or require additional information, please don’t hesitate to call.

Sincerely,

Nicole Thompson
March 9, 2011

SSL
4740 Scotts Valley Dr.
Suite E
Scotts Valley, CA 95066

RE: OUR ORDER#: Z103-175
JOB#: 2310
JOB NAME#: HWY 97
MSE WALLS#: A
MATERIAL: Mesh and Pins

PROCESS CERTIFICATION

This is to certify the material referenced above has been hot dip galvanized in accordance with ASTM-A123/A123M-02 specifications.

Sincerely,

Oscar Ochoa, Jr.
Shipping Manager
March 10, 2011

SSL
4740 Scotts Valley Dr.
Suite E
Scotts Valley, CA 95066

RE: OUR ORDER#: Z103-200
     JOB#: 2310
     JOB NAME#: HWY 97
     MSE WALLS#: A
     MATERIAL: Mesh and Pins

PROCESS CERTIFICATION

This is to certify the material referenced above has been hot dip galvanized in accordance with ASTM-A123/A123M-02 specifications.

Sincerely,

Oscar Ochoa, Jr.
Shipping Manager
CONCRETE REINFORCEMENTS, INC.

QUALITY CONTROL DEPARTMENT

TEST DATA SHEET

DATE: 3/7/2011
CUSTOMER: SSL

PROJECT NUMBER: 13065A1

TESTING DATE: 3/4/11

| 13056A1-1  | 8XV D11XD11 TAG: HWY 97 - WALL A - 4W - 11X11X.5' - 8F - IIA - 117 SHEETS |
| 13056A1-3  | 8XV D11XD11 TAG: HWY 97 - WALL A - 4W - 11X11X.5' - 12F - GGE1 - 60 SHEETS |
| 13056A1-5  | 8XV D11XD11 TAG: HWY 97 - WALL A - 4W - 11X11X.5' - 13F - FFF3 - 80 SHEETS |
| 13056A1-6  | 8XV D11XD11 TAG: HWY 97 - WALL A - 4W - 11X11X.5' - 14F - CCG1 - 107 SHEETS |

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
<th>D11</th>
</tr>
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<tbody>
<tr>
<td>HEAT NUMBER</td>
<td>1090004454</td>
</tr>
<tr>
<td>SAMPLE NUMBER</td>
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<tr>
<td>LOAD (LBS)</td>
<td>9880</td>
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<tr>
<td>TENSILE (PSI)</td>
<td>89.8K</td>
</tr>
<tr>
<td>BEND TEST</td>
<td>PASS</td>
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<tr>
<td>YIELD STRENGTH</td>
<td>76.9K</td>
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<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
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The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-487 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2790
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 3/7/2011
CUSTOMER: SSL

<table>
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<th>TESTING DATE:</th>
<th>3/4/11</th>
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<tbody>
<tr>
<td>13066A1-7</td>
<td>8XV D11XD11 TAG: HWY 97 - WALL A - 4W - 11X11X1'- 18F - RI3-3E SHEETS</td>
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<td>13066A1-11</td>
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</tbody>
</table>

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
<th>D11</th>
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</thead>
<tbody>
<tr>
<td>HEAT NUMBER</td>
<td>1090004454</td>
</tr>
<tr>
<td>SAMPLE NUMBER</td>
<td>1</td>
</tr>
<tr>
<td>LOAD (LBS)</td>
<td>9880</td>
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<tr>
<td>TENSILE (PSI)</td>
<td>89.8K</td>
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<tr>
<td>BEND TEST</td>
<td>PASS</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>76.8K</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2790
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 3/7/2011
CUSTOMER: SSL
TESTING DATE: 3/4/11
PROJECT NUMBER: 13066A1

| 13065A1-13 | 8XV D11XD11 TAG: HWY 97 - WALL A - 6W - 11X11X1' - 14F - CCG3-90 SHEETS |
| 13066A1-14 | 8XV D11XD11 TAG: HWY 97 - WALL A - 6W - 11X11X1' - 16F - R12-30 SHEETS |
| 13066A1-16 | 8XV D11XD11 TAG: HWY 97 - WALL A - 6W - 11X11X1' - 20F - PM3-48 SHEETS |

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

| WIRE SIZE | D11 |
| HEAT NUMBER | 1090004454 |
| SAMPLE NUMBER | 1 |
| LOAD (LBS) | 9880 |
| TENSILE (PSI) | 89.8K |
| BEND TEST | PASS |
| YIELD STRENGTH | 76.8K |
| WELD SHEAR | PASS |

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2990
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 3/8/2011
CUSTOMER: SSL
PROJECT NUMBER: 13068A2
TESTING DATE: 3/7/11

<table>
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<tr>
<th>PROJECT</th>
<th>WIRE SIZE</th>
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<tr>
<td>13068A2-1</td>
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<tr>
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The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

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<th>PROPERTY</th>
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<td>SAMPLE NUMBER</td>
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<tr>
<td>LOAD (LBS)</td>
<td>9880</td>
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</tr>
<tr>
<td>TENSILE (PSI)</td>
<td>88.6K</td>
<td>85.7K</td>
</tr>
<tr>
<td>BEND TEST</td>
<td>PASS</td>
<td>PASS</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>76.9K</td>
<td>79.3K</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwree
Shawn K. McElwree
Quality Assurance

13450 W. Peoria Ave.  Surprise, Arizona 85379  Phone 623-975-2970  Fax 623-975-2790
DATE: 3/8/2011
CUSTOMER: SSL
TESTING DATE: 3/7/11

PROJECT NUMBER: 13066A2

13066A2-7 8XV D20XD11 TAG: HWY 97 - WALL A - 6W - 20X11X1' - 20F - EXT MIN 15 SHEETS
13066A2-8 W31 X 27" PIN TAG: HWY 97 - WALL A - SMOOTH PIN - 2318 PCS

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
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</tr>
<tr>
<td>LOAD (LBS)</td>
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<td>17150</td>
</tr>
<tr>
<td>TENSILE (PSI)</td>
<td>89.8K</td>
<td>85.7K</td>
</tr>
<tr>
<td>BEND TEST</td>
<td>PASS</td>
<td>PASS</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>76.9K</td>
<td>79.3K</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-436/A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379  Phone 623-975-2970  Fax 623-975-2790
J.3 Wall B Mill Certifications
April 19, 2011

Ledcor CMI, Ltd.
1200-1067 West Cordova St.
Vancouver, BC V6C-1C7
Attention: Amri Benjamin

Reference: Hwy 97- MSE Wall B
Contract No. 22402-00
SSL Job # 2310
Material Certification

Dear Amri,

Please find attached certificates of compliance for the following MSE wall material:

Galvanized Welded Wire Reinforcement:
- Fabricated by Concrete Reinforcements, Inc. according to ASTM A-496/A-497
- Galvanized by Arizona Galvanizing according to ASTM A-123/A-123M-02

Miscellaneous Wall Components:
- 4 oz geo-textile Fabric as supplied by WhiteCap Industries,
- HDPE Bearing Pads, as supplied by Hayes Industries
- OSI sealant, as supplied by WhiteCap Industries

The above listed material as supplied for the above referenced project complies with the standard specifications and project special provisions.

Should you have any questions or require additional information, please don’t hesitate to call.

Sincerely,

[Signature]
Nicole Thompson
March 28, 2011

SSL
4740 Scotts Valley Dr.
Suite E
Scotts Valley, CA 95066

RE: OUR ORDER#: Z003-654
JOB#: 2310
JOB NAME#: Hwy-97
MSE WALLS#: B
MATERIAL: Mesh and Pins

PROCESS CERTIFICATION

This is to certify the material referenced above has been hot dip galvanized in accordance with ASTM-A123/A123M-02 specifications.

Sincerely,

Oscar Ochoa, Jr.
Shipping Manager
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 3/28/2011
CUSTOMER: SSL
TESTING DATE: 3/25/11
PROJECT NUMBER: 13066B

<table>
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<tr>
<td>SAMPLE NUMBER</td>
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<td>LOAD (LBS)</td>
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<tr>
<td>TENSILE (PSI)</td>
<td>89.8K</td>
</tr>
<tr>
<td>BEND TEST</td>
<td>PASS</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>76.9K</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2790
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 3/28/2011
CUSTOMER: SSL
TESTING DATE: 3/25/11

<table>
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<tr>
<td>8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X.5 - 14F - CG1-1 SHEETS</td>
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<td>13086B-8</td>
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<td>8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X.5 - 16F - F11-2 SHEETS</td>
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<td>13086B-11</td>
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<td>8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X1.5 - 10F - EL5-22 SHEETS</td>
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<td>13086B-12</td>
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<td>8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X..5 - 14F - CG1-4 SHEETS</td>
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The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
<th>D11</th>
<th>D20</th>
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<tbody>
<tr>
<td>HEAT NUMBER</td>
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</tr>
<tr>
<td>SAMPLE NUMBER</td>
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<td>1</td>
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<tr>
<td>LOAD (LBS)</td>
<td>9880</td>
<td>17150</td>
</tr>
<tr>
<td>TENSILE (PSI)</td>
<td>89.8K</td>
<td>85.7K</td>
</tr>
<tr>
<td>BEND TEST</td>
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<tr>
<td>YIELD STRENGTH</td>
<td>76.9K</td>
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<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
<td>PASS</td>
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</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-976-2970 Fax 623-975-2790
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 3/28/2011
CUSTOMER: SSL

TESTING DATE: 3/25/11

<table>
<thead>
<tr>
<th>PROJECT NUMBER: 13066B</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
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<td>13066B-14 8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X1'-17F - D1-7 SHEETS</td>
</tr>
<tr>
<td>13066B-15 8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X1'-17F - DJ3-28 SHEETS</td>
</tr>
<tr>
<td>13066B-16 8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X1'-19F - EL1-22 SHEETS</td>
</tr>
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<td>13066B-17 8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X1'-19F - EL5-22 SHEETS</td>
</tr>
<tr>
<td>13066B-18 8XV D20XD11 TAG: HWY 97 - WALL B - 6W - 20X11X1'-28F - EXTU1-10 SHEETS</td>
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</table>

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
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<th>D20</th>
</tr>
</thead>
<tbody>
<tr>
<td>HEAT NUMBER</td>
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<td>1190005607</td>
</tr>
<tr>
<td>SAMPLE NUMBER</td>
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<td>1</td>
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<tr>
<td>LOAD (LBS)</td>
<td>9880</td>
<td>17150</td>
</tr>
<tr>
<td>TENSILE (PSI)</td>
<td>89.8K</td>
<td>85.7K</td>
</tr>
<tr>
<td>BEND TEST</td>
<td>PASS</td>
<td>PASS</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>76.6K</td>
<td>79.5K</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records.

This document certifies that the material mentioned above meets the required A.S.T.M. A-496/A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2790
J.4  Wall C Mill Certifications
November 4, 2010

Ledcor CMI, Ltd.
1200-1067 West Cordova St.
Vancouver, BC V6C-1C7
Attention: Amri Benjamin

Reference: Hwy 97 - MSE Wall C
Contract No. 22402-00
SSL Job # 2310
Material Certification

Dear Amri,

Please find attached certificates of compliance for the following MSE wall material:

Galvanized Welded Wire Reinforcement:
- Fabricated by Concrete Reinforcements, Inc. according to ASTM A-496/A-497
- Galvanized by Arizona Galvanizing according to ASTM A-123/A-123M-02

Miscellaneous Wall Components:
- 4 oz geo-textile Fabric as supplied by WhiteCap Industries,
- HDPE Bearing Pads, as supplied by Hayes Industries
- OSI sealant, as supplied by WhiteCap Industries

The above listed material as supplied for the above referenced project complies with the standard specifications and project special provisions.

Should you have any questions or require additional information, please don't hesitate to call.

Sincerely,

Nicole Thompson
October 25, 2010

SSL
4740 Scotts Valley Dr.
Suite E
Scotts Valley, CA 95066

JOB#: 2310
JOB NAME#: HWY 97
WALLS#: C
MATERIAL: Mesh and Pins

PROCESS CERTIFICATION

This is to certify the material referenced above has been hot dip galvanized in accordance with ASTM-A123/A123M-02 specifications.

Sincerely,

Oscar L. Ochoa
Operations Manager
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 10/22/2010
CUSTOMER: SSL

TESTING DATE: 10/21/10

<table>
<thead>
<tr>
<th>PROJECT NUMBER: 3318A</th>
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<td>3318A-5</td>
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<td>3318A-6</td>
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</table>

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
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<th>WIRE SIZE</th>
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<td>SAMPLE NUMBER</td>
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<tr>
<td>LOAD (LBS)</td>
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<tr>
<td>TENSILE (PSI)</td>
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<td>92.2k</td>
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<td>BEND TEST</td>
<td>PASS</td>
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<tr>
<td>YIELD STRENGTH</td>
<td>82.7k</td>
<td>81.8k</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave.  Surprise, Arizona 85379  Phone 623-975-2970  Fax 623-975-2790
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 10/22/2010
CUSTOMER: SSL
TESTING DATE: 10/21/10
PROJECT NUMBER: 3318A

<table>
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<th>TEST DESCRIPTION</th>
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<td>3318A-8</td>
<td>8XV D20XD11 TAG: HWY 97, WALL C - 6W - 20X11X1.5' - 23F - EP5 - 32 SHEETS</td>
</tr>
<tr>
<td>3318A-9</td>
<td>8XV D20XD11 TAG: HWY 97, WALL C - 6W - 20X11X2.5' - 23F - EP6 - 64 SHEETS</td>
</tr>
<tr>
<td>3318A-10</td>
<td>8XV D20XD11 TAG: HWY 97, WALL C - 7W - 20X11X.5' - 15F - HH1 - 8 SHEETS</td>
</tr>
<tr>
<td>3318A-11</td>
<td>8XV D20XD11 TAG: HWY 97, WALL C - 7W - 20X11X.5' - 17F - GJ1 - 9 SHEETS</td>
</tr>
<tr>
<td>3318A-12</td>
<td>8XV D20XD11 TAG: HWY 97, WALL C - 7W - 20X11X1' - 22F - FO3 - 4 SHEETS</td>
</tr>
</tbody>
</table>

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
<th>HEAT NUMBER</th>
<th>SAMPLE NUMBER</th>
<th>LOAD (LBS)</th>
<th>TENSILE (PSI)</th>
<th>YIELD STRENGTH</th>
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<td>82.7K</td>
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<td>1</td>
<td>18440</td>
<td>92.2K</td>
<td>81.8K</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-496/ A-497 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwee
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2790
Concrete Reinforcements, Inc.
Quality Control Department
Test Data Sheet

DATE: 10/22/2010
CUSTOMER: SSL
TESTING DATE: 10/21/10
PROJECT NUMBER: 3318A

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<th>YIELD STRENGTH</th>
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</tr>
</tbody>
</table>

The mechanical properties of the material ordered were tested for compliance with A.S.T.M. A-496 & A-497 the results are as follows:

<table>
<thead>
<tr>
<th>WIRE SIZE</th>
<th>D11</th>
<th>D20</th>
<th>W31</th>
</tr>
</thead>
<tbody>
<tr>
<td>HEAT NUMBER</td>
<td>1090004142</td>
<td>10103008</td>
<td>515022</td>
</tr>
<tr>
<td>SAMPLE NUMBER</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>LOAD (LBS)</td>
<td>10320</td>
<td>18440</td>
<td>22300</td>
</tr>
<tr>
<td>TENSILE (PSI)</td>
<td>93.8K</td>
<td>92.2K</td>
<td>72K</td>
</tr>
<tr>
<td>BEND TEST</td>
<td>PASS</td>
<td>PASS</td>
<td>PASS</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>82.7K</td>
<td>81.8K</td>
<td>60.8K</td>
</tr>
<tr>
<td>WELD SHEAR</td>
<td>PASS</td>
<td>PASS</td>
<td>PASS</td>
</tr>
</tbody>
</table>

The undersigned certifies that the above is a true and accurate representation of the test results obtained on the described material as appearing on company records. This document certifies that the material mentioned above meets the required A.S.T.M. A-486/A-487 designation. All of the material herein has been melted and manufactured in the U.S.A.

Very Truly Yours,
Concrete Reinforcements, Inc.

Shawn K. McElwec
Shawn K. McElwee
Quality Assurance

13450 W. Peoria Ave. Surprise, Arizona 85379 Phone 623-975-2970 Fax 623-975-2790
J.5 Wall B Quality Control Checklists & Documentation
# Quality Control Check List

**Hwy 97 Westside Interchange Project, Phase 2**

**Form No. 202**

<table>
<thead>
<tr>
<th>MoT Project Number 221402-0000</th>
<th>LCMI Project Number 6210007</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element of Work:</strong> Granular Surfacing, Base, Sub-Bases</td>
<td><strong>Date of Inspection:</strong> June 16, 2017</td>
</tr>
<tr>
<td><strong>Applicable References:</strong> MoT Standard Specifications (SS) Section 202 Special Provisions (SP) 6.04</td>
<td><strong>Time of Inspection:</strong> 4:00 PM</td>
</tr>
</tbody>
</table>

**PART A – SUBMITTAL/APPROVAL REQUIREMENTS**

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Item Description of Required Submittals</th>
<th>(Y/N)</th>
<th>Contract Reference Section</th>
<th>Date Required</th>
<th>Date Submitted</th>
<th>Accepted? If NO, complete Part C</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>Have Aggregate results for surfacing, base, sub-base and bridge end fill showing they meet the requirements of Table 202-B been submitted?</td>
<td>Y</td>
<td>SS 202.04.02</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-2</td>
<td>Has a Design Aggregate Gradation been declared in writing?</td>
<td>Y</td>
<td>SS 202.20.01</td>
<td>&quot;...within production of the first 10% of a given aggregate classification&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-3</td>
<td>Has the Construction Project Manager assessed the pertinent foundation conditions of the bridge and embankments and authorized continuation of construction?</td>
<td>N/A</td>
<td>SS 202.23</td>
<td>&quot;...prior to the construction of the bridge end fill...&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PART B – TEST/INSPECTION REQUIREMENTS**

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Item/Inspection of Required Tests and Inspections</th>
<th>Contract Reference Section</th>
<th>Compliant?</th>
<th>If NO, was NCR issues? (Y/N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-1</td>
<td>Has a Proposed Design Aggregate Gradation field adjustment been submitted in writing with supporting documentation?</td>
<td>SS 202.20.03</td>
<td>√</td>
<td></td>
</tr>
<tr>
<td>I-2</td>
<td>Was the sub-grade noted to be soft due to excessive moisture?</td>
<td>SS 202.22.02</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>I-3</td>
<td>Are lifts of bridge end fill being placed at the required thickness? (i.e., lifts not exceeding 150mm) 10&quot; lifts (250mm) has been approved</td>
<td>SS 202.23</td>
<td>√</td>
<td></td>
</tr>
</tbody>
</table>

(Continued on next page)
| I-4 | Has a Standard Proctor been determined for this material? | SS 202.23  
SS 202.25.02  
SS 202.26.02 | Y |
| I-5 | Are the required 100% (Standard Proctor) densities being achieved? | SS 202.23  
SS 202.25.02  
SS 202.26.02 | Y |
| I-6 | Are the finished surfaces to the required tolerances for line and grade? | SS 202.26.04 | Y |
| I-7 | Has the base course been Proof Rolled by a 9 tonne single axle dual tire or 17 tonne tandem axle group with dual tires with a tire pressure of 600kPa? | SS 202.29 | Y |
| I-8 | Does the gradation of drainage course material follow the table in section 6.04 of the Special Provisions? | SP 6.04 | Y |

**PART C – ACTION REQUIREMENTS**

<table>
<thead>
<tr>
<th>Referenced Item No.</th>
<th>Describe Action Taken or Proposed to be Taken</th>
<th>Follow-up Required (Y/N)</th>
<th>If YES, Action By?</th>
<th>Target Completion Date</th>
</tr>
</thead>
</table>

**PART D – OTHER COMMENTS/ OBSERVATIONS**

<table>
<thead>
<tr>
<th>Referenced Item No.</th>
<th>Described other comments or observations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>All consecutive lifts have made 100% compaction</td>
</tr>
<tr>
<td></td>
<td>Columns 1-10 first wash 11 to 37 second wash</td>
</tr>
</tbody>
</table>

Completed By: [Signature]  | Reviewed By: [Signature]  
Print Name: [Name]  | Print Name: [Name]  
Title: [Title]  | Title: [Title]  
Date: [Date]  | Date: [Date]  
Signature: [Signature]  | Signature: [Signature]  
Date: [Date]  | Date: [Date]
**FIELD DENSITY REPORT**

**PROJECT NO. 001698**

**CLIENT LEDCOR CMI LTD**

**C.C.**

**PROJECT HWY 97 - WESTSIDE ROAD INTERCHANGE**

**MATERIALS TESTING**

**HIGHWAY 97**

**W KELOWNA**

**REPORT NO. 203**

**NO. OF DENSITIES 4**

**TESTED BY PS**

**DATE TESTED 2011.Jun.16**

**CONTRACTOR**

BC GENERAL CONTRACTING

**AREA**

MSE WALL 'B'

**CONSTRUCTION TYPE**

GRID STABILIZATION BACKFILL

**TIME TESTED 15:00**

<table>
<thead>
<tr>
<th>DENSITY NUMBER</th>
<th>LOCATION</th>
<th>LAB REFERENCE AND</th>
<th>MOISTURE FIELD</th>
<th>OPTIMUM</th>
<th>OVERSIZE MATERIAL</th>
<th>DRY DENSITY FIELD</th>
<th>DRY DENSITY LAB</th>
<th>COMPACATION %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MSE WALL 'B' @ 10+10-3m O/S WALL (2ND GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>4.8</td>
<td>5.9</td>
<td>25.0</td>
<td>2193</td>
<td>2160</td>
<td>102</td>
</tr>
<tr>
<td>2</td>
<td>MSE WALL 'B' @ 10+30-4.5m O/S WALL (3RD GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>4.6</td>
<td>5.9</td>
<td>25.0</td>
<td>2205</td>
<td>2160</td>
<td>102</td>
</tr>
<tr>
<td>3</td>
<td>MSE WALL 'B' @ 10+50-2m O/S WALL (3RD GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>4.4</td>
<td>5.9</td>
<td>25.0</td>
<td>2175</td>
<td>2160</td>
<td>101</td>
</tr>
<tr>
<td>4</td>
<td>MSE WALL 'B' @ 10+65-2m O/S WALL (3RD GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>4.9</td>
<td>5.9</td>
<td>25.0</td>
<td>2191</td>
<td>2160</td>
<td>101</td>
</tr>
</tbody>
</table>

**FIELD METHOD**

Nuclear ASTM D6938

**LABORATORY METHOD**

Standard Proctor ASTM D698

**ROCK CORRECTION METHOD**

BC MOTH Proctor Density Correction

**OVERSIZE SCREEN SIZE**

Passing 3/4" - 19mm

**SPECIFIED COMPACATION 100**

**LOW DENSITIES INDICATED ***

**COMMENTS**

---

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.

Report System Software Registered to: Okanagan Testing Laboratories, Kelowna
**FIELD DENSITY REPORT**

**PROJECT:** Mission Rd

**CONTRACTOR:** BC General Contracting

**OTL PROJECT NO.:** 1698

**TIME ON SITE:** 15:00

**DATE TESTED:** June 16, 11

**CLIENT:** LEDCOR CAN

**AREA:** MSE wall B

**TESTED BY:** PS

**NO. OF DENSITIES:** 4

**CONSTRUCTION TYPE:** Sand Stabilization Excavate

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DENSITY TEST LOCATION</th>
<th>LAB PROCTOR REFERENCE</th>
<th>MOISTURE (%)</th>
<th>OVERSIZE MATERIAL</th>
<th>WET DENSITY (Kg/m³)</th>
<th>DRY DENSITY (Kg/m³)</th>
<th>COMPACTION %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MSE wall B (0-10+10)</td>
<td>5</td>
<td>4.855</td>
<td>75</td>
<td>2298</td>
<td>2107 2160</td>
<td>102</td>
</tr>
<tr>
<td>2</td>
<td>4.5m clears (3' previous)</td>
<td>4.9</td>
<td></td>
<td></td>
<td>2306</td>
<td>2159</td>
<td>102</td>
</tr>
<tr>
<td>3</td>
<td>7m clears (11)</td>
<td>4.4</td>
<td></td>
<td></td>
<td>2271</td>
<td>2174</td>
<td>101</td>
</tr>
<tr>
<td>4</td>
<td>2m clears (4)</td>
<td>4.9</td>
<td></td>
<td></td>
<td>7298</td>
<td>2191</td>
<td>102</td>
</tr>
</tbody>
</table>

**FIELD METHOD** Nuclear ASTM D2922

**LABORATORY PROCTOR METHOD (v)** Standard (ASTM D698)

**ROCK CORRECTION METHOD (v)** ASTM

**SPECIFIED COMPACTION (%)** 100

**B.C. M.O.T.H.**

**COMMENTS:**

---

OKANAGAN TESTING LABORATORIES LTD. PER.
TO
LEDCOR CMI LTD
1500 - 1055 W HASTINGS STREET
VANCOUVER, BC
V6E 2E9

PROJECT NO. 001698
CLIENT LEDCOR CMI LTD
C.C.

PROJECT HWY 97 - WESTSIDE ROAD INTERCHANGE MATERIALS TESTING HIGHWAY 97
W KELOWNA

REPORT NO. 199 NO. OF DENSITIES 3 TESTED BY PS DATE TESTED 2011.Jun.16

<table>
<thead>
<tr>
<th>DENSITY NUMBER</th>
<th>LOCATION</th>
<th>LAB REFERENCE AND</th>
<th>MOISTURE</th>
<th>OVERSIZED MATERIAL</th>
<th>DRY DENSITY</th>
<th>COMPACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WALL 'B' @ 10+15, 2m O/S WALL (0.6m ABOVE 1ST GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>5.7</td>
<td>5.9 25.0</td>
<td>2167 2160</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>WALL 'B' @ 10+35, 3m O/S WALL (0.3m ABOVE 2ND GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>5.3</td>
<td>5.9 25.0</td>
<td>2189 2160</td>
<td>101</td>
</tr>
<tr>
<td></td>
<td>WALL 'B' @ 10+65, 4m O/S WALL (0.3m ABOVE 2ND GRID)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>5.4</td>
<td>5.9 25.0</td>
<td>2178 2160</td>
<td>101</td>
</tr>
</tbody>
</table>

FIELD METHOD Nuclear ASTM D6938
LABORATORY METHOD Standard Proctor ASTM D698
ROCK CORRECTION METHOD BC MOTH Proctor Density Correction
OVERSIZE SCREEN SIZE Passing 3/4" - 19mm

COMMENTS

Page 1 of 1 2011.Jun.17 OKANAGAN TESTING LABORATORIES PER.  
Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.
PROJECT NO. 001698  
CLIENT LEDCOR CMI LTD  

LEDCOR CMI LTD  
1500 - 1055 W HASTINGS STREET  
VANCOUVER, BC  
V6E 2E9

PROJECT HWY 97 - WESTSIDE ROAD INTERCHANGE  
HIGHWAY 97  
W KELOWNA

MATERIALS TESTING

REPORT NO. 200  NO. OF DENSITIES 3  TESTED BY PS  
DATE TESTED 2011.Jun.16

<table>
<thead>
<tr>
<th>DENSITY NUMBER</th>
<th>LOCATION</th>
<th>LAB REFERENCE AND</th>
<th>MOISTURE FIELD</th>
<th>MOISTURE OPTIMUM</th>
<th>OVERSIZE MATERIAL FIELD</th>
<th>OVERSIZE MATERIAL LAB</th>
<th>DRY DENSITY FIELD</th>
<th>DRY DENSITY LAB</th>
<th>COMPACTION %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>WALL 'B' @ 10+15, 8m O/S WALL (0.6m ABOVE 1ST GRID)</td>
<td>Proctor 2 50mm SILT</td>
<td>8.1</td>
<td>10.4</td>
<td>12.0</td>
<td>1982</td>
<td>2029</td>
<td>98</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>WALL 'B' @ 10+35, 10m O/S WALL (0.3m ABOVE 2ND GRID)</td>
<td>Proctor 2 50mm SILT</td>
<td>7.8</td>
<td>10.4</td>
<td>12.0</td>
<td>2010</td>
<td>2029</td>
<td>99</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>WALL 'B' @ 10+65, 8m O/S WALL (0.3m ABOVE 2ND GRID)</td>
<td>Proctor 2 50mm SILT</td>
<td>7.2</td>
<td>10.4</td>
<td>12.0</td>
<td>2006</td>
<td>2029</td>
<td>99</td>
<td></td>
</tr>
</tbody>
</table>

FIELD METHOD Nuclear ASTM D6938  
LABORATORY METHOD Standard Proctor ASTM D698  
ROCK CORRECTION METHOD BC MOOTH Proctor Density Correction  
OVERSIZE SCREEN SIZE Passing 3/4" - 19mm  

SPECIFIED COMPACTION 95%  
LOW DENSITIES INDICATED *

COMMENTS

Page 1 of 1  2011.Jun.17  OKANAGAN TESTING LABORATORIES PER.

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.
**PROJECT:** Westside Rd Interchange  
**CONTRACTOR:** BC General (Contractor)  
**OTL PROJECT NO.:** 1698  
**TIME ON SITE:** 8:00  
**DATE TESTED:** Jun 16, 14  
**CLIENT:** Lepox (CM)  
**AREA:** MSE Wall B  
**TESTED BY:** PS  
**NO. OF DENSITIES:** 6  
**CONSTRUCTION TYPE:** Grid Stabilization Backfill

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DENSITY TEST LOCATION</th>
<th>LAB PROCEDURE REFERENCE</th>
<th>MOISTURE (%)</th>
<th>OVERSIZE MATERIAL</th>
<th>WET DENSITY (Kg/m³)</th>
<th>DRY DENSITY (Kg/m³)</th>
<th>COMPACTION %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Wall B @ 10 + 15 2 m N of SW Corner</td>
<td>5 7 2 9 1 1 2 7 1 6 7 1 6 0</td>
<td>5.7 5.9</td>
<td>2291</td>
<td>2167 7 1 6 0</td>
<td>107</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>8 m N of SW Corner</td>
<td>c 7 8 1 0 4</td>
<td>6.1 1 0 4</td>
<td>1 2 7 1 4 3</td>
<td>1 9 8 2 2 0 2 9</td>
<td>98</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>10 m N of SW Corner</td>
<td>c 1 0 4 3 5</td>
<td>7 8</td>
<td>2 1 6 7</td>
<td>2 0 1 0</td>
<td>&quot;</td>
<td>99</td>
</tr>
<tr>
<td>4</td>
<td>16 m N of SW Corner</td>
<td>c 1 0 4 6 5</td>
<td>5 3 5 3 7 5</td>
<td>2 3 0 5</td>
<td>2 1 8 9 2 1 6 0</td>
<td>1 0 1</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>11 m N of SW Corner</td>
<td>c 1 0 4 7 5</td>
<td>5 4 8 1</td>
<td>2 2 9 6</td>
<td>2 1 9 7 8 2 0 2 9</td>
<td>1 0 1</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>8 m N of SW Corner</td>
<td>c 7 7 1 0 4</td>
<td>2 7 7 1 0 4</td>
<td>2 1 5 0</td>
<td>2 0 0 6 2 2 2 9</td>
<td>9 8</td>
<td></td>
</tr>
</tbody>
</table>

**FIELD METHOD:** Nuclear ASTM D2922  
**SPECIFIED COMPACTION (%):** 100  

**LABORATORY PROCEDURE METHOD (v):**  
- Standard (ASTM D698)  
- Modified (ASTM D1557)  
- B.C. M.O.T.H.  

**COMMENTS:**  
SPECD COMPACT FOR TEST 425% 96%  
PS FRICTION

OKANAGAN TESTING LABORATORIES LTD.
# FIELD DENSITY REPORT

**PROJECT NO.** 001698  
**CLIENT** LEDCOR CMI LTD  
**C.C.**

---

**PROJECT HWY 97 - WESTSIDE ROAD INTERCHANGE**  
**MATERIALS TESTING**  
**HIGHWAY 97**  
**W KELOWNA**

**REPORT NO.** 197  
**NO. OF DENSITIES** 5  
**TESTED BY** PS  
**DATE TESTED** 2011-Jun-15  
**TIME TESTED** 08:30

**CONTRACTOR** BC GENERAL CONTRACTING  
**AREA** MSE WALL "B"  
**CONSTRUCTION TYPE** GRID STABILIZATION BACKFILL

<table>
<thead>
<tr>
<th>DENSITY NUMBER</th>
<th>LOCATION</th>
<th>LAB REFERENCE AND</th>
<th>MOISTURE FIELD</th>
<th>MOISTURE OPTIMUM</th>
<th>OVERSIZE MATERIAL</th>
<th>DRY DENSITY FIELD</th>
<th>DRY DENSITY LAB</th>
<th>COMPACTION %</th>
</tr>
</thead>
</table>
| 1              | WALL 'B' @ 10+10, 2m O/S WALL (0.3m ABOVE 1ST GRID) | Proctor 5  
75mm CRUSH | 4.5 | 5.9 | 25.0 | 2171 | 2160 | 101 |
|                | WALL 'B' @ 10+30, 4.5m O/S WALL (0.3m ABOVE 2ND GRID) | Proctor 5  
75mm CRUSH | 5.9 | 5.9 | 25.0 | 2184 | 2160 | 101 |
| 2              | WALL 'B' @ 10+30, 8m O/S WALL (0.3m ABOVE 2ND GRID) | Proctor 2  
50mm SILT | 7.5 | 10.4 | 12.0 | 1956 | 2029 | 96 * |
| 3              | WALL 'B' @ 10+50, 2m O/S WALL (0.3m ABOVE 2ND GRID) | Proctor 5  
75mm CRUSH | 5.8 | 5.9 | 25.0 | 2181 | 2160 | 101 |
| 4              | WALL 'B' @ 10+70, 4m O/S WALL (0.3m ABOVE 2ND GRID) | Proctor 5  
75mm CRUSH | 4.3 | 5.9 | 25.0 | 2161 | 2160 | 100 |

**FIELD METHOD** Nuclear ASTM D6938  
**LABORATORY METHOD** Standard Proctor ASTM D698  
**ROCK CORRECTION METHOD** BC MOTH Proctor Density Correction  
**OVERSIZE SCREEN SIZE** Passing 3/4" - 19mm

**SPECIFIED COMPACTION** 100  
**SPECIFIED COMPACTION FOR DENSITY #3 = 95% STD PROCTOR.**

---

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<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DENSITY TEST LOCATION</th>
<th>LAB PROCTOR REFERENCE</th>
<th>MOISTURE (%)</th>
<th>OVERSIZE MATERIAL</th>
<th>WET DENSITY (Kg/m³)</th>
<th>DRY DENSITY (Kg/m³)</th>
<th>COMPACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Wall B @ 10 + 10</td>
<td></td>
<td>45.5%</td>
<td>25</td>
<td>2269</td>
<td>2171</td>
<td>101</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Wall B @ 20 + 00</td>
<td></td>
<td>55.1%</td>
<td>1</td>
<td>2313</td>
<td>2180</td>
<td>102</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Aid with (10)</td>
<td>2</td>
<td>75.10%</td>
<td>15</td>
<td>2102</td>
<td>1956</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Aid with (10)</td>
<td>5</td>
<td>58.58%</td>
<td>25</td>
<td>2208</td>
<td>2181</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Aid with (10)</td>
<td>4</td>
<td>43.11%</td>
<td>1</td>
<td>2254</td>
<td>2161</td>
<td>100</td>
</tr>
</tbody>
</table>

FIELD METHOD: Nuclear ASTM D2922

SPECIFIED COMPACTION (%): 100

LABORATORY PROCTOR METHOD (Y): Standard (ASTM D698)  
Modified (ASTM D1557)  

ROCK CORRECTION METHOD (Y): ASTM  
B.C. M.O.T.H.  

COMMENTS:  
CPFM D. EMBAYMENT TEST: #3  96% COMPACTION  

OKANAGAN TESTING LABORATORIES LTD.  PER.
# MSE Wall - Inspection Record

<table>
<thead>
<tr>
<th>Project:</th>
<th>Westside Road Interchange</th>
<th>Date:</th>
<th>14/6/2011</th>
</tr>
</thead>
<tbody>
<tr>
<td>Owner:</td>
<td>MoT/WFN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contractor:</td>
<td>Ledcor/BCG</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MSE Wall ID:</td>
<td>MSE Wall A, B</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Location:</td>
<td>Column ID: 1-38, L100 line</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Activity:

- WALL PANEL INSTALLATION

- Installed proper top panels on column ID 31, 33, 35, 37

<table>
<thead>
<tr>
<th>Remarks:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>✓ Install Mesh</td>
<td></td>
</tr>
<tr>
<td>□ Do Not Install Mesh</td>
<td></td>
</tr>
</tbody>
</table>

- WIRE MESH INSTALLATION

- Installed mesh from column ID 1 to 37
- Proper mesh installed for given column ID and level
- Replacement mesh used on column ID 21, 6W20x0.50 (20") used instead of damaged 5W20x1.50 (19")

<table>
<thead>
<tr>
<th>Remarks:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>✓ Place Backfill</td>
<td></td>
</tr>
<tr>
<td>□ Do Not Place Backfill</td>
<td></td>
</tr>
</tbody>
</table>

This field memorandum is provided as preliminary information only and is subject to confirmation and acceptance by the client.

Ledcor CMI Ltd.

**Ledcor Representative**

Name: **James Lowe**

Signature: [Signature]

**BCG Representative**

Name: **Artur Beint**

Signature: [Signature]
J.6 Wall C Quality Control Checklists & Documentation
Quality Control Check List

Hwy 97 Westside Road Interchange
Project, Phase 2

Form No. 202

MoT Project Number 22402-0000
LCMI Project Number 6210007

Element of Work: Granular Surfacing, Base, Sub-Bases, Bridge End fill
Applicable References: Standard Specifications (SS) Section 202
Special Provisions (SP) 6.04

R.S.E. Wall C all 10" lift 100% up to 2nd mesh
(2nd row of mesh)

PART A – SUBMITTAL/APPROVAL REQUIREMENTS

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Item Description of Required Submittals</th>
<th>(Y/N)</th>
<th>Contract Reference Section</th>
<th>Date Required</th>
<th>Date Submitted</th>
<th>Accepted? If NO, complete Part C</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>Have Aggregate results for surfacing, base, sub-base and bridge end fill showing they meet the requirements of Table 202-B been submitted?</td>
<td>√</td>
<td>SS 202.04.02</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-2</td>
<td>Has a Design Aggregate Gradation been declared in writing?</td>
<td>√</td>
<td>SS 202.20.01</td>
<td>&quot;...within production of the first 10% of a given aggregate classification&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-3</td>
<td>Has the Construction Project Manager assessed the pertinent foundation conditions of the bridge and embankments and authorized continuation of construction?</td>
<td>√</td>
<td>SS 202.23</td>
<td>&quot;...prior to the construction of the bridge end fill...&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-4</td>
<td>Has a Proposed Design Aggregate Gradation field adjustment been submitted in writing with supporting documentation?</td>
<td>√</td>
<td>SS 202.20.03</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

PART B – TEST/INSPECTION REQUIREMENTS

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Item/Inspection of Required Tests and Inspections</th>
<th>Contract Reference Section</th>
<th>Compliant? If NO, complete Part C</th>
<th>If NO, was NCR issues? (Y/N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-1</td>
<td>Was the sub-grade moisture content acceptable? (no excessive rutting)</td>
<td>SS 202.22.02</td>
<td>√</td>
<td></td>
</tr>
<tr>
<td>I-2</td>
<td>Are lifts being placed at the required thickness? (loose lifts not exceeding 150mm) Embankments: layers of 200mm except for top 500mm must be 100mm lifts Endfill, sub-base, granular course, base; 150mm loose lift thickness</td>
<td>SS 202</td>
<td>√</td>
<td></td>
</tr>
<tr>
<td>I-3</td>
<td>Has a Standard Proctor been determined for this material? ASTM D698</td>
<td>SS 202</td>
<td>√</td>
<td></td>
</tr>
<tr>
<td>I-4</td>
<td>Are the required (Standard Proctor) densities being achieved for the respective material? Embankments 95% except top 300mm must achieve 100%</td>
<td>SS 202.23</td>
<td>√</td>
<td></td>
</tr>
</tbody>
</table>
  SS 202.25.02 | √ |                             |
<table>
<thead>
<tr>
<th>Referenced Item No.</th>
<th>Describe Action Taken or Proposed to be Taken</th>
<th>Follow-up Required (Y/N)</th>
<th>If YES, Action By?</th>
<th>Target Completion Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-5</td>
<td>Sufficient water used to achieve optimum moisture content?</td>
<td>SS 202.60.02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I-6</td>
<td>Are the finished surfaces to the required tolerances for line and grade?</td>
<td>SS 202.60.03</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I-7</td>
<td>Has the base course been Proof Rolled by a 9 tonne single axle dual tire or 17 tonne tandem axle group with dual tires with a tire pressure of 600kPa?</td>
<td>SS 202.29</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PART C – ACTION REQUIREMENTS**

**PART D – OTHER COMMENTS/OBSERVATIONS**

<table>
<thead>
<tr>
<th>Referenced Item No.</th>
<th>Described other comments or observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-2</td>
<td>10&quot; lifts have been approved, all lifts up to 2nd row of mesh have been 100% densities</td>
</tr>
</tbody>
</table>

Completed By: Todd Beynonchuk
Print Name: Todd Beynonchuk
Title: For man
Signature: [Signature]
Date: Nov 16/2010

Reviewed By: [Signature]
Print Name: [Print Name]
Title: [Title]
Date: [Date]

Ami Bernstein / Project Manager
Signature: [Signature]
Date: 10/2/11
**FIELD DENSITY REPORT**

**PROJECT NO. 001698**
**CLIENT LEDCOR CMI LTD**
**C.C.**

**LEDCOR CMI LTD**
**1500 - 1055 W HASTINGS STREET**
**VANCOUVER, BRITISH COLUMBIA**
**V6E 2E9**

**PROJECT HWY 97 - WESTSIDE ROAD INTERCHANGE**
**MATERIALS TESTING**

**HIGHWAY 97**
**W KELOWNA**

**REPORT NO. 21**
**NO. OF DENSITIES 3**
**TESTED BY OIL (PS)**
**DATE TESTED 2010. NOV. 15**

<table>
<thead>
<tr>
<th>DENSITY NUMBER</th>
<th>LOCATION</th>
<th>LAB REFERENCE AND</th>
<th>MOISTURE FIELD</th>
<th>MOISTURE OPTIMUM</th>
<th>OVERSIZE MATERIAL</th>
<th>DRY DENSITY FIELD</th>
<th>DRY DENSITY LAB</th>
<th>COMPACTION %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MID WALL, 2m S O/S (3rd LIFT ON 1st MESH)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>7.6</td>
<td>5.9</td>
<td>25.0</td>
<td>2187</td>
<td>2160</td>
<td>101</td>
</tr>
<tr>
<td></td>
<td>5m S, 9m E OF W END OF WALL (3rd LIFT ON 1st MESH)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>5.6</td>
<td>5.9</td>
<td>25.0</td>
<td>2164</td>
<td>2160</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>2m S, 2m E OF W END OF WALL (3rd LIFT ON 1st MESH)</td>
<td>Proctor 5 75mm CRUSH</td>
<td>5.8</td>
<td>5.9</td>
<td>25.0</td>
<td>2164</td>
<td>2160</td>
<td>100</td>
</tr>
</tbody>
</table>

**FIELD METHOD**
Nuclear ASTM D6938

**LABORATORY METHOD**
Standard Proctor ASTM D698

**ROCK CORRECTION METHOD**
BC MOTH Proctor Density Correction

**OVERSIZE SCREEN SIZE**
Passing 3/4" - 19mm

**REPORTING**
Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.

Report System Software Registered to: Okanagan Testing Laboratories, Kelowna

---

Page 1 of 1 2010. Nov. 18  OKANAGAN TESTING LABORATORIES. PER.
**PROJECT:** Westside Interchange  
**CONTRACTOR:** B.C. General OTL  
**PROJECT NO.:** 1698  
**TIME ON SITE:** 15:00  
**DATE TESTED:** Nov 15, 2010  
**CLIENT:** L.E.C.O.R.  
**AREA:** MSE Wall'  
**TESTED BY:** PS  
**NO. OF DENSITIES:**  
**CONSTRUCTION TYPE:** Bridge and Backfill

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DENSITY TEST LOCATION</th>
<th>LAB PROCTOR REFERENCE</th>
<th>MOISTURE (%)</th>
<th>OVERSIZE MATERIAL</th>
<th>WET DENSITY (Kg/m³)</th>
<th>DRY DENSITY (Kg/m³)</th>
<th>COMPACTON %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MSE Wall Core</td>
<td>1.5</td>
<td>7.6</td>
<td>25</td>
<td>2373</td>
<td>2160</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>MSE for YME of N End W1 (3:1)</td>
<td>45</td>
<td>5.6</td>
<td>25</td>
<td>2185</td>
<td>2169</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>MSE for YME of N End W2 (3:1)</td>
<td>12</td>
<td>6.8</td>
<td>2289</td>
<td>2163</td>
<td></td>
<td>100</td>
</tr>
</tbody>
</table>

**FIELD METHOD**  
Nuclear ASTM D2922  
**SPECIFIED COMPACTON (%)** 100

**LABORATORY PROCTOR METHOD (✓)**  
Standard (ASTM D698)  
Modified (ASTM D1557)

**ROCK CORRECTION METHOD (✓)**  
ASTM  
B.C. M.O.T.H.

**COMMENTS:**

OKANAGAN TESTING LABORATORIES LTD.  
PER.
# Field Density Report

## Project Information
- **Project No.**: 000698
- **Client**: LEDCOR CMI LTD
- **Address**: 1500 - 1055 W HASTINGS STREET, VANCOUVER, BRITISH COLUMBIA, V6E 2E9
- **Highway**: 97
- **Location**: WESTSIDE ROAD INTERCHANGE
- **Reports No.**: 20
- **No. of Densities**: 4
- **Date Tested**: 2010, Nov. 15
- **Tested By**: OTL (PS)
- **Time Tested**: 12:00

## Contractor Information
- **Contractor**: BC GENERAL CONTRACTING
- **Area**: MSE WALL 'C'
- **Construction Type**: BRIDGE-END BACKFILL

## Density Details

<table>
<thead>
<tr>
<th>Density Number</th>
<th>Location</th>
<th>Lab Reference and</th>
<th>Moisture</th>
<th>Oversize Material</th>
<th>Dry Density</th>
<th>Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Field</td>
<td>Optimum</td>
<td>Field</td>
<td>Lab</td>
</tr>
<tr>
<td>1</td>
<td>4m S, 8m E of W End of Wall (1st Lift on 1st Mesh)</td>
<td>75mm Proctor 5</td>
<td>4.8</td>
<td>5.9</td>
<td>2155</td>
<td>2160</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CRUSH</td>
<td></td>
<td>FIELD</td>
<td>LAB</td>
</tr>
<tr>
<td>1</td>
<td>1.5m S of W End of Wall (1st Lift on 1st Mesh)</td>
<td>75mm Proctor 5</td>
<td>4.0</td>
<td>5.9</td>
<td>2158</td>
<td>2160</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CRUSH</td>
<td></td>
<td>FIELD</td>
<td>LAB</td>
</tr>
<tr>
<td>3</td>
<td>2m S, 10m E of W End of Wall (2nd Lift on 1st Mesh)</td>
<td>75mm Proctor 5</td>
<td>5.1</td>
<td>5.9</td>
<td>2170</td>
<td>2160</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CRUSH</td>
<td></td>
<td>FIELD</td>
<td>LAB</td>
</tr>
<tr>
<td>4</td>
<td>4m S, 4m E of W End of Wall (2nd Lift on 1st Mesh)</td>
<td>75mm Proctor 5</td>
<td>4.7</td>
<td>5.9</td>
<td>2159</td>
<td>2160</td>
</tr>
</tbody>
</table>

## Field Method
- Nuclear ASTM D6938

## Laboratory Method
- Standard Proctor ASTM D698

## Rock Correction Method
- BC MOTH Proctor Density Correction

## Oversize Screen Size
- Passing 3/4" - 19mm

## Comments

SPECIFIED COMPACTION: 100

LOW DENSITIES INDICATED:

* Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of test results is provided only on written request.

Report System Software Registered to: Okanagan Testing Laboratories, Kelowna
### Field Density Report

**Project:** Westside Interchange  
**Client:** Leduc  
**Contractor:** BC General  
**OTL Project No.:** 1698  
**Time on Site:** 12:00  
**Date Tested:** Nov. 15, 10  
**Area:** MSE Wall C  
**Tested By:** PS  
**No. of Densities:** 4  

**Construction Type:** SGSB Backfill

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density Test Location</th>
<th>Lab Proctor Reference</th>
<th>Moisture (%)</th>
<th>Oversize Material</th>
<th>Wet Density (Kg/m³)</th>
<th>Dry Density (Kg/m³)</th>
<th>Compaction %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4m 5 of 5m E Wend (Trench 5)</td>
<td>405110545</td>
<td>4.88</td>
<td>125</td>
<td>2258</td>
<td>2154</td>
<td>160</td>
</tr>
<tr>
<td>2</td>
<td>1.5m S of Wend Wall (1st Lift)</td>
<td>405110545</td>
<td>4.0</td>
<td></td>
<td>2144</td>
<td>2158</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>2m S of 10m Wend (2nd Lift)</td>
<td>405110545</td>
<td>5.1</td>
<td></td>
<td>2281</td>
<td>2170</td>
<td>101</td>
</tr>
<tr>
<td>4</td>
<td>4m S of 4m E of Wend Wall 11</td>
<td>405110545</td>
<td>7.7</td>
<td></td>
<td>2260</td>
<td>2159</td>
<td>100</td>
</tr>
</tbody>
</table>

**Field Method:** Nuclear ASTM D2922  
**Specified Compaction (%):** 100

**Laboratory Proctor Method:** Standard (ASTM D698)  
**Rock Correction Method:** ASTM

**Comments:**

---

OKANAGAN TESTING LABORATORIES LTD.  
PER.
J.7  Gradation Reports - Backfill Generation (Crushing)
TO: Jim Tait
COMPANY: Urban Systems Ltd.

EMAIL: jtait@urban-systems.com
DATE: March 22, 2010
FILE: K13101409
FROM: Heather for Mike Laverdiere
PAGE 1 OF: 3

SUBJECT: Westside Road Interchange Project

Aggregate Analysis Report, Sample No. 4547, 4548
AGGREGATE ANALYSIS REPORT
ASTM D422

Project: Westside Road Interchange Project
Client: Urban Systems Ltd.
Project No.: K13101409
Description: 100mm(-) Gravel, some sand, trace silt.
Source: Manufactured Crushed Blast Rock
Test Date: March 18, 2010
Sample No.: 4547

<table>
<thead>
<tr>
<th>Sieve Sizes</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm</td>
<td>100</td>
</tr>
<tr>
<td>75 mm</td>
<td>94.4</td>
</tr>
<tr>
<td>50 mm</td>
<td>75.9</td>
</tr>
<tr>
<td>19 mm</td>
<td>42.6</td>
</tr>
<tr>
<td>4.75 mm</td>
<td>20.8</td>
</tr>
<tr>
<td>1.18 mm</td>
<td>12.5</td>
</tr>
<tr>
<td>300 µm</td>
<td>9.1</td>
</tr>
<tr>
<td>75 µm</td>
<td>4.9</td>
</tr>
</tbody>
</table>

Remarks: Sampled from production stockpile at Site "F" by TJ on March 17, 2010.
Material plotted to Mot 2009 Standard Specifications, Table 202-C Bridge End Fill gradation requirements. Initial Total sample Mass = 61,258.5g.

Reviewed By: [Signature]

Data presented herein is for the sole use of the stipulated client. EBA is not responsible, nor can be held liable, for use made of this report by any other party, with or without the knowledge of EBA. The testing services reported herein have been performed by an EBA technician to recognised industry standards, unless otherwise noted. No other warranty is made. These data do not include or represent any interpretation or opinion of specifications compliance or material suitability. Should engineering interpretation be required, EBA will provide it upon written request.
AGGREGATE ANALYSIS REPORT
ASTM D422

Project: Westside Road Interchange Project
Client: Urban Systems Ltd.
Project No.: K13101409
Description: 100mm(-) Gravel, trace sand, trace silt.
Source: Manufactured Crushed Blast Rock
Test Date: March 18, 2010
Sample No.: 4548

<table>
<thead>
<tr>
<th>Sieve Sizes</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm</td>
<td>100</td>
</tr>
<tr>
<td>75 mm</td>
<td>97.6</td>
</tr>
<tr>
<td>50 mm</td>
<td>80.3</td>
</tr>
<tr>
<td>19 mm</td>
<td>34.9</td>
</tr>
<tr>
<td>4.75 mm</td>
<td>15.5</td>
</tr>
<tr>
<td>1.18 mm</td>
<td>12.7</td>
</tr>
<tr>
<td>300 µm</td>
<td>11.1</td>
</tr>
<tr>
<td>75 µm</td>
<td>5.8</td>
</tr>
</tbody>
</table>

Remarks: Sampled from production stockpile at Site "H" by TJ on March 17, 2010.
Material plotted to Mot 2009 Standard Specifications, Table 202-C Bridge End Fill gradation requirements. Initial Total sample Mass = 53,560.4g.

Reviewed By: [Signature]

Data presented herein is for the sole use of the stipulated client. EBA is not responsible, nor can be held liable, for use made of this report by any other party, with or without the knowledge of EBA. The testing services reported herein have been performed by an EBA technician to recognized industry standards, unless otherwise noted. No other warranty is made. These data do not include or represent any interpretation or opinion of specification compliance or material suitability. Should engineering interpretation be required, EBA will provide it upon written request.
TO: Jim Tait

COMPANY: Urban Systems Ltd.

EMAIL: jttait@urban-systems.com

DATE: May 28, 2010

FILE: K13101409

FROM: Heather for Mike Laverdiere

PAGE 1 OF: 2

SUBJECT: Westside Road Interchange Project

Aggregate Analysis Report, Sample No. 4683
Project: Westside Road Interchange Project
Client: Urban Systems Ltd.
Project No.: K13101409
Description: 100mm(-) Gravel, some sand, trace silt.
Source: Manufactured Crushed Blast Rock
Test Date: May 27, 2010
Sample No.: 4683

Remarks: Sampled from production stockpile (excavator assisted) at Site "G" by TJ on May 27, 2010.
Material plotted to Mot 2009: Standard Specifications, Table 202-C Bridge
End Fill gradation requirements. Initial Total sample Mass = 63,752.0 g.

Reviewed By: [Signature]
J.8 Gradation Reports - MSE Wall Construction
Client: BC General Contracting
Sample Description: 3" Minus
Location: STOCKPILE, W.Side Road Interchange Job
Specification: MOTH - Bridge End Fill
Sampled by: DD
DATE: 10-01-12
Screen Analysis: ST
DATE: 10-01-12
Copies To: BC General Contracting, Attn: Jason
MOTH
Bridge End Fill
Section 202-C
75 mm

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Specification</th>
<th>Total % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>100 - 100</td>
<td>92.1</td>
</tr>
<tr>
<td>50</td>
<td>30 - 100</td>
<td>71.1</td>
</tr>
<tr>
<td>19</td>
<td>20 - 100</td>
<td>36.0</td>
</tr>
<tr>
<td>4.75</td>
<td>10 - 60</td>
<td>15.5</td>
</tr>
<tr>
<td>1.18</td>
<td>6 - 32</td>
<td>8.3</td>
</tr>
<tr>
<td>0.300</td>
<td>4 - 15</td>
<td>6.0</td>
</tr>
<tr>
<td>0.075</td>
<td>0 - 5</td>
<td>3.2</td>
</tr>
</tbody>
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Client: Westside Road Interchange, Ph.1
Sample Description: 3” Minus
Location: Site "H"
Specification: MOTH - Bridge End Fill
Sampled by: DD
Screen Analysis: RH
Copies To: BC General Contracting, Attn: Jason

DATE: 10-04-26
DATE: 10-04-26
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Client: Westside Road Interchange, Ph.1
Sample Description: 3" Minus
Location: Site "F"
Specification: MOTH - Bridge End Fill
Sampled by: DD
Screen Analysis: DD
Copies To: BC General Contracting, Attn: Jason
Appendix K
Survey Results

Surveys have been completed to establish the present elevations of key bridge and wall elements. The surveys were intended to identify structure and site settlements, the extent and magnitudes of such settlements and their relevance to the wall facing collapse.

BC MoT surveyors completed a survey of key structural elements on 2011 December 09. McElhanney Engineering Consultants Ltd. completed an additional survey of the bridge deck and of the top of pavement of Highway 97 on 2012 January 24. The raw data from the surveys is attached. The BC MoT was not initially tied into the project datum but was corrected later by 23 mm in order to tie the survey elevations to the project datum.

In addition, we have received the deck screed elevation survey completed by the Contractor dated 2011 July 22 and an as-built top of pavement survey completed by Urban Systems Ltd. on 2011 December 18.

Survey results pertaining to the bridge structure are summarized on the attached sheets titled Summary of Settlement & Rotation. Highway 97 elevations were taken by McElhanney and these are summarized on the attached drawing 1973-SK-006.

K.1 Highway 97 Elevations

Highway 97 elevations were surveyed in order to assess the extent of any overall site settlement. Top of pavement elevations along the west edges of the highway were found to be about 25 mm in general below the target design elevations. The elevation difference is reasonably consistent along the length surveyed.

Pavement elevations along the Highway 97 median are significantly lower than the target design elevations where the median includes a barrier (generally between points 353 to 386). The extent of the elevation differences ranges between about 30 and 80 mm. It is clear though from surveyed pier elevations and girder elevations, that the bridge pier underwent no significant settlement and as such, it appears that the Highway 97 median pavement elevations indicate that the road was constructed at or near the surveyed elevations, lower than required by design. A scenario whereby the pier including its buried spread footing underwent little settlement while the highway directly above the footing underwent significant settlement is not plausible.
Pavement elevations along the east edge of the highway on ramp were found to be varying. The highway section to the north of the bridge seems to deviate by 0 to 65 mm from the design elevations (points 324 to 342) while the highway section to the south of the bridge seems to deviate by 16 mm (below design) to 52 mm (above design). The data for the east edge of the highway pavement is deemed to be inconclusive albeit, there's no evidence to support the occurrence of a significant overall site settlement that would have triggered the MSE wall facing collapse.

**K.2 Bridge Abutments and Differential Settlement**

The BC MoT and McElhanney surveys indicate that the west abutment (Wall B) is lower than its intended design elevation by up to 60 mm at the south end and 74 mm at the north end of the abutment. The east abutment (Wall C) is lower than its intended design elevation by up to 66 mm at the south end and 58 mm at the north end of the abutment. The apparent settlement of both abutments is attributed to the voids and light compaction of fill within a 900 mm wide zone behind the precast MSE wall panels. The east and west abutment footings partially rest on the soil in the 900 mm wide zone.

Note that as settlement was occurring under the abutments within the backfill containing the reinforcing mesh, a similar degree of vertical movement did not occur in the precast concrete facing panels resting on the concrete levelling pad and subgrade. Survey indicates that the top of coping for Wall C settled 13 to 17 mm from design elevation. That gives a relative vertical displacement of 41 to 53 mm, refer Figure D.12. The mesh, embedded in the backfill and connected to the facing panels would experience this differential settlement at the panel connection.

According to the surveys, the east abutment has rotated by 1.6% to 3.4% towards the pier. The surveys indicate that the west abutment has not rotated to any appreciable extent. The west abutment is situated further back from the MSE wall face so less of its footing bears on the soil within the 900 mm zone while the east abutment footing is close to the MSE wall face and comparatively, more of its footing rests on soil within the 900 mm zone. The wall rotation is significant where the abutment footing is closer to the MSE wall face and where the soil support is weaker over a wider area under the footing.
K.3 Bridge Girders

A survey completed by Urban Systems on the 2011 July 22 for the purpose of establishing screed elevations, indicated that the bridge precast concrete girders were lower than their intended design elevations by:

- 12 mm at the north end of the west abutment;
- 18 mm at the south end of the west abutment;
- 28 mm at the north end of the east abutment; and
- 38 mm at the south end of the east abutment.

At the time of the survey, the concrete deck, bridge parapets and concrete median were yet to be constructed.

Surveys completed by BC MoT surveyors and McElhanney Consultants Ltd. after the facing panel collapse are in reasonably close agreement and indicate that the bridge girders are below their intended design elevations by:

- 64 to 66 mm at the north end of the west abutment;
- 50 to 61 mm at the south end of the west abutment;
- 38 to 44 mm at the north end of the east abutment; and
- 41 to 48 mm at the south end of the east abutment.

The above suggests that some appreciable settlement had occurred after the precast concrete girders were installed and that further structural settlements had occurred after construction of the bridge deck primarily at the west abutment.
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*Average of inner and outer edge of footing

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*Top of exterior girder roughly 1m from end of girder

NOTES:
1. 2011-07-22 data from Urban Systems screed calculations
2. 2011-12-09 survey by BCMoT (corrected by adding 0.022m)
3. 2012-01-19 abutment rotation data collected by B&T
4. 2012-01-24 survey by McElhanney
5. All units are in meters unless noted otherwise
6. Settlements are shown relative to original design elevation, positive downwards
7. Positive rotation indicates top of wall rotation toward pier
8. File location E:\1973\NIUX\1973 Settlement
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### NOTES:
1. 2011-12-09 survey by BCMoT (corrected by adding 0.022m)
2. 2011-12-18 survey by Urban Systems
3. 2012-01-24 survey by McElhanney
4. Coping as built data from Marcon drawings dated 2011-09-22
5. All units are in meters unless noted otherwise
6. Settlements are shown relative to original design elevation, positive downwards
7. File location E:\1973\NIUX\1973 Settlement
Appendix L

Wire Breakage Maps

The west abutment support wall (Wall B) and the east abutment support wall (Wall C) are MSE wall systems that comprise precast concrete facing panels connected to soil reinforcing grids of galvanized welded wire mesh. The soil reinforcing girds have longitudinal wires that are bent upwards to form a 90° hook. The hooks connect to the embedded loops on the rear face of the panels and are held in place with a locking bar.

The standard panels have two rows of soil reinforcing mesh. The larger panels at the top of Wall B and Wall C have three rows of soil reinforcing mesh. Each reinforcing mesh typically has six longitudinal wires. Mesh in rows 1, 2 and 7 (numbered from top to bottom) in Column 4 through 21 of Wall B have five longitudinal wires and mesh in row 3 in Column 5 through 20 of Wall C have seven longitudinal wires. The longitudinal wires in Columns 3 through 22 of Wall B and Columns 2 through 23 of Wall C are D20 wires. The remaining columns have D11 longitudinal wires.

L.1 West Abutment Support Wall (Wall B)

On 2011 November 20, a portion of the precast concrete facing panels of Wall B collapsed. The panels that collapsed were the top two rows of panels in Column 5 through 15 and single panels from Columns 4 and 16 as shown in attached sketch 1973-SK003. An assessment of the collapse on November 22 revealed that all except one of the D20 longitudinal wires for the collapsed panels were broken at the bend where the mesh is connected to the facing panels.

The panels in Column 1 through 22 at Wall B were removed down to but not including the bottom panels embedded in the roadway grade. This includes all columns that contain D20 longitudinal wires in the reinforcing mesh. During the sequential removal of the panels below and to either side of the collapsed area, which occurred from November 22 to December 4, the condition of the longitudinal wires was observed. In the collapse area, all but one of the wires were broken in the first five rows of mesh counting from the top of the wall and under the abutment footing (excluding row 5, wall column 15 where there was no data). A reduction in the number of breaks occurs in the sixth or seventh row of mesh as well as to either side of the footing. At the base of the wall most of the wires were intact. In all instances, the wire breaks were at the bend locations. As discussed in Appendix M, it is
believed that many of these wire breaks pre-dated the collapse of the facing panels. Some information regarding the condition of the reinforcing wires in Column 15 to 22 of Wall B is missing because the D20 wires in this area were cut for cold-galvanizing before field staff could inspect the wires. Sketch 1973-SK003 is an elevation view of the Wall B showing all the longitudinal reinforcing wires and their condition during deconstruction. The broken wires are represented in red, intact wires that were cut to remove panels are represented in green and wires with no information are represented in grey.

L.2 East Abutment Support Wall (Wall C)

Following the deconstruction of Wall B, all the panels in Column 2 through 23 in Wall C were removed down to but not including the bottom panels embedded in the roadway grade. During the sequential removal of the panels, which occurred December 8 to January 6, the condition of the longitudinal wires was observed. The pattern of longitudinal wire breaks in Wall C was similar to Wall B. At the top of the wall, near the abutment footing, all of the wires were broken. The breaks began to diminish further down the wall and at the base of the wall most of the wires were intact. Across the width of the wall, the breaks are concentrated under the abutment footing and diminish to either side. In all instances, the wire breaks were at the bend locations.

Attached sketch 1973-SK004 is an elevation view of the Wall C showing all the longitudinal reinforcing wires and their condition during deconstruction. Again, the broken wires are represented in red, intact wires that were cut to remove panels are represented in green and wires with no information are represented in grey.

In addition to cataloguing longitudinal wire failures, observations were made of the wires, panels and soil in an undisturbed state by looking behind panels that remained in place beside the panels being removed. These observations confirmed the significant relative vertical movement between the underpass abutment footing and the MSE wall facing panels. The relative vertical displacement of the broken mesh ends directly beneath the abutment footing was measured as approximately 75 mm (Appendix D, Figure D.14). This value is larger than the 41 to 53mm of differential settlement from the survey information (refer Appendix K Survey Results), but must include for initial drape of the mesh due to elevation mismatch between the mesh and the panel embeds.
As noted above, observations during deconstruction show a relative difference in the positions of soil reinforcing mesh in the backfill to broken hook connections locked to the loop embedded in the facing panels. Typically, whenever broken wires were found at the connection to the facing panels, the wire had been displaced downwards relative to the panel connection. As both the MSE soil mass and the levelling pad under the facing panels rest on the subgrade, this indicates settlement within the MSE soil mass causing downward displacement of the reinforcement with respect to the adjacent facing panel connection. The relative movement was generally greater at the top of the wall than the bottom.

The wall also appeared to have moved horizontally away from the soil slightly. The greatest degree of horizontal movement was observed at the top of the wall, where it was measured as approximately 25 mm, as was evidenced by the horizontal offset of the wall panels relative to the broken wire ends.
Appendix M
Levelton Materials Report
ANALYSIS OF MSE WALL FAILURE
WESTSIDE ROAD UNDERPASS
WESTBANK, B.C.

Prepared for:
Buckland & Taylor Ltd.
101 - 788 Harbourside Drive
North Vancouver, BC V4P 3R7
Attention: Mr. Murray M. Johnson

Prepared by:
Levelton Consultants Ltd.
# 150 - 12791 Clarke Place
Richmond, British Columbia
V6V 2H9

Reviewed by:
J.P. Davidson, P.Eng.
R.S. Charlton, P.Eng.
24 April 2013
File: RI11-2552
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Figures 1 to 43
Appendix I
Appendix II
Appendix III
1.0 INTRODUCTION

The Materials and Corrosion Group at Levelton Consultants Ltd. were engaged by Buckland & Taylor Ltd. to undertake failure analysis of welded wire mesh reinforcement utilized for Mechanically Stabilized Earth (MSE) Walls for the Westside Road Underpass, Westbank, BC.

It is our understanding that:

- A portion of MSE Wall B below the west abutment of the Westside Road Underpass failed Sunday, November 20, 2011. The failed wall components came to rest adjacent to the southbound lanes of Hwy. 97.

- At the time of the Wall B failure, the overnight low temperature was -15.2°C, the coldest temperature reached since the bridge was constructed.

- The MSE Wall components and soil reinforcing mesh were furnished by SSL (Scotts Valley, California) as part of the MSE PLUS™ Wall System.

- The Prime Contractor for the construction was Ledcor.

- The Westbank First Nation, who are delivering the project for the Ministry of Transportation and Infrastructure MoTI Southern Interior region had engaged Golder Associates, CWMM Consultants, Urban Systems and EBA Engineering Consultants for engineering services pertaining to the underpass construction.

- The Ministry of Transportation and Infrastructure, Victoria have retained Buckland & Taylor to investigate the Westside Road Underpass Wall failure.

- The Westside Road Bridge over Hwy. 97 had been in operation for approximately three weeks at the time of the MSE Wall failure.

Levelton’s work to date has included:

- Periodic site reconnaissance on and after 23 November 2011.

- Laboratory analysis and testing of hot dip galvanized, welded wire mesh reinforcement including:
  - Visual examination
  - stereo macroscopic fractography
  - optical microscopy
  - tensile testing
  - chemical analysis (optical emission spectroscopy)
  - SEM/EDS (Scanning Electron Microscopy / Energy Dispersive Spectroscopy)
  - bend testing (wire straightening, including hammer tests)
  - hardness testing
• analytical testing of a backfill sample
• non-standard tensile testing of typical reinforcement panel connections
• longer term exposure testing of typical wire bends in backfill.

Our observations and opinions concerning the fractures of longitudinal earth reinforcement wire ends are the subject of this communication.

2.0 SUMMARY OF OPINIONS

Without exception, fractures of the reinforcing mat D20 (deformed wire with a nominal cross-section of 0.20 inch²) wires occurred at the 90° bend of the wire end utilized for the panel connection. The wire end fractures initiated at the inside diameter of the bend and progressed diametrically toward the outside diameter radius in a very brittle manner. Most fractures exhibited distinctly sloped surfaces on either side of an inflection line at or near the neutral axis of the wire. The upturned ends of the reinforcing mats failed due to a tensile tear developed in bending; the fractures appeared to "hinge" about the toe of the failure at the outside diameter surface of the wire.

We strongly associated the observed D20 wire failures with an embrittlement phenomena and severe cold work of the wire about a tight radius bend. The mechanism responsible for embrittlement and premature failure of the wire ends was strain age embrittlement. The failure of the D20 wires was brittle tensile tear which initiated on the inside diameter wire bend radius, often at prior strain age embrittlement cracks. Metallographic and fractographic examination of D20 wire bends indicated that, in some instances, cracking pre-existed hot dip galvanizing whereby the mouth of the crack was bridged by zinc/zinc-iron alloy.

In our opinion, the loading of the reinforcement-to-panel connection is complex and may have involved both vertical tensile stresses (developed by bearing of the cross wires on the panel loops), reverse bending (opening of the 90-degree wire bends) and slight horizontal tension developed by deflection (elevation mismatch) of the horizontal elements of the mat. Investigation of the panel connections following the failure revealed elevation mismatches of up to 0.25 m (10 inches) without any apparent bending of the horizontal wires. Based on the absence of reinforcement bending in the horizontal plane (tending to open the wire hook), it was apparent that at least some wires had failed during placement and compaction of the fill. The presence of black smut on portions of the D20 wire fractures was strongly associated with appreciable cracking of the wire cross-section and exposure of the fracture surface prior to installation.

To date we have only emulated fracture of a D20 wire bend samples in the laboratory using high strain rates (hammer strikes) at temperatures below -12°C. High strain rate testing achieved using a sharp hammer blow at reduced temperature was analogous to CSA G164 (R2003) “Hot Dip Galvanizing of Irregularly Shaped Articles” Clause 6.5.1.1 whereby fracture of the base metal is considered a failure with respect to avoidance of strain age embrittlement.

Straight portions of the D20 and D11 wires retain tensile properties that exceed ASTM A 496 requirements although tensile testing of the wire showed definite yield points and limited elongation. Material that exhibits a definite yield point prior to reaching ultimate tensile load is non-conforming with ASTM A 496-07 Clause 8.1.4, but is consistent with the tensile requirements of ASTM A 1064.
Chemical analysis of the wire samples from Westside Road indicated that the wire was low carbon (0.06-0.09 wt %) and was essentially devoid of typical grain refining constituents such as aluminum, titanium and niobium. Low carbon composition, the absence of grain-refining elements and fine grained structure are associated with susceptibility to strain age embrittlement in carbon steels that have been subject to a critical level of cold work.

3.0 OBSERVATIONS

Our observations and comments are summarized as follows:

3.1 GENERAL

- MSE Walls B and C were oriented parallel to the north and south bound lanes of Highway 97, respectively.
- Bridge abutments which support a Westside Road overpass bear on backfill retained by MSE Walls B and C.
- Hot dip galvanized welded wire mesh in four, five and six wire wide segments is utilized in the SSL MSE Plus™ Wall System (refer to Figure 1).
- D11 wires (deformed wire with a nominal cross-sectional area of 0.11 inch²) are utilized as transverse elements (cross wires) to the D20 longitudinal wires.
- D11 longitudinal wire mats were utilized in other MSE walls or portions of MSE walls on the project.
- The longitudinal wire ends are bent 90°, are abutted to the backside of precast concrete panels and are engaged by a round bar key inserted through loops protruding from the backside of the panel (see Figure 2).
- Samples retrieved from site are documented in Appendix I and consisted of:
  - Fractured D20 longitudinal wire ends (see Figure 3).
  - Fractured D20 longitudinal wire ends with round bar locking keys (see Figure 4).
  - End portions of reinforcing mesh mats prepared by transverse cuts of the longitudinal wires with features of interest (see Figure 5).
  - End segments of D11 reinforcing mesh mats.
  - The deformed wire had the manufacturer's identification "CRI" (Concrete Reinforcements Inc.) embossed on the wire surface (see Figure 6).
  - It was apparent that the mesh junctions had been resistance welded prior to hot dip galvanizing.
  - The bend radius of the D20 diameter wire appeared to be approximately 10 mm, consistent with bending around an approximately 20 mm diameter pin (see Figure 7).
• We estimated that the D11 diameter wire ends had been bent around a slighter larger diameter pin (approximately 22 mm) than the D20 wire (see Figure 8).

3.2 **VISUAL FRACHTOGRAPHIC EXAMINATION**

The fractured ends of the wires were examined stereomacroscopeically in the as-received condition. Numerical identifiers for the wire mesh segments were preserved as labeled. Individual wires were numbered consecutively from perceived north to south positions in Wall B. For example, “B&T 4-3” referred to Buckland & Taylor sample segment #4 (as labelled), wire # 3 (third wire from the perceived north end of sample segment). Five samples considered representative of the sample lot were sectioned from the vertical/horizontal wire ends available and were photo documented in the as-received conditions and following ultrasonic cleaning in warm Alconox solution and in ethanol (refer to Figures 9 to 19 and Appendix I). It was noted that:

• The fractured wire ends tended to show distinct, sloped regions which often intersected at a line of inflection near the neutral axis of the wire.

• Appreciable black deposits (suspected smut) existed on portions of the wire fracture surfaces. The black deposits were readily friable in Alconox-water solution.

• Orange corrosion products (iron hydroxide) were observed on portions of the brittle fractures adjacent to the inside diameter of the bend and superimposed on portions of the black deposit.

• The macroscopic appearance of the fractures following cleaning included extensive facets and radial ridges indicative of brittle fracture (cleavage).

• It was common to see a small, blunted toe on the fracture surface adjacent to the outside diameter radius of the bend indicative of rotation of the horizontal and vertical legs of the wire tending to straighten the bend.

3.3 **METALLOGRAPHIC EXAMINATION**

A segment of the fractured wire sample B&T 4-3 was sectioned longitudinally, polished to a 1 micron finish and etched with 2% nital for metallographic examination (see Figure 20). Metallographic examination revealed the following:

• The metallurgical structure of the wire was primarily ferrite with very little pearlite present (consistent with low carbon steel).

• To the extent determinate with optical microscopy, the fracture appeared extensively faceted and transgranular, consistent with a cleavage fracture.

• Minor secondary cracks parallel to and perpendicular to the fracture surface existed (see Figure 21 and 22).

• No appreciable zinc or zinc iron-alloy was detected in the crack at depth.
3.4 SEM/EDS EXAMINATION

Buckland & Taylor Samples 1-2 and 4-3 were examined in the scanning electron microscope using energy dispersive spectroscopy (SEM/EDS).

The EDS analysis spectra of the black deposit on the fracture surface of Sample 1-2 are shown in Figure 23. Relative elemental composition was as shown in Table 1. The small sulphur peak observed is considered independent of potential interference of lead and molybdenum peaks.

Table 1 – SEM Qualitative Elemental Analysis of Black Deposit

<table>
<thead>
<tr>
<th>Element</th>
<th>Concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon</td>
<td>10.61 wt %</td>
</tr>
<tr>
<td>Oxygen</td>
<td>44.39 wt %</td>
</tr>
<tr>
<td>Magnesium</td>
<td>0.42 wt %</td>
</tr>
<tr>
<td>Aluminum</td>
<td>1.80 wt %</td>
</tr>
<tr>
<td>Silicon</td>
<td>4.53 wt %</td>
</tr>
<tr>
<td>Sulfur</td>
<td>&lt;0.02 wt %</td>
</tr>
<tr>
<td>Potassium</td>
<td>0.24 wt %</td>
</tr>
<tr>
<td>Calcium</td>
<td>0.68 wt %</td>
</tr>
<tr>
<td>Titanium</td>
<td>0.03 wt %</td>
</tr>
<tr>
<td>Manganese</td>
<td>0.08 wt %</td>
</tr>
<tr>
<td>Iron</td>
<td>27.95 wt %</td>
</tr>
<tr>
<td>Copper</td>
<td>0.46 wt %</td>
</tr>
<tr>
<td>Zinc</td>
<td>8.83 wt %</td>
</tr>
</tbody>
</table>

A small radial crack adjacent to the toe of a deformation on the inside diameter of the Buckland & Taylor Sample 4-3 D20 wire bend was observed and investigated using area scans of the crack (see Figures 20 and 24).

It was noted that:

- Although zinc coating bridged the mouth the crack (indicating that the crack pre-existed the galvanizing dip) the relative concentration of detectable zinc decreased rapidly away from the diametrical surface of the wire. Zinc concentrations at Area 004 to Area 001 were 85%, 10.7%, 0.75%, 0% respectively.

- The crack tips appeared relatively blunt.

- The extent of the crack beyond the Area 002 scan location was not readily resolvable optically.
3.5 Bend Testing

Bend testing of intact wire ends was undertaken at room temperature, -7°C, -12°C and -22°C by reverse bending with a snipe placed over the wire end (slow strain rate) or by reverse bending achieved with a strike of a three pound hammer (high strain rate). By reverse bending, we refer to loading that would tend to straighten the wire, placing the inside diameter of the wire in tension. High strain rate testing achieved by hammer blow at reduced temperature is analogous to test criteria described in CSA G164 Clause 6.5.1.1, although we had no opportunity to test similar non-galvanized wire for comparison. Results are shown in Table 2. Individual D20 wires invariably fractured when subjected to high strain rate (hammer strike) at -22°C and in one of two tests conducted at -12°C. High strain rate tests of D20 wire at -7°C, low strain rate testing of D20 wires and all reverse bending of D11 wires failed to produce any fractures in the laboratory.

Table 2 – Bend Test Results

<table>
<thead>
<tr>
<th>MSE Wall</th>
<th>Wire Type</th>
<th>Identification</th>
<th>Strain Rate</th>
<th>Temp (°C)</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>D11</td>
<td>Item 14-Wire #2</td>
<td>Low</td>
<td>Room</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D11</td>
<td>Item 14-Wire #4</td>
<td>Low</td>
<td>Room</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D11</td>
<td>Item 14-Wire #5</td>
<td>Low</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D11</td>
<td>Item 14-Wire #6</td>
<td>High</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D20</td>
<td>Item 14-Wire #1</td>
<td>High</td>
<td>Room</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D20</td>
<td>Item 15-Wire #2</td>
<td>Low</td>
<td>Room</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D20</td>
<td>Item 15-Wire #4</td>
<td>Low</td>
<td>Room</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D20</td>
<td>Item 15-Wire #5</td>
<td>Low</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D20</td>
<td>Item 13-Wire #5</td>
<td>Low</td>
<td>Room</td>
<td>No fracture.</td>
</tr>
<tr>
<td>A</td>
<td>D20</td>
<td>Item 15-Wire #6</td>
<td>High</td>
<td>-22</td>
<td>Fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D11</td>
<td>Item 23-1 of 2-A</td>
<td>High</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D11</td>
<td>Item 23-1 of 2-B</td>
<td>High</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D11</td>
<td>Item 24-2 of 2-A</td>
<td>High</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D11</td>
<td>Item 24-2 of 2-B</td>
<td>High</td>
<td>-22</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D20</td>
<td>Item 25-1 of 4</td>
<td>High</td>
<td>-22</td>
<td>Fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D20</td>
<td>Item 26-2 of 4</td>
<td>High</td>
<td>-22</td>
<td>Fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D20</td>
<td>Item 27-3 of 4</td>
<td>High</td>
<td>-7</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D20</td>
<td>Item 28-4 of 4-A</td>
<td>High</td>
<td>-12</td>
<td>No fracture.</td>
</tr>
<tr>
<td>Stockpile</td>
<td>D20</td>
<td>Item 28-4 of 4-B</td>
<td>High</td>
<td>-12</td>
<td>Fracture.</td>
</tr>
</tbody>
</table>

Note: “Stockpile” referred to excess mesh from the construction of Walls A and B which was located between Westside Road and the Lecor site trailers, refer to Appendix I.

The reverse bend testing of individual D20 wires from the Wall A stockpile panel samples #1 of 4, #2 of 4 and #4 of 4 wire produced fractures very similar to those examined from Wall B (see Figures 25 to 27).
3.6 SEM Fractography

The similarity of the laboratory reverse bending fracture morphology to the fractured ends of longitudinal wires exposed by the failure of Walls B and deconstruction of Wall C motivated more detailed fractography due to the opportunity for examination of the wire break in the absence of corrosion. The Wall A D20-6 sample was selected as an exemplar. Fractographic examination showed that:

- The majority of the horizontal leg and vertical leg D20 wire fracture surfaces adjacent to the inside diameter of the bend appeared extremely brittle with features indicative of cleavage (see Figure 28).

- A very small region of the fracture adjacent to the outside diameter of the wire bend showed features consistent with deformation in compression (see Figure 29).

- Owing to the presence of primarily cleavage features and shallow, featureless rupture limited to the “fulcrum” (toe) of the break in bending, we would characterize the mode of failure as cleavage manifested as a tensile tear in bending.

3.7 Mechanical Test Results

3.7.1 Tensile Testing

Tensile testing of individual wires removed from the mesh segments was undertaken (refer to Appendix II, Reports 01, 04 and 10 and Table 3). To the extent possible, we attempted to exclude weld zones associated with the transverse wires from the gauge length tested. The test results showed that:

- D20 and D11 wires demonstrated yield and ultimate tensile strengths in excess of 480 MPa (70 ksi) and 550 MPa (80 ksi) respectively.

- All samples tested showed a definite yield and not discontinuous yielding as sought by the related product standard ASTM A496-97 “Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement”.

- The tensile test of full cross-section samples revealed modest total elongation in the order of 7 to 10% in 200 mm (8 inches). ASTM specifications for welded wire mesh do not stipulate elongation requirements.

- The tensile test of the Wall C wire was limited to a subsize sample machined from an upturned D20 wire end. Comparison of the subsize sample tensile properties with the full-size tests should be done with caution.
Table 3 – Tensile Testing

<table>
<thead>
<tr>
<th>Wall</th>
<th>Sample</th>
<th>Wire Size</th>
<th>Yield Stress (Upper Yield) ksi (MPa)</th>
<th>Ultimate Stress ksi (MPa)</th>
<th>Elongation %</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Item 6. B&amp;T #6 (unused panel, stockpile)</td>
<td>D20</td>
<td>82.8 (571)</td>
<td>89.7 (618)</td>
<td>7.0</td>
</tr>
<tr>
<td>B</td>
<td>Item 6. B&amp;T #6 (unused Item 10. panel, stockpile)</td>
<td>D20</td>
<td>83.1 (573)</td>
<td>89.4 (616)</td>
<td>10.1</td>
</tr>
<tr>
<td>B</td>
<td>Item 9. Middle Level (Col 3)</td>
<td>D20</td>
<td>82.9 (572)</td>
<td>89.3 (616)</td>
<td>9.6</td>
</tr>
<tr>
<td>B</td>
<td>Item 9. Bottom Level (Col 6)</td>
<td>D20</td>
<td>80.5 (555)</td>
<td>87.4 (603)</td>
<td>9.0</td>
</tr>
<tr>
<td>A</td>
<td>Item 15. Wall A - #1</td>
<td>D11</td>
<td>84.1 (580)</td>
<td>90.7 (625)</td>
<td>9.0</td>
</tr>
<tr>
<td>C*</td>
<td>Item 20.</td>
<td>D20</td>
<td>87.4 (603)</td>
<td>90.8 (626)</td>
<td>20.2</td>
</tr>
</tbody>
</table>

Specification Requirements:

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A 496</td>
<td>70.0 (480) min.</td>
<td>80.0 (550) min.</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>A 1064</td>
<td>75.0 (515) min.</td>
<td>85 (585) min.</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Notes:

1. Tensile properties of D20 wire (excluding Wall C) based on wire size nominal cross-section (0.200 in2, 129 mm2).
2. "Tensile testing of Wall C sample based on subsize 1/4 inch diameter round machined from upturned wire end.
3. ASTM A 496 precludes yield strength by half-of-beam criteria.

The standard tensile testing described in Table 3 was limited to straight, hot dip galvanized wire segments. Non-standard tensile testing undertaken to investigate the behaviour of the panel connection is described in Section 3.9.

3.7.2 HARDNESS TESTING

Rockwell hardness testing of D11 and D20 wire cross-sections was undertaken (refer to Appendix II Reports 07 and 08). Typical core hardnesses of intact D20 and D11 wire bends ranged in the order of 52.6 to 54.8 HRA and 56.4 to 57.8 HRA, respectively. Subsequent hardness testing of a fractured D20 wire sample cross-section (Wall C, Column 18, Panel IX) suggested that the embrittled wire appeared similar, 53.4 to 55.5 HRA on average (see Report 08). Microhardness testing of the fractured Wall C section suggested a slight but measurable increase in hardness of the wire in the vicinity of the bend (and wire fracture in an embrittled bend) in comparison with the bulk of the wire, although the bulk wire hardness determined by microhardness testing was greater than the bulk hardness suggested by Rockwell testing (refer to Report 11).
3.7.3  **CHARPY IMPACT TESTING**

Impact testing of a typical D20 longitudinal wire was undertaken at the three test temperatures (room temperature, 0ºC and -20ºC). Results are shown in Report 09.

The average impact energy of 75 J (55 ft-lbs) observed for room temperature testing was consistent with expectations for fine-grained carbon steel. While a reduction carbon steel impact toughness is anticipated at lower test temperatures, we noted that the impact toughness exceeded 27 J (20 ft-lbs) for samples tested at both 0ºC and -20ºC. This result suggested that toughness of non-embrittled D20 wire is preserved to at least temperatures of -20ºC.

3.8  **ANALYTICAL TESTING**

3.8.1  **CHEMICAL ANALYSIS**

Chemical analysis of representative D11 and D20 wires was completed using optical emission spectroscopy (OES) (refer to Appendix II, Reports 02 and 03). We compared the results with CSA 30.18 criteria for avoidance of strain age embrittlement in hot dip galvanized reinforcing steel. It should be appreciated that ASTM A82, ASTM A496 and ASTM A1064 do not stipulate composition requirements. We noted that:

- The chemical composition of the wires tested indicated that the material was low carbon ranging from 0.06 to 0.09 wt%.
- It appeared that negligible grain refining elements had been utilized to alloy the material.
- Trace lead (Pb) was detected in D11 and D20 samples associated with Report 03.

3.8.2  **SOIL ANALYSIS**

A sample of Wall B backfill (Column W) was evaluated per AWWA C105 as to corrosivity to ferrous products. Results are shown in Appendix II. The analysis indicated that the sample was not considered corrosive to ductile iron, although conspicuously high levels of sulphate reducing bacteria (SRB) were indicated by an SDIX immunoassay (10^5 SRB).

3.9  **SERVICE LOADING**

Non-standard tensile testing of individual wires was undertaken to investigate the behaviour of the panel connection when loaded normal to the rear face of the panel and to investigate behaviour of the 90-degree bends of D20 wire when straightened. The testing was completed using a Tinius Olsen Super L60 universal tester and three methods as described below:

- Axial tensile testing of individual wires with the hook end engaged in a fixture intended to emulate the geometry of the panel connection (see Figures 30, 31 and 32).
- Axial tensile testing of a D20 wire hook end 90-degree bend as manufactured (see Figures 33 and 34).
- Axial tensile testing of a cold 90-degree bend induced in a typical straight segment of hot dip galvanized D20 wire (see Figures 35 and 36).
Results are summarized in Table 4.1 and Table 4.2.

### Table 4.1 – Non-standard Axial Tensile Testing of Simulated Panel Connection

<table>
<thead>
<tr>
<th>Wire Sample</th>
<th>Estimated Bend Angle (as fabricated)</th>
<th>Ultimate Load N (lb)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>D20 Wire 3, Subpanel 1 of 4</td>
<td>84.3°</td>
<td>42,463 (9,547)</td>
<td>Hook end contacted jig at approximately 10 kN. Failure due to shear of D11 bearing bar.</td>
</tr>
<tr>
<td>D20 Wire 3, Subpanel 2 of 4</td>
<td>85.4°</td>
<td>42,999 (9,867)</td>
<td>Hook end contacted at approximately 10 kN. Failure due to shear of D11 bearing bar.</td>
</tr>
<tr>
<td>D20 Wire 4, Subpanel 1 of 4</td>
<td>83.8°</td>
<td>41,001 (9,218)</td>
<td>Wire at reduced temperature (estimated -10 to -15°C) when tested. Hook end contacted jig at approximately 11 kN. Failure due to shear of D11 bearing bar.</td>
</tr>
</tbody>
</table>

The behaviour of individual wires tested using the panel connection fixture was very similar. It was noted that:

- The ultimate load in each test case was associated with shear of the transverse D11 bearing bar from the longitudinal wire (see Figure 37).

- Tensile loading of the individual wires tended to draw the hook end of the wire out of the clevis fixture.

- The testing tended to open the wire end hook angle approximately 10-12 degrees (see Figure 38). The resultant reverse bending of the wire hook ends did not result in any wire fractures. Slight radial cracking to a depth of approximately 1 mm (0.040 inch) was observed on the inside diameter surface of the Wire 4, Panel 1 of 4 sample following testing.

### Table 4.2 – Tensile Testing of 90-degree Bends

<table>
<thead>
<tr>
<th>Wire Sample</th>
<th>Estimated Bend Angle</th>
<th>Ultimate Load N (lb)</th>
<th>Comments</th>
<th>Figure Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>D20 Wire 6, Subpanel 2 of 4; as fabricated hook end</td>
<td>85°</td>
<td>21,748 (4,889)</td>
<td>90-degree SSL hook end bend essentially straightened prior to fracture. Hot dip galvanized after 90° bend completed.</td>
<td>39 and 40</td>
</tr>
<tr>
<td>Straight leg (longitudinal D20) Wire 6, Subpanel 2 of 4</td>
<td>90°</td>
<td>21,379 (4,806)</td>
<td>90-degree cold bend (LCL) essentially straightened prior to wire fracture. Hot dip galvanized prior to cold bending.</td>
<td>41 and 42</td>
</tr>
</tbody>
</table>

The average tensile strength of D20 wires tested was 89.3 ksi (615.6 MPa) (refer to Section 3.7.1). The ultimate load associated with the average tensile strength was 17,864 lb, based on a nominal 0.200 in² cross-sectional area for D20 wire. Comparison of the 90-degree bend tensile test results with the average tensile strength of similar D20 wires indicated that the as-fabricated 90-degree SSL bend segment and 90-degree cold bent (by Levelton) test segment
failed at 27.3% and 26.9% respectively of the typical ultimate load capacity of straight D20 wire, although both segments tested did straighten appreciably prior to fracture. The ability of the 90° “as-fabricated” and laboratory bends to straighten when the wire was loaded axially indicated the preservation of ductility in the particular wires tested.

3.10 EMULATED EXPOSURE TESTING

A fractured D20 wire and five additional D20 wire hook ends were collected from the sample lot available, prepared as described in Table 5, embedded in a sample of backfill retrieved from a stockpile on site, and left to weather in Richmond, B.C. for a period of 30 days (12 March to 12 April 2012) (refer to Figure 43 and Appendix III).

Table 5 – Exposure Emulation

<table>
<thead>
<tr>
<th>Sample Identification</th>
<th>Sample Origin</th>
<th>Initial Condition</th>
<th>Observations Following Exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1</td>
<td>High strain rate test at -12°C (from Not to be Used 4 of 4)</td>
<td>Fractured</td>
<td>Slight rust bloom (hydrated oxides of iron) visible. No black smut.</td>
</tr>
<tr>
<td>E2</td>
<td>High strain rate test at -12°C</td>
<td>Non-fractured.</td>
<td>Visibly unchanged.</td>
</tr>
<tr>
<td>E3</td>
<td>Not to be Used 4 of 4</td>
<td>Non-fractured, strained by flexing to open bend.</td>
<td>Visibly unchanged.</td>
</tr>
<tr>
<td>E4</td>
<td>Not to be Used 4 of 4</td>
<td>Non-fractured, bend ID notched with saw cut; strained.</td>
<td>Negligible corrosion visible at perimeter of notch.</td>
</tr>
<tr>
<td>E5</td>
<td>Not to be Used 4 of 4</td>
<td>Unstrained, bend ID notched with saw cut.</td>
<td>Negligible corrosion visible at perimeter of notch.</td>
</tr>
<tr>
<td>E6</td>
<td>Not to be Used 4 of 4</td>
<td>As fabricated.</td>
<td>Visibly unchanged.</td>
</tr>
</tbody>
</table>

Unlike the failed D20 wires observed in Walls B and C, the exposure Samples E1 to E6 emerged relatively free of corrosion/reaction products.

4.0 DISCUSSION

Strain age embrittlement refers to a loss in ductility accompanied by an increase in hardness and strength that occurs when low carbon steel is aged following plastic deformation. Aging refers to a change in material property manifested by exposure to a particular environment for some undefined interval of time. The change in properties is often, but not necessarily, due to a phase change but never involves a change in chemical composition. Generally, the degree of deformation or cold work is important. The resulting brittleness varies with aging temperature and time, occurring in a matter of minutes at elevated temperatures but requiring hours to a year or more at room temperature. In low carbon steels, strain aging is caused chiefly by the presence of interstitial solutes (carbon and nitrogen). Strain aging attributable to nitrogen is associated with the absence of strong nitride formers like aluminum, titanium, zirconium, vanadium or boron. In this regard, the risk of strain age embrittlement in low carbon steels is
significant for cold worked materials that are produced without grain-refining elements. The response may be most pronounced in fine-grained steels with low carbon content; as carbon content increases, susceptibility to strain aging is reduced. The common theme observed in the D20 wires sampled from the Westside Road MSE wall construction appears to be the fine-grained, low carbon composition of the wire and the severe cold work afforded by the right-angled bend of the wire about, what appears to have been, a 20 mm diameter pin.

It should be appreciated that any lot of material similarly processed will demonstrate varied or a distribution of strain aged embrittlement responses. The distribution of cracking in D20 wire bends may, in part, be explained by variation in the sample lot response to strain aging and to variations in processing due to bending and/or hot dip galvanizing of the reinforcement in batches, as well as the extent to which the wire ends may have been strained following embrittlement. The variability attributable to batch processing presents a challenge to identifying the presence of embrittlement in stockpiled product due to the fact that the processing history of the reinforcement cannot be assured to be consistent.

We strongly associated the development of black smut on portions of the wire fractures with cracking and exposure to acid at the time of acid pickling prior to hot dip galvanizing. It is considered possible, although less likely, that the black smut may be a manifestation of crevice corrosion attributable to the exposure. Recall that the black deposit was characterized as sulphur-bearing, moreover, testing of a backfill material sample suggested the presence of appreciable sulphate reducing bacteria (SRB). Better characterization of the response of wire fractures and/or crevices resulting from cracks in the wire bends would be dependent on longer term exposure testing in backfill material and ambient conditions consistent with the site. It should be appreciated that the sample of backfill utilized for the 30-day exposure test was unlike the Wall B Column W backfill material sampled 4 December 2011 in that it was aerated and comparatively devoid of detectable sulphate reducing bacteria.

It should be understood that material specifications for welded wire reinforcement do not stipulate chemical composition nor impact toughness requirements. Our laboratory impact testing of typical D20 longitudinal wires showed that the reinforcement has appreciable toughness at room temperature and toughness exceeding 27 J (20 ft-lbs) at 0ºC and -20ºC. Due to the geometry of the wire ends, we had no opportunity to undertake charpy impact testing of the 90-degree bend itself, although the hammer tests of the same strongly suggested very low impact toughness of strain aged embrittled wire. Notch sensitivity of embrittled wire is considered significant as a result.

5.0 GLOSSARY

Aging A change in material properties of certain metals and alloys that occurs at ambient or moderately elevated temperatures after hot working or heat treatment (quench aging in ferrous alloys, natural or artificial aging in ferrous and non-ferrous alloys) or after a cold-working operation (strain aging). The change in properties is often, but not always, due to a phase change (precipitation) but never involves a change in chemical composition.

Alconox A concentrated, powdered anionic (negatively charged) detergent utilized for manual and ultrasonic cleaning.
Cleavage Fracture  A fracture, usually of a polycrystalline metal in which most of the grains have failed by cleavage, resulting in bright reflecting facets. Cleavage is associated with low-energy brittle fracture and is in contrast with shear fracture.

Discontinuous Yielding  The non-uniform plastic flow of a metal exhibiting a yield point in which plastic deformation is inhomogeneously distributed along the gauge length.

Embrittlement  The severe loss of ductility or toughness or both of a material.

Energy Dispersive Spectroscopy (EDS)  An analytical, spectroscopic technique which measures the intensity of discrete X-rays emitted from an excited atom as an indication of the concentration of an element present in the sample examined. EDS is commonly associated with scanning electron microscopy.

Faceted  Flat faces on surfaces indicative of underlying crystal structure.

Fractography  Descriptive treatment of material fracture with specific reference to photographs of the fracture surface. Macro-fractography involves photographs at low magnifications (less than 25 X).

Grain Refining Elements  A material added to a molten metal to induce a finer-than-normal grain size in the final structure.

Interstitial Solutes  Elements of small atomic size that tend to occupy positions or spaces (interstices) between metal atoms in the unit cell (lattice).

Iron Hydroxide  Compound produced when iron (II) ions react with hydroxide (OH\(^-\)) ions. Also known as Iron (II) hydroxide or ferrous hydroxide.

Liquid Metal Embrittlement (LME)  Catastrophic brittle fracture of a normally ductile metal when in contact with a liquid metal and subsequently stressed in tension.

Morphology  The characteristic shape, form or surface texture or contours of the crystals, grains or particles of (or in) a material, generally on a microscopic scale.

Nital  Common etchant, 2-5% nitric acid in methanol or ethanol.

Nitride Formers  Alloying elements with a strong affinity to form compounds with nitrogen and act as strengthening agents.

Notch Sensitivity  The extent to which the sensitivity of a material to fracture is increased by the presence of a stress concentration, a sudden change in cross-section, a crack or a scratch. Low notch sensitivity is usually associated with ductile materials and high notch sensitivity is usually associated with brittle materials.

Optical Emission Spectroscopy (OES)  An analytical, spectroscopic technique which measures the intensity of discrete wavelengths emitted from an excited atom as an indication of the concentration of an element present in the sample examined.
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smut</td>
<td>A reaction product sometimes left on the surface of a metal after pickling, electroplating or etching.</td>
</tr>
<tr>
<td>Solid Metal Induced Embrittlement (SMIE)</td>
<td>Embrittlement of a constituent in a metallic couple manifested at temperatures below the melting temperature of the embrittled species (refer to iron-lead, iron-indium).</td>
</tr>
<tr>
<td>Strain Age Embrittlement</td>
<td>A loss in ductility accompanied by an increase in hardness and strength that occurs when low carbon steel (especially rimmed or capped steel) is aged following plastic deformation. The degree of embrittlement is a function of time and temperature.</td>
</tr>
<tr>
<td>Sulphate Reducing Bacteria (SRB)</td>
<td>Anaerobic bacteria that reduce sulphate ($\text{SO}_4^{2-}$) to a form of sulphide ($\text{S}^{2-}$).</td>
</tr>
<tr>
<td>Tensile Tear</td>
<td>Fracture mode typified by elongated dimples that point in the same direction on mating fracture surfaces (distinguishable from shear dimples which point in opposite directions on mating fracture surfaces).</td>
</tr>
<tr>
<td>Toughness</td>
<td>Ability of material to absorb energy and deform plastically before fracturing.</td>
</tr>
<tr>
<td>Transgranular</td>
<td>Through or across crystals or grains.</td>
</tr>
</tbody>
</table>
**Figure 1:** General view of typical six-wire galvanized steel soil reinforcing mats identified as excess material.

**Figure 2:** Profile of typical reinforcement connection to Wall B below bridge abutment at the time of remediation. Note the fracture of the reinforcement wires at the panel connection and the relative displacement of the horizontal wires below the elevation of the connection.
Figure 3: General view of Buckland & Taylor Sample 2 retrieved from a pickup box following the failure. Fractures of the longitudinal wires at the upturned, 90° bend are visible.

Figure 4: Typical fractured reinforcing mat remnant with round bar key for SSL panel connection.
Figure 5: Oblique view of soil reinforcing mat and segment retrieved from controlled stockpile excluded from wall construction.

Figure 6: Close up of D20 wire showing "CRI" identification marking.
Figure 7: Side view of typical D20 wire 90-degree bend with 20 mm diameter comparator placed to estimate bend diameter at approximately 20 mm.

Figure 8: Side view of typical D11 wire 90-degree bend with 22 mm diameter comparator to estimate bend diameter at approximately 22 mm.
Figure 9: Fracture surface of Buckland & Taylor Sample 1-2, as received.
**Figure 10:** Fracture surface of Buckland & Taylor Sample 1-5, as received.

**Figure 11:** As in figure 10 following ultrasonic cleaning.
Figure 12: Fracture surface of Buckland & Taylor Sample 2-3, as received.

Figure 13: As in figure 12 following ultrasonic cleaning.
Figure 14: Fracture surface of Buckland & Taylor Sample 3-2, as received.

Figure 15: As in Figure 14 following ultrasonic cleaning.
Figure 16: Fracture surface of MoTI Sample 4, as received.

Figure 17: As in Figure 16 following ultrasonic cleaning.
Figure 18: Fracture surface of MoTI Sample 5, as received. Fracture characteristics suggest brittle overload, likely due to prior failure of adjacent wires.

Figure 19: As in Figure 18 following ultrasonic cleaning.
Figure 20: Composite optical metallograph of Buckland & Taylor Sample 4-3 longitudinal cross-section. Upper portion of composite image was adjacent to inside diameter radius of 90° bend. Secondary crack visible near middle of composite image is shown in Figure 21. Arrow indicates cracking at toe of deformation shown in Figure 24. Nital etch. Approximately 20X magnification.
Figure 21: Optical metallograph of secondary cracking approximately perpendicular to Buckland & Taylor Sample 4-3 fracture. Nital etch. Approximately 250X magnification.

Figure 22: Optical metallograph of secondary cracking approximately parallel to Buckland & Taylor Sample 4-3 fracture. Nital etch. Approximately 650X magnification.
Figure 23: SEM/EDS spectra for analysis of black deposit observed on Buckland & Taylor Sample 1-2 fracture surface.

Figure 24: Composite SEM image of secondary crack observed on Buckland & Taylor Sample 4-3 near inside diameter bend radius of D20 wire.
Figure 25: General view of Unused Wall A Position 6 D20 wire fractured by hammer strike at -22°C. Horizontal notch in wire end denotes horizontal leg of reinforcement.
Figure 26: Close-up view of Unused Wall A D20 wire fracture, vertical leg.

Figure 27: Close-up view of Unused Wall A D20 wire fracture, horizontal leg.
**Figure 28:** SEM image of facetted D20 wire fracture consistent with cleavage.

**Figure 29:** SEM image of fracture at “toe” showing shallow deformation developed in compression about fulcrum of tensile tear in bending.
**Figure 30:** General view of D20 wire sample utilized for tensile testing in the jig (fixture) constructed to emulate the panel connection.

**Figure 31:** General view of D20 wire sample engaged in panel connection test fixture. Note the contact of the bearing bar D11 wire on the test fixture clevis (see arrow).
**Figure 32:** Oblique view of single D20 wire in panel connection test fixture with bearing bar contacting clevis and nominal 5/8-inch (16 mm) diameter key inserted through holes in clevis ears.

**Figure 33:** General view of as-fabricated, D20 SSL 90-degree bend with 45-degree bends of longitudinal wire (left-hand side) and hook end (right-hand side) to produce an axial tensile test coupon.
Figure 34: General view of wire segment shown in Figure 33 gripped in crossheads of tensile test machine.

Figure 35: General view of cold-formed, 90-degree bend in former straight segment of longitudinal D20 wire with 45-degree bends of wire ends to facilitate an axial tensile test. 90-degree bend was formed using a nominal 20 mm diameter plunger.
Figure 36: General view of wire segment shown in Figure 35 gripped in crossheads of tensile test machine.

Figure 37: General view of typical D20 wire hook and D11 bearing bar showing shear of longitudinal wire to bearing bar weldment.
Figure 38: Elevation view of D20 wire hook end following tensile testing in panel connection fixture to show extent of reverse bending achieved by test in comparison to typical reinforcing mat as manufactured.

Figure 39: Close-up view of fractured, as-manufactured 90-degree SSL bend segment loaded to failure in tension.
**Figure 40:** Close-up view of as-manufactured 90-degree bend axial tensile fracture.

**Figure 41:** Close-up view of fractured wire segment from axial tensile test of cold-formed 90-degree bend of hot dip galvanized D20 wire segment.
Figure 42: Close-up view of fractured wire segment from axial tensile test of cold-formed 90-degree bend.

Figure 43: General arrangement of Samples E1 to E6 utilized for exposure test in backfill.
<table>
<thead>
<tr>
<th>Item No.</th>
<th>Sample Description</th>
<th>Wall</th>
<th>Date Retrieved</th>
<th>Image Reference</th>
<th>Tensile Test</th>
<th>Reverse Bend</th>
<th>Wires Remaining</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B&amp;T #1 P/U truck at bridge. Fractured D20 hook ends.</td>
<td>B</td>
<td>22 Nov 2011</td>
<td>Item No. 1 – B&amp;T #1, Figures A-1 through A-7</td>
<td>Not a candidate.</td>
<td>Not a candidate.</td>
<td>Hook ends only.</td>
</tr>
<tr>
<td>6</td>
<td>B&amp;T #6 Cut from unused panel with failed hooks with ID tag (at Urban site office). Single, longitudinal wire fractured end.</td>
<td>-</td>
<td>23 Nov 2011</td>
<td>Item No. 6 – B&amp;T #6, Figure A-1</td>
<td>Yes</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>X5 Failed section. Top of panel from Wall B in Louie’s Pit.</td>
<td>B</td>
<td>24 Nov 2011</td>
<td>Item No. 7 – X5, Figures A-1 through A-7</td>
<td>Not a candidate.</td>
<td>Not a candidate.</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>97 X6/3/11 Top of panel in failed area of Wall B, Louie’s Pit.</td>
<td>B</td>
<td>24 Nov 2011</td>
<td>Item No. 8 – X6, Figures A-1 through A-7</td>
<td>Not a candidate.</td>
<td>Not a candidate.</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>Bottom Level, Column 6, South Bar Intact Wall B, 4 ft down from top. Single longitudinal wire; fractured at hook end.</td>
<td>B</td>
<td>24 Nov 2011</td>
<td>Item No. 9 – Wall B, Column 6, Bottom Level, South Bar, Figure A-1</td>
<td>Yes</td>
<td>Not a candidate.</td>
<td>0</td>
</tr>
<tr>
<td>Item No.</td>
<td>Sample Description</td>
<td>Wall</td>
<td>Date Retrieved</td>
<td>Image Reference</td>
<td>Tensile Test</td>
<td>Reverse Bend</td>
<td>Wires Remaining</td>
</tr>
<tr>
<td>---------</td>
<td>-------------------------------------------------------------------------------------</td>
<td>------</td>
<td>----------------</td>
<td>--------------------------------------------------------------------------------</td>
<td>--------------</td>
<td>------------------------</td>
<td>----------------</td>
</tr>
<tr>
<td>10</td>
<td>Middle level, Column 3 (south end), North Bar Intact Wall B, 2 ft down from top.</td>
<td>B</td>
<td>24 Nov 2011</td>
<td>Item No. 10 – Wall B, Column 3, Middle Level, North Bar, Figure A-1</td>
<td>Yes</td>
<td>Not a candidate.</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>Top Level, Column 10, South Bar Intact Wall B, 1 ft down from top long bar.</td>
<td>B</td>
<td>24 Nov 2011</td>
<td>Item No. 11 – Wall B, Column 10, Top Level, South Bar, Figure A-1</td>
<td>No</td>
<td>Not a candidate.</td>
<td>1 (longitudinal only)</td>
</tr>
<tr>
<td>13</td>
<td>Ledor rejected cage supply onsite from MOTI site office area.</td>
<td>-</td>
<td>23 Nov 2011</td>
<td>Item No. 13 – Ledor Rejected Cage, Figure A-1</td>
<td>No</td>
<td>Wire #5 - low strain at RT</td>
<td>1</td>
</tr>
<tr>
<td>14</td>
<td>Not used – Wall A “6D20x11x5.5x20” 20 of 20, 2310, DMG’ From site office location.</td>
<td>A</td>
<td>28 Nov 2011</td>
<td>N/A</td>
<td>No</td>
<td>Wire #2 and #4 – low strain at RT</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wire #5 – low strain at -22°C</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wire #6 – high strain at -22°C</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wire #1 – high strain at RT</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Not used – Wall A “6D11x11x1.0x16’ 5 of 5, 2310, DMG” From site office location.</td>
<td>A</td>
<td>28 Nov 2011</td>
<td>N/A</td>
<td>Yes</td>
<td>Wire #2 and #4 – low strain at RT</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wire #5 – low strain at -22°C</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wire #6 – high strain at -22°C</td>
<td></td>
</tr>
<tr>
<td>Item No.</td>
<td>Sample Description</td>
<td>Wall</td>
<td>Date Retrieved</td>
<td>Image Reference</td>
<td>Tensile Test</td>
<td>Reverse Bend</td>
<td>Wires Remaining</td>
</tr>
<tr>
<td>---------</td>
<td>------------------------------------------------------------------------------------</td>
<td>------</td>
<td>----------------</td>
<td>--------------------------------------------------------------------------------</td>
<td>---------------</td>
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<td>-----------------</td>
</tr>
<tr>
<td>20</td>
<td>Wall C, Column 17, Panel T4, Top Grid Fractured D20 hook ends.</td>
<td>C</td>
<td>15 Dec 2011</td>
<td>Item No. 20 – Wall C, Column 17, Panel T4, Top Grid, Figures A-1 through A-7</td>
<td>N/A</td>
<td>Not a candidate.</td>
<td>Hook ends only.</td>
</tr>
<tr>
<td>23</td>
<td>&quot;NOT TO BE USED&quot; From onsite stockpile with bottom of pile tagged &quot;4W11x11x1.0&quot;x20&quot; 4</td>
<td>-</td>
<td>10 Jan 2012</td>
<td>N/A</td>
<td>No</td>
<td>2 wires – high strain at -22°C</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>OF 4 2310 DMG*. #1 of 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>&quot;NOT TO BE USED&quot; From onsite stockpile with bottom of pile tagged &quot;4W11x11x1.0&quot;x20&quot; 4</td>
<td>-</td>
<td>10 Jan 2012</td>
<td>N/A</td>
<td>No</td>
<td>2 wires – high strain at -22°C</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>OF 4 2310 DMG*. #2 of 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Item No.</td>
<td>Sample Description</td>
<td>Wall</td>
<td>Date Retrieved</td>
<td>Image Reference</td>
<td>Tensile Test</td>
<td>Reverse Bend</td>
<td>Wires Remaining</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------------------</td>
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<td>------------------------------------------------------------------------------</td>
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<td>----------------</td>
</tr>
<tr>
<td>25</td>
<td>&quot;NOT TO BE USED&quot; From onsite stockpile with bottom of pile tagged &quot;6W20x11x.5\times20' 20 OF 20 2310 DMG&quot;. #1 of 4</td>
<td>-</td>
<td>10 Jan 2012</td>
<td>N/A</td>
<td>Wire #3, #4 in panel connection jig; remainder of one longitudinal wire</td>
<td>1 wire – high strain at -22°C</td>
<td>3</td>
</tr>
<tr>
<td>26</td>
<td>&quot;NOT TO BE USED&quot; From onsite stockpile with bottom of pile tagged &quot;6W20x11x.5\times20' 20 OF 20 2310 DMG&quot;. #2 of 4</td>
<td>-</td>
<td>10 Jan 2012</td>
<td>N/A</td>
<td>Wire #3 in panel connection jig; Wire #6 in tensile tests of 90-degree bends</td>
<td>1 wire – high strain at -22°C</td>
<td>4</td>
</tr>
<tr>
<td>27</td>
<td>&quot;NOT TO BE USED&quot; From onsite stockpile with bottom of pile tagged &quot;6W20x11x.5\times20' 20 OF 20 2310 DMG&quot;. #3 of 4</td>
<td>-</td>
<td>10 Jan 2012</td>
<td>N/A</td>
<td>No</td>
<td>1 wire – high strain at -7°C</td>
<td>5</td>
</tr>
<tr>
<td>28</td>
<td>&quot;NOT TO BE USED&quot; From onsite stockpile with bottom of pile tagged &quot;6W20x11x.5\times20' 20 OF 20 2310 DMG&quot;. #4 of 4</td>
<td>-</td>
<td>10 Jan 2012</td>
<td>N/A</td>
<td>No</td>
<td>2 wires – high strain at -12°C</td>
<td>0 – remaining; Wires 1-6 used for exposure test</td>
</tr>
</tbody>
</table>
Item No. 1 – B&T #1

Figure A-1: General view of B&T #1; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
**Figure A-5**: Close-up view of the fracture surface from Wire 4.

**Figure A-6**: Close-up view of the fracture surface from Wire 5.

**Figure A-7**: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of B&T #2; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Item No. 3 – B&T #3

**Figure A-1:** General view of B&T #3; individual wires labelled in ascending order from left to right.

**Figure A-2:** Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

**Figure A-3:** Close-up view of the fracture surface from Wire 2.

**Figure A-4:** Close-up view of the fracture surface from Wire 3.
Item No. 3 – B&T #3

**Figure A-5:** Close-up view of the fracture surface from Wire 4.

**Figure A-6:** Close-up view of the fracture surface from Wire 5.

**Figure A-7:** Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of B&T #4; individual wires labelled in ascending order from left to right. Note: the fracture of Wire 3 was cross-sectioned before documentation.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 4 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of B&T #5; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of B&T #6.
Figure A-1: General view of X5; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Item No. 7 – X5

**Figure A-5:** Close-up view of the fracture surface from Wire 4.

**Figure A-6:** Close-up view of the fracture surface from Wire 5.

**Figure A-7:** Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of X6; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of Wall B, Column 6, Bottom Level, South Bar.
Figure A-1: General view of Wall B, Column 3, Middle Level, North Bar.
Item No. 11 – Wall B, Column 10, Top Level, South Bar

Figure A-1: General view of Wall B, Column 10, Top Level, South Bar.
Item No. 12 – MOTI

Figure A-1: General view of MOTI; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Item No. 13 – Ledcor Rejected Cage

Figure A-1: General view of Ledcor Rejected Cage.
Figure A-1: General view of Wall B, Column 7, Row 6; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 5 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of Wall B, Column 9, Row 6; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of Wall B, Column 11, Row 6; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Item No. 19 – Wall C, Column 8, Panel T5, Upper Grid

Figure A-1: General view of Wall C, Column 8, Panel T5, Upper Grid; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Profile of Wire 4 fracture shown in Figure A-5. Note the “beak-like” appearance of the fracture.

Figure A-7: Close-up view of the fracture surface from Wire 5.

Figure A-8: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Item No. 19 – Wall C, Column 8, Panel T5, Upper Grid
Figure A-1: General view of Wall C, Column 17, Panel T4, Top Grid; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).

Figure A-3: Close-up view of the fracture surface from Wire 2.

Figure A-4: Close-up view of the fracture surface from Wire 3.
Figure A-5: Close-up view of the fracture surface from Wire 4.

Figure A-6: Close-up view of the fracture surface from Wire 5.

Figure A-7: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
Figure A-1: General view of Wall C, Column 18, Panel 1X, Bottom Grid; individual wires labelled in ascending order from left to right.

Figure A-2: Close-up view of the fracture surface from Wire 1 (leftmost fracture shown in Figure A-1).
Figure A-1: General view of Wall C, Column 18, Panel 1X, Top Grid; individual wires labelled in ascending order from left to right.

Figure A-2: Side view of Wire 1 (leftmost wire shown missing in Figure A-1) section displaying a crack at the inside of the bend.

Figure A-3: Close-up view of the fracture surface from Wire 5.

Figure A-4: Close-up view of the fracture surface from Wire 6 (rightmost fracture shown in Figure A-1).
APPENDIX II
MATERIAL TEST CERTIFICATE  
TENSILE TEST

CLIENT: BC Ministry of Transportation and Infrastructure  
Southern Interior Region  
231 – 447 Columbia Street  
Kamloops, BC V2C 2T3

REPORT DATE: 28 November 2011  
TEST DATE: 25 November 2011  
PROJECT NO.: RI11-0610-17.10

P.O. NO.:  
JOB NO.:  
REPORT NO.: 01

Attention: Mr. Jim E. Tait, AScT.

TEST REPRESENTING: Longitudinal HDG reinforcing elements from MSE Wall B – Deformed Wire.

MATERIAL SPECIFICATION: ASTM A 496

| Sample                  | Area \(\text{in}^2\) | Yield Load (Upper Yield) \(\text{lbs}\) | Yield Stress (Upper Yield) \(\text{ksi}\) | Ultimate Stress \(\text{ksi}\) | Elongation (in 8") \(|\%|\) | Reduction of Area \(|\%|\) |
|-------------------------|----------------------|----------------------------------------|----------------------------------|------------------|----------------|-----------------|
| B&T #6 (unused)         | 0.200                | 16,560                                 | 82.8                             | 89.7             | 7.0            |                  |
| B&T #6 (unused)         | 0.200                | 16,621                                 | 83.1                             | 89.4             | 10.1           |                  |
| Middle Level (Column 3) | 0.200                | 16,576                                 | 82.9                             | 89.3             | 9.6            |                  |
| Bottom Level (Column 6) | 0.200                | 16,101                                 | 80.5                             | 87.4             | 9.0            |                  |
| Specification Requirements: ASTM A 496, Table 4 |  | 70.0 (485 MPa) \(\text{min.}\) | 80.0 (550 MPa) \(\text{min.}\) |                  | -              |                  |

Test Method: ASTM A 370  
Sample Size: Full Section

Witnessed by:

Remarks: 1. Tensile properties calculated based on D-20 deformed wire size number \((0.200 \text{ in}^2, 129 \text{ mm}^2)\).
2. Test specimens exhibited defined yield point – nonconformance with respect to A 496-07 Clause 8.1.4.
3. ASTM A 496 includes no requirement for elongation.

LEVELTON CONSULTANTS LTD.
Per:

Patrick P. Ho, EIT

Reviewed by: John P. Davidson, P.Eng.

- Specimens are retained for 60 days unless otherwise notified.
- Baldwin BTE-120 and Tinius Olsen Super "L-60" test machines are verified annually to the requirements of ASTM E 4.
METALLURGICAL TEST REPORT
CHEMICAL COMPOSITION

CLIENT: BC Ministry of Transportation and Infrastructure
Southern Interior Region
231 – 447 Columbia Street
Kamloops, BC V2C 2T3

Attention: Mr. Jim E. Tait, AScT

REPORT DATE: 28 November 2011
TEST DATE: 25 November 2011
PROJECT NO.: RI11-0610-17.10
P.O. NO.: 
JOB NO.: 
REPORT NO.: 02

TESTS REPRESENTING: Longitudinal HDG reinforcing elements from MSE Wall B.

MATERIAL SPECIFICATION: -

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<tr>
<th>Sample</th>
<th>C</th>
<th>Mn</th>
<th>Si</th>
<th>P</th>
<th>S</th>
<th>Ni</th>
<th>Cr</th>
<th>Mo</th>
<th>Cu</th>
<th>Al</th>
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</thead>
<tbody>
<tr>
<td>Nominal 15.9 mm diameter X5 Plain</td>
<td>0.067</td>
<td>0.43</td>
<td>0.15</td>
<td>0.01</td>
<td>0.05</td>
<td>0.10</td>
<td>0.09</td>
<td>0.02</td>
<td>0.26</td>
<td>0.002</td>
</tr>
<tr>
<td>B&amp;T #6</td>
<td>0.058</td>
<td>0.38</td>
<td>0.14</td>
<td>0.01</td>
<td>0.04</td>
<td>0.13</td>
<td>0.13</td>
<td>0.04</td>
<td>0.30</td>
<td>0.002</td>
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<td>Specification Comparison</td>
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<td>-</td>
</tr>
<tr>
<td>CSA G30.18-09 Preferred for HDG Grade 300W, 500W</td>
<td>&lt;0.25</td>
<td>&lt;1.35</td>
<td>&lt;0.04 &lt;0.15-0.25</td>
<td>&lt;0.04</td>
<td>0.053 max.-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Remarks:

LEVELTON CONSULTANTS LTD.
Per:

Patrick P. Ho, EIT

Reviewed by: John P. Davidson, P.Eng.

• Specimens are retained for 60 days unless otherwise notified.

LEVELTON CONSULTANTS LTD.
#150 – 12791 Clarke Place
Richmond, B.C., V6V 2H9

TEL: (604) 278-1411
FAX: (604) 278-1042
# METALLURGICAL TEST REPORT

## CHEMICAL COMPOSITION

### CLIENT:
Buckland & Taylor  
101-788 Habourside Drive  
North Vancouver, BC  V7P 3R7

### REPORT DATE:
21 December 2011

### TEST DATE:
9 December 2011

### PROJECT NO.:
RI11-2552

### P.O. NO.:

### JOB NO.:

### REPORT NO.:
03

### Attention:
Mr. Murray M. Johnson, P.Eng

### TESTS REPRESENTING:
Longitudinal HDG reinforcing wire mesh from site supply (not used during construction).

### MATERIAL SPECIFICATION:
ASTM A1064/A1064M – 10

### CHEMICAL COMPOSITION - Wt (%)

<table>
<thead>
<tr>
<th>Sample</th>
<th>C</th>
<th>Mn</th>
<th>Si</th>
<th>P</th>
<th>S</th>
<th>Ni</th>
<th>Cr</th>
<th>Mo</th>
<th>Cu</th>
<th>Al</th>
<th>Nb</th>
<th>Ti</th>
<th>Pb</th>
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<tbody>
<tr>
<td>D11 bar (not used from Wall A)</td>
<td>0.08</td>
<td>0.40</td>
<td>0.122</td>
<td>0.011</td>
<td>0.033</td>
<td>0.117</td>
<td>0.112</td>
<td>0.027</td>
<td>0.272</td>
<td>0.002</td>
<td>0.002</td>
<td>0.001</td>
<td>0.004</td>
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<tr>
<td>D20 bar (not used from Wall A)</td>
<td>0.09</td>
<td>0.46</td>
<td>0.172</td>
<td>0.009</td>
<td>0.032</td>
<td>0.109</td>
<td>0.078</td>
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<td>0.000</td>
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<td>D20 bar (not used from Wall B)</td>
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<td>0.39</td>
<td>0.147</td>
<td>0.011</td>
<td>0.034</td>
<td>0.154</td>
<td>0.134</td>
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<td></td>
</tr>
<tr>
<td>CSA G30.18-09 Preferred for HDG Grade 300W, 500W</td>
<td>&lt;0.25</td>
<td>&lt;1.35</td>
<td>&lt;0.04 or 0.15-0.25</td>
<td>&lt;0.04 \text{max.}</td>
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</tr>
</tbody>
</table>

### Remarks:

LEVELTON CONSULTANTS LTD.  
Per: Patrick P. Ho, EIT

Reviewed by: John P. Davidson, P.Eng.

- Specimens are retained for 60 days unless otherwise notified.
MATERIAL TEST CERTIFICATE
TENSILE TEST

CLIENT: Buckland & Taylor
101-788 Habourside Drive
North Vancouver, BC V7P 3R7

REPORT DATE: 21 December 2011
TEST DATE: 19 December 2011
PROJECT NO.: RI11-2552
P.O. NO.: 
JOB NO.: 
REPORT NO.: 04

Attention: Mr. Murray M. Johnson, P.Eng

TEST REPRESENTING: D11 longitudinal HDG reinforcing element from a wire mesh sample from MSE Wall A site supply (not used during construction).

MATERIAL SPECIFICATION: ASTM A1064/A1064M – 10

<table>
<thead>
<tr>
<th>Sample</th>
<th>Area (in²)</th>
<th>Yield Load (Upper Yield) lbs</th>
<th>Yield Stress (Upper Yield) ksi</th>
<th>Ultimate Stress ksi</th>
<th>Elongation (in 8”) %</th>
<th>Reduction of Area %</th>
</tr>
</thead>
<tbody>
<tr>
<td>D11 Wall A - #1</td>
<td>0.110</td>
<td>9,251</td>
<td>84.1</td>
<td>90.7</td>
<td>9.0</td>
<td></td>
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</table>

Specification Requirements:  
A1064/A1064M – 10, Table 10

Test Method: ASTM A 370  
Sample Size: Full Section

Witnessed by:

Remarks:  
1. Tensile properties calculated based on D11 deformed wire size number (0.110 in², 71.0 mm²).
2. ASTM A1064/A1064M – 10 includes no requirement for elongation.

LEVELTON CONSULTANTS LTD.
Per:

Patrick P. Ho, EIT

Reviewed by: John P. Davidson, P.Eng.

- Specimens are retained for 60 days unless otherwise notified.
- Baldwin BTE-120 and Tinius Olsen Super 'L-.60' test machines are verified annually to the requirements of ASTM E 4.
MATERIAL TEST REPORT
HARDNESS RESULTS

CLIENT: Buckland & Taylor
101-788 Habourside Drive
North Vancouver, BC V7P 3R7

REPORT DATE: 6 February 2012
TEST DATE: 24 January 2012
PROJECT NO.: RI11-2552
P.O. NO.: 
JOB NO.: 
REPORT NO.: 07

Attention: Mr. Murray M. Johnson, P.Eng

TEST REPRESENTING: Unused longitudinal HDG elements from site supply stockpile of welded, deformed wire mesh.

MATERIAL SPEC.: ASTM A 497

TEST METHOD: ASTM E 18 (Clark Hardness Tester)

SETTINGS: Scale: Rockwell A (HRA)  Penetrator: Diamond
Load: 60 kgf  Duration: 3 seconds

<table>
<thead>
<tr>
<th>Sample</th>
<th>Hardness Readings (HRA)</th>
<th>Average Reading (HRA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal Leg</td>
<td>Bend</td>
</tr>
<tr>
<td>D11 ID Edge</td>
<td>54.5, 55.0, 54.5, 57.0, 54.5, 56.0, 57.0, 57.5, 57.0, 58.0, 57.0, 58.0, 56.0, 57.5, 56.5, 57.5, 55.0</td>
<td>56.4</td>
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<td>D11 Centre</td>
<td>56.5, 56.5, 56.5, 56.5, 56.5, 56.5, 57.5, 57.5, 57.5, 57.5, 57.5, 57.5, 57.0, 57.0, 57.0</td>
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<tr>
<td>D11 OD Edge</td>
<td>56.5, 56.5, 57.0, 57.5, 57.0, 57.5, 57.5, 58.0, 60.0, 59.0, 58.5, 59.0, 58.0, 58.5, 57.5, 57.5</td>
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<td>D20 ID Edge</td>
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<tr>
<td>D20 Centre</td>
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<td>52.6</td>
</tr>
</tbody>
</table>

Witnessed by:

Remarks: Sample image.

LEVELTON CONSULTANTS LTD.
Per:

Patrick P. Ho, EIT

Reviewed by: Jim Coulman, AScT

LEVELTON CONSULTANTS LTD.
#150 – 12791 Clarke Place
Richmond, B.C., V6V 2H9

TEL: (604) 278-1411
FAX: (604) 278-1042

- Specimens are retained for 60 days unless otherwise notified.
- Clark Hardness Test Machine is verified annually to the requirements of ASTM E 18.
MATERIAL TEST REPORT
HARDNESS RESULTS

CLIENT: Buckland & Taylor
101-788 Habourseide Drive
North Vancouver, BC V7P 3R7

REPORT DATE: 6 February 2012
TEST DATE: 26 January 2012
PROJECT NO.: RI11-2552
P.O. NO.: 
JOB NO.: 
REPORT NO.: 08

Attention: Mr. Murray M. Johnson, P.Eng

TEST REPRESENTING: Longitudinal D20 HDG element from MSE Wall C, Column 18, Panel IX, top grid with crack at ID of bend.

MATERIAL SPEC.: ASTM A 497

TEST METHOD: ASTM E 18 (Clark Hardness Tester)

SETTINGS: Scale: Rockwell A (HRA)
Load: 60 kgf
Penetrator: Diamond
Duration: 3 seconds

<table>
<thead>
<tr>
<th>Sample</th>
<th>Horizontal Leg</th>
<th>Hardness Readings (HRA)</th>
<th>Bend</th>
<th>Vertical Leg</th>
<th>Average Reading (HRA)</th>
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<tbody>
<tr>
<td>ID</td>
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<td>Ccntrc</td>
<td>55.0, 54.5, 54.0, 54.5, 54.0, 54.0, 54.0, 53.5, 52.5, 52.5, 53.0, 53.0, 53.5, 54.0, 54.5, 55.0, 55.5, 55.5</td>
<td>54.0</td>
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<tr>
<td>OD</td>
<td>55.0, 55.0, 54.0, 55.0, 55.0, 55.0, 54.0, 53.5, 49.0, 50.0, 52.0, 51.5, 53.5, 53.0, 54.0, 53.5, 53.5</td>
<td>53.4</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

Witnessed by:

Remarks: Sample image.

LEVELTON CONSULTANTS LTD.
Per:

Patrick P. Ho, EIT

Reviewed by: Jim Coulman, AScT

- Specimens are retained for 60 days unless otherwise notified.
- Clark Hardness Test Machine is verified annually to the requirements of ASTM E 18.
**MATERIAL TEST CERTIFICATE**  
**CHARPY IMPACT TESTING**

**CLIENT:** Buckland & Taylor  
101-788 Habourside Drive  
North Vancouver, BC V7P 3R7

**REPORT DATE:** 15 February 2012  
**TEST DATE:** 14 February 2012  
**PROJECT NO.** RI11-2552  
**P.O. NO.**  
**JOB NO.**  
**REPORT NO.** 09

**Attention:** Mr. Murray M. Johnson, P.Eng

**SAMPLE DESCRIPTION:** Longitudinal D20 HDG element from Westside Road Overpass site stockpile.

**MATERIAL SPECIFICATION:** ASTM A 496

**SPECIMEN TYPE:** ASTM A 370, V-Notch  
**SIZE:** 5 mm x 10 mm

**TEST ENERGY:** 220 ft-lb, 298 J  
**TEST TEMPERATURE:** (see below)

**MATERIAL REQUIREMENTS:**  
- Minimum Energy:  
- Single Specimen:  

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Impact Energies (ft-lb)</th>
<th>Average Energy (ft-lb)</th>
<th>Shear Fracture (%)</th>
<th>Lateral Expansion (mils)</th>
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</thead>
<tbody>
<tr>
<td>-20°C</td>
<td>13, 12, 9</td>
<td>26, 24, 18</td>
<td>23</td>
<td>31</td>
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<td>0°C</td>
<td>13, 10, 13</td>
<td>26, 20, 26</td>
<td>24</td>
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<td>23°C (room temperature)</td>
<td>26, 31, 26</td>
<td>52, 62, 52</td>
<td>55</td>
<td>75</td>
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Witnessed by:

**Remarks:**

LEVELTON CONSULTANTS LTD.  
Per:  

Patrick P. Ho, EIT  
Reviewed by: John P. Davidson, P.Eng.

- 1 ft-lb = 1.356 J  
- Specimens are retained for 60 days unless otherwise notified.
**MATERIAL TEST CERTIFICATE**

**TENSILE TEST**

**CLIENT:** Buckland & Taylor  
101 - 788 Harbourside Drive  
North Vancouver, B.C.  
V7P 3R7

**REPORT DATE:** 16 February 2012  
**TEST DATE:** 16 February 2012  
**PROJECT NO.:** RI11-2552  
**P.O. NO.:**  
**JOB NO.:**  
**REPORT NO.:** 10A

**Attention:** Mr. Murray Johnson, P.Eng.

**TEST REPRESENTING:** Vertical subsize HDG reinforcing element from Wall C, Column 17, Panel T4, Top Grid.

**MATERIAL SPECIFICATION:** ASTM A 497

<table>
<thead>
<tr>
<th>Sample</th>
<th>Area (in²)</th>
<th>Yield Load (Upper Yield) lbs</th>
<th>Yield Stress (Upper Yield) ksi</th>
<th>Ultimate Stress ksi</th>
<th>Elongation (in 1&quot;) %</th>
<th>Reduction of Area %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall C</td>
<td>0.0491</td>
<td>4,290</td>
<td>87.4</td>
<td>90.8</td>
<td>20.2</td>
<td>67.4</td>
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</tbody>
</table>

**Specification Requirements:** ASTM A 496, Table 4  
70.0 min. 80.0 min. - -

**Test Method:** ASTM A 370  
**Sample Size:** Standard 1/4" diameter round

**Witnessed by:**  

**Remarks:**

**LEVELTON CONSULTANTS LTD.**

Per:

Patrick P. Ho, EIT  
Reviewed by: John P. Davidson, P.Eng.

- Specimens are retained for 60 days unless otherwise notified.
- Baldwin BTE-120 and Tinius Olsen Super "L-60" test machines are verified annually to the requirements of ASTM E 4.

**LEVELTON CONSULTANTS LTD.**

#150 – 12791 Clarke Place  
Richmond, B.C., V6V 2H9  
TEL: (604) 278-1411  
FAX: (604) 278-1042
METALLURGICAL TEST REPORT
CHEMICAL COMPOSITION

CLIENT: Buckland & Taylor
101-788 Habourside Drive
North Vancouver, BC V7P 3R7

REPORT DATE: 21 February 2012
TEST DATE: 21 February 2012
PROJECT NO.: RI11-2552
P.O. NO.: 
JOB NO.: 
REPORT NO.: 10B

Attention: Mr. Murray M. Johnson, P.Eng

TESTS REPRESENTING: D20 vertical HDG reinforcing element from Wall C, Column 17, Panel T4, Top Grid.

MATERIAL SPECIFICATION: ASTM A1064/A1064M – 10

<table>
<thead>
<tr>
<th>Sample</th>
<th>C</th>
<th>Mn</th>
<th>Si</th>
<th>P</th>
<th>S</th>
<th>Ni</th>
<th>Cr</th>
<th>Mo</th>
<th>Cu</th>
<th>Al</th>
<th>Nb</th>
<th>Ti</th>
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<tr>
<td>Wall C</td>
<td>0.077</td>
<td>0.424</td>
<td>0.13</td>
<td>0.007</td>
<td>0.027</td>
<td>0.104</td>
<td>0.103</td>
<td>0.024</td>
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<td>0.004</td>
<td>0.004</td>
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<td>ASTM A 1064/</td>
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<td>Comparison:</td>
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<td></td>
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<tr>
<td>CSA G30.18-09</td>
<td>&lt;0.25</td>
<td>&lt;1.35</td>
<td>&lt;0.04</td>
<td>&lt;0.04</td>
<td>0.053 max.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>Preferred for HDG Grade 300W, 500W</td>
<td>&lt;0.15-0.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Remarks:

LEVELTON CONSULTANTS LTD.
Per:

Patrick P. Ho, EIT

Reviewed by: John P. Davidson, P.Eng.

• Specimens are retained for 60 days unless otherwise notified.

LEVELTON CONSULTANTS LTD.
#150 – 12791 Clarke Place
Richmond, B.C., V6V 2H9

TEL: (604) 278-1411
FAX: (604) 278-1042
MATERIAL TEST REPORT
MICROHARDNESS TESTING

CLIENT: Buckland & Taylor
101-788 Habourside Drive
North Vancouver, BC V7P 3R7

REPORT DATE: 22 February 2012
TEST DATE: 15 February 2012
PROJECT NO.: RI11-2552
P.O. NO.: 
JOB NO.: 
REPORT NO.: 12

Attention: Mr. Murray M. Johnson, P.Eng

TEST REPRESENTING: Wall C fractured D20.

MATERIAL SPEC.: ASTM A 496

TEST METHOD: ASTM E 18

SETTINGS: Scale: HVN Load: 0.5 kgf Duration: -

HARDNESS READINGS

<table>
<thead>
<tr>
<th>Location</th>
<th>Hardness Readings (HVN)</th>
<th>Average Reading (HVN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical OD, moving away from crack.</td>
<td>260, 264, 253, 249, 255, 254, 244</td>
<td>254 (62.0 HRA)</td>
</tr>
<tr>
<td>Horizontal OD, moving away from crack.</td>
<td>260, 265, 266, 258, 263, 266, 269</td>
<td>264 (62.6 HRA)</td>
</tr>
<tr>
<td>Vertical parent.</td>
<td>238, 248, 238, 218, 232, 217, 232</td>
<td>232 (60.5 HRA)</td>
</tr>
<tr>
<td>Horizontal parent.</td>
<td>242, 247, 230, 245, 238, 239, 252</td>
<td>242 (61 HRA)</td>
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<tr>
<td>Vertical ID, moving away from crack.</td>
<td>263, 271, 252, 261, 256, 249, 241</td>
<td>256 (~62.2 HRA)</td>
</tr>
<tr>
<td>Horizontal ID, moving away from crack.</td>
<td>258, 246, 256, 251, 247, 256, 257</td>
<td>253 (61.9 HRA)</td>
</tr>
</tbody>
</table>

Witnessed by:

Remarks:

LEVELTON CONSULTANTS LTD.
Per:

Reviewed by: John P. Davidson, P.Eng.

- Specimens are retained for 60 days unless otherwise notified.
- Clark Hardness Test Machine is verified annually to the requirements of ASTM E 18.
**Electrochemical Analysis of Soils for Corrosivity to Ductile Iron Pipe**

*(From ANSI/AWWA Standard C105, Appendix A)*

**Client:** Buckland & Taylor  
**Project No.:** RI11-2552

**Project:** Westbank MSE Wall – Soils Testing  
**Sample No.:** Wall B - Col W

**Location:** Wall B, Column 4, Upper Tier  
**Date Sampled:** 4 December 2011  
**Initials:** -

**Depth:** Unspecified  
**Date Lab. Tested:** 16 February 2012  
**Initials:** EM

### Soil Characteristics

<table>
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<tr>
<th>Soil Characteristic</th>
<th>Points</th>
<th>Actual Test Value</th>
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</thead>
<tbody>
<tr>
<td><strong>Temperature of Soil:</strong> 13.5°C</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Resistivity - Ω-cm</strong> (based on single probe at pipe depth or water-saturated Miller soil box)</td>
<td>Resistivity corrected.</td>
<td>Resistivity after temperature and meter accuracy correction = 276,267</td>
</tr>
<tr>
<td>&lt;1500</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>1500 - 1,800</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>1,800 – 2,100</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>2,100 – 2,500</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>2,500 – 3,000</td>
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<tr>
<td>&gt;3,000</td>
<td>0</td>
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<tr>
<td><strong>pH</strong></td>
<td>Average 7.47</td>
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<tr>
<td>0 - 2</td>
<td>5</td>
<td>1. 7.51</td>
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<td>2 - 4</td>
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<td>4 - 6.5</td>
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<td>3. 7.41</td>
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<tr>
<td>6.5 - 7.5</td>
<td>0**</td>
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<tr>
<td>7.5 - 8.5</td>
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</tr>
<tr>
<td>&gt;8.5</td>
<td>3</td>
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</tr>
<tr>
<td><strong>Redox Potential</strong></td>
<td>Average 174.93</td>
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</tr>
<tr>
<td>&gt; +100 mV</td>
<td>0</td>
<td>1. 179 mV</td>
</tr>
<tr>
<td>+50 to 100 mV</td>
<td>3.5</td>
<td>2. 176 mV</td>
</tr>
<tr>
<td>0 to 50 mV</td>
<td>4</td>
<td>3. 170 mV</td>
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<tr>
<td>Negative</td>
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</tr>
<tr>
<td><strong>Sulphides</strong></td>
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<td>3.5</td>
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<tr>
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</tr>
<tr>
<td>Trace</td>
<td>2</td>
<td>Positive</td>
</tr>
<tr>
<td>Negative</td>
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<td></td>
</tr>
<tr>
<td><strong>Moisture</strong></td>
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<td></td>
</tr>
<tr>
<td>Poor Drainage, continuously wet</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Fair Drainage, generally moist</td>
<td>1</td>
<td>Moist</td>
</tr>
<tr>
<td>Good Drainage, generally dry</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td><strong>Soil Type / Colour</strong></td>
<td></td>
<td>Grey sand and gravel</td>
</tr>
<tr>
<td><strong>Soil Textural Classification</strong></td>
<td>% Clay</td>
<td>% Silt</td>
</tr>
<tr>
<td></td>
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</tr>
</tbody>
</table>

**Total Score***: 4.5

* Total score ≥ 10.0 indicates the soil is aggressive to grey or ductile cast iron pipe.  
** If sulphides are present and low or negative redox potential results are obtained, three points shall be given for this range.
# SIEVE ANALYSIS REPORT

**To:** Buckland & Taylor Ltd.
101 - 788 Harbourside Dr.
North Vancouver, BC
V7P 3R7

**Attention:** Mr. Murray Johnson, P.Eng.

**Levelton Consultants Ltd.**

#150 - 12791 Clarke Place
Richmond, B.C.
Canada V6Y 2H0
Tel.: 604-278-1411
Fax.: 604-278-1042
E-mail: cmd@levelton.com

**File No:** R11-2552-00
**Report No:** 1
**Date issued:** February 17, 2012

**SAMPLING INFORMATION**

- **Source:** Wall Backfill from Column
- **Material:** 4" Minus
- **Specification:** N/A

<table>
<thead>
<tr>
<th>Material Specification</th>
<th>Sieve Analysis</th>
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<tr>
<td>Sieve Size</td>
<td>High Limit</td>
</tr>
<tr>
<td>90</td>
<td></td>
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<tr>
<td>75</td>
<td></td>
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<tr>
<td>50</td>
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<td>37.5</td>
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<td>25</td>
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<td>19</td>
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<td>12.5</td>
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<td>9.5</td>
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<td>4.75</td>
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<tr>
<td>0.15</td>
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<tr>
<td>0.075</td>
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</tr>
</tbody>
</table>

**REMARKS:** Sample tested to ASTM Standards C136 & C117.

---

**LEVELTON CONSULTANTS LTD.**

Per: [Signature]
Norman Chang, ASC/TE

Reviewed by: [Signature]
Arnold Palmer, ASC/TE

---

This report constitutes a testing service only. No engineering interpretation opinion is expressed or implied. Engineering review and interpretation can be provided on written request.
**MATERIAL TEST CERTIFICATE**

**TENSILE TEST**

**CLIENT:** Buckland & Taylor  
101 - 788 Harbourside Drive  
North Vancouver, B.C.  
V7P 3R7

**Attention:** Mr. Murray Johnson, P.Eng.

**REPORT DATE:** 20 April 2012  
**TEST DATE:** 27 March 2012  
**PROJECT NO.:** RI11-2552  
**P.O. NO.:**  
**JOB NO.:**  
**REPORT NO.:** 13

**TEST REPRESENTING:**  D20 longitudinal HDG element, Wire No. 3 from Stockpile No. 1 of 4.

**MATERIAL SPECIFICATION:**  ASTM A 497

<table>
<thead>
<tr>
<th>Sample</th>
<th>Area Nominal in²</th>
<th>Yield Load (Upper Yield) lbs</th>
<th>Yield Stress (Upper Yield) ksi</th>
<th>Ultimate Stress ksi</th>
<th>Elongation (in 8&quot;) %</th>
<th>Reduction of Area %</th>
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</thead>
<tbody>
<tr>
<td>Wire 3, No. 1 of 4</td>
<td>0.200</td>
<td>16,159</td>
<td>80.8</td>
<td>88.0</td>
<td>10.2</td>
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<td>Specification Requirements:  ASTM A 496, Table 4</td>
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<td>Test Method:  ASTM A 370</td>
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</tr>
</tbody>
</table>

Witnessed by:

Remarks:

**LEVELTON CONSULTANTS LTD.**

Per:

Patrick P. Ho, EIT  
Reviewed by:  John P. Davidson, P.Eng.

- Specimens are retained for 60 days unless otherwise notified.
- Baldwin BTE-120 and Tinius Olsen Super "L-60" test machines are verified annually to the requirements of ASTM E 4.
24 lip scale

Buckland & Taylor
KZ11-2552
Mar. 27/12

N-43
#1 H-4

Model: A874 A 497
Exposure Testing

Prior to Exposure | Following Exposure
--- | ---

E-1 Fracture wire

E-2 HSR tested, not fractured

E-3 Strained
Exposure Testing

Prior to Exposure | Following Exposure

E-4
Strained and notched

E-5
Not strained, notched

E-6
As fabricated
Appendix N
Naesgaard Geotechnical Report
Westside Road Interchange Project
MSE Wall Facing Failure, Kelowna, BC

Site Inspection Report
&
Results of Geotechnical Investigations

Prepared for:
Buckland & Taylor Ltd.
Suite 101 - 788 Harbourside Drive
North Vancouver, BC, Canada V7P 3R7

Attention: Mr. Murray Johnson, P.Eng.
Executive Engineer

By: Ali Amini, P.Eng.
Ernest Naesgaard, P.Eng.

April 23, 2012
1 INTRODUCTION

The construction of the Westside Road Interchange Project in Kelowna, BC (Figure 1) began in 2010 and the Westside Road Underpass No. 7558 opened to traffic on October 29, 2011. The interchange includes a two span (17.55m and 25.57m) underpass supported on a middle pier and two abutments which in turn are supported on Mechanically Stabilized Earth (MSE) walls (Figure 2). The MSE wall facing panels at the west bridge abutment (Wall B) collapsed on November 20, 2011 after being in service for about 3 weeks (Figure 3). However, the reinforced soil structure of Wall B supported the west bridge abutment, even though the facing panels were missing.

A similar Wall C supporting the east underpass abutment did not collapse but upon inspection was found to have broken connections between the soil reinforcing grids and the facing panels. No damage was observed for Wall A, which is to the north of Wall B, and does not support a bridge abutment.

MSE walls supplied to this project were MSE Plus™ system patented by SSL Construction Products (SSL). This system utilizes precast concrete facing panels and galvanized steel grid-type wire reinforcement. The panels are typically 1810 mm by 1505 mm and have a minimum of two rows of embedded loops connecting them to the soil reinforcing grids. The pre-bent ends of reinforcing grids slide between pairs of loops and connect to the facing panel by inserting two galvanized L-shaped pins into the loops on either end of the panels (Figure 4).

At the time of our site inspection (on December 14 and 15) the underpass was closed to traffic and repair works were in progress on Walls B and C.

Naesgaard Geotechnical Ltd. (NGL) is currently retained by Buckland & Taylor Ltd. (B&T) to provide expert geotechnical input to the forensic investigation initiated by the Ministry of Transportation and Infrastructure to find the causes for the MSE wall collapse.

This report includes a brief inspection report and a summary of geotechnical findings.

2 SITE INSPECTION REPORT


2.1 Observations at the Site Visit

Figure 5 shows the general conditions of the site at the time of the site visit. The temperature was in the range of 0 to -3°C with light snow on the evening of December 14th, 2011.

Wall A

A hole (about 0.2m diameter) had been dug (probably using vacuum excavation) from the top of the approach embankment behind Wall A down to the top of a connection between the wire grid and concrete panel. The longitudinal wire (normal to the wall facing) was in place and was unbroken. A close inspection was not possible due to the size and depth of the hole.
Wall C

The entire working area at Wall C was enclosed with tarps. Temporary supports were installed under the bridge girders near the abutment. The facing panels under the bridge abutment had not been removed except for a few panels below the two ends of the abutment footing. Shotcrete had been placed at these locations to provide support to the retained soil in lieu of the panels. Lock blocks were placed against the facing panels as a precaution (Figure 6).

Where panels had been removed, a number of exposed longitudinal deformed wires (D20, 12.8 mm diameter) were found broken at the sharp 90° bend at the panel connection. Some of the broken reinforcing wires had an elevation mismatch, up to 250 mm, between the wires protruding from the backfill and the embedded loops in the back faces of the panels (Figure 7). Some of the reinforcing grids sloped downward in some areas after shotcreting. The slope was measured in two cases and was 4 and 11 degrees (Figure 8).

Survey data gathered by Buckland & Taylor Ltd. indicates that the bridge abutment footing rotated towards the middle of the bridge and this likely resulted in the observed gap between the abutment and girder bearings (Figure 9). The top of abutment footing is lower than the top of wall coping whereas drawings show them at the same elevation (Figure 10). Our observation is consistent with the Buckland & Taylor Ltd. surveys that confirm that the structure had settled relative to the wall.

The main construction activity during the night shift was vacuum excavation of the backfill material behind three facing panels as highlighted in yellow on Figure 11. An air lance was used to assist movement of backfill to the vacuum inlet (Figure 12). The backfill material generally consisted of coarse grained angular rocks up to about 100 mm in size with some finer soils. The backfill immediately behind panels appeared to be loose. It was observed that in some cases the backfill did not fall through the grid openings and hung up on the grids, leaving voids or loose soil below the grids. This was observed at the edges of the zones where panels had been removed as part of inspection and repair work. The grid wires appeared to be bent under the weight of the hung up backfill (Figure 13). One pail of typical backfill material was taken for further examination.

During the vacuum excavation process unwanted sloughing of the backfill created a void about 1.5m deep by one meter high behind the existing remedial shotcrete. The unbroken wires behind panel X in Column 5 (5th column from left, Figure 11) were then cut using a grinder. It is understood that SSL had asked the crew not to cut the unbroken longitudinal wires behind Panel X in Column 6 and instead try to free up the panel by pulling out the L-shaped pin. An attempt was made to hammer out the pins connecting the wire grids to the panels but it was unsuccessful as pounding the grids aggravated the sloughing. The unbroken wires were eventually cut and the panels were pulled out using a crane (Figures 14 and 15). Vacuum excavation and removal of two panels took about 5 hours. All the longitudinal wire observed were D20 deformed wire (12.8 mm diameter). Figure 16 shows a grid behind the removed panel including two broken wires.
Wall B

The entire area was enclosed with tarps and heated. Temporary supports were installed under the bridge girders near the abutment. The majority of facing panels were removed except for a few at the base of the wall. Backfill material was covered with shotcrete with the original reinforcing grids protruding from the shotcrete (Figure 17). A significant number of longitudinal wires were broken at the sharp 90 degree bend where they were connected to the panels. All the broken wires appeared to be D20 deformed wire. There was no construction activity at Wall B area during the site visit.

3 AVAILABLE INFORMATION

The following documents were used in NGL’s work.

- CWMM drawings 7758-1 to 21 dated March 01, 2010.
- SSL drawings
  - RW-01 to 31, Issued for Approval, dated 02/09/11 for Wall A
  - RW-01 to 33, Issued for Approval, dated 11/04/10 for Wall B
  - RW-01 to 15, Issued for Approval, dated 09/17/10 for Wall C
- Golder’s response to RFI-001 on MSE wall allowable bearing, July 02, 2010.
- Ministry of Transportation and Infrastructure Geotechnical Report, Highway 97 Westside Road Interchange Road A and Spland Road, dated January 15, 2009.
- Ministry of Transportation and Infrastructure Test Hole Information, Proposed Westside Road Interchange, Kelowna, B.C. dated September 17, 2009.
- LEDCOR CMI Ltd. Letter, Highway No. 97 Westside Road Interchange, PN# 22402-0000 Re: Major Works Contract; Soil settlement and impacts on MSE Walls, dated December 20, 2011.
- Westbank First Nation and Ministry of Transportation and Infrastructure, Major Works Contract, Project N0. 22402-0000, Highway No. 97 Westside Road Interchange, Schedule 3, Special Provisions.
- Construction photos

3.1 MSE Wall Backfill Placement Procedure

Backfill placement and compaction procedures are indicated on the SSL design drawings. Based on these design drawings and our discussion with B&T, the backfill placement procedures were as summarized below and illustrated in Figure 18.

- Zone (1a) was placed in 250 mm thick lifts and compacted using a vibratory roller, either Caterpillar CS 323C or Bomag BW 213DH-3. 100 mm minus crushed rock was used for the MSE backfill. A gap, specified as 910 mm, but variable in size in the field (possibly up
to 1500 mm wide), is left between the recently placed backfill lift and the back face of the MSE facing panels.

- Reinforcing grids were placed on top of zone (1a) and connected to the facing panels. Site photos showed that in many cases the panel end of the grid was bent upwards in order to match the elevation of the panel connection loops.
- Zones (2a), (1b) and (2b) were placed. Zone (1b) below the reinforcing grid was filled by sifting fill through the grid mesh openings.
- Zone (2a) was compacted using the vibratory roller while zones (1b) and (2b) were compacted from the top of zone (2b) using a light tamper.
- Zone (3a) was placed in 250 mm thick lifts and compacted with the vibratory roller. A gap was left behind the facing panels. The width of the gap at the bottom and top of zone (3b) was specified as 610 mm and 910 mm, respectively. However, based on construction photos the size of the gap was larger than specified values in some cases.
- Zone (3b) was placed by sifting fill through the grid mesh openings.
- The procedure was repeated until the fill reached the top of the MSE wall. The procedure for placing and compacting a 300 mm thick layer of 19 mm crushed rock beneath abutment footings was not specified on SSL drawings.

The backfill construction methodology has resulted in the backfill within to the specified 910 mm zone immediately behind the MSE precast panels being poorly compacted. The SSL specifications furthermore state, that no compaction tests should be taken within 910 mm from the panels so the compaction level in this zone was not measured and recorded.

### 3.2 Field Observations

#### 3.2.1 Movements/settlements

Bridge component and MSE wall settlements and movements were inferred by Buckland & Taylor Ltd. based on surveys completed for them as compared to the design elevations. Based on the survey data, the inferred bridge settlements and other components of movement are:

- West abutment (on MSE Wall B)
  - 49 to 74 mm of vertical movement.
  - 0% and 0.4% of tilt towards the pier.

- East abutment (on MSE Wall C)
  - 48 to 66 mm of vertical movement.
  - 1.6 to 3.4% of tilt towards the pier.

- The Wall B leveling pad settled by 21 to 38 mm.

- Wall C coping settled by 13 to 17 mm.

- The vertical movement of the toes of the abutment spread footings relative to the copings was in the range of 18 to 50 mm.
• The pier footing settled by 10 to 12 mm.

3.2.2 Wire failure mapping

An elevation view of Wall B and C showing the mapped locations of the broken wire is included in Appendix L of the B&T report. In summary the majority of the wires in the upper 2 to 3 rows of the precast panels in Wall B and the upper 1 to 2 rows of the precast panels in Wall C were found broken at the connection. The broken wires are concentrated below the abutment footing. In the lower segments there were fewer broken wires. All the broken wires were D20 type.

4 GEOLOGY OF THE SITE

Between approximately 13,000 and 11,000 years ago during the retreat of the last major glaciations, an ice lobe created an ice dam across the Okanagan valley near Okanagan Falls. A large lake known as Glacial Lake Penticton formed behind the dam to elevations approximately 100m above the current lake level (Okanagan Geology, ed. Roed & Greenough, Kelowna Geology Committee, 1995). The sediments deposited in this lake form many of the silt terraces around the current Okanagan Lake and the soil underlying the Westside Road interchange site. These lacustrine and glacio-lacustrine soils are generally coarser with depth, grading from plastic silty clays, to silts, and silty sands, with the lower deposits possibly being of glacio-fluvial origin. The soils are typically over-consolidated and very stiff and/or very dense. The overconsolidation is attributed to erosion, aging affects, and desiccation. Pleistocene tills and/or bedrock underlie the sediments.

5 GEOTECHNICAL REVIEW AND ANALYSES

5.1 Site Characterization

The native soils underlying the site to depths of 26 m consist of over-consolidated clays, silty clays, and interlayered silts and fine sands, and sands with some silt and gravel to depths. The upper soils are definitely desiccated and not saturated. A Piezometer installed to 25 m below the ground surface by MoTI did not detect any groundwater in August. This supports the position that to the depths investigated the soils are not saturated. Table 1 summarizes a generalized soil profile for the Westside Road site derived from MoTI test holes TH08-14, 15, 16, 17, and 18, and Golder Associates BH G09-1 with depths ranging from 15.2 to 33.2 m. Figure 19 shows the location of the boreholes. The upper 1.5 to 2.0 m depth forms an active zone subject to freeze-thaw, wetting and desiccation. This zone may be fissured and variable in moisture content.

This section is based on the following information:

• Test hole logs and descriptions from Golder Associates Ltd report, Preliminary Geotechnical Assessment- Proposed Westside Road Interchange, Kelowna, BC, dated August 21, 2009.
5.2 MSE Wall Design Review

Internal and external stability of MSE Walls A, B and C at their tallest section were checked based on the information provided on SSL drawings. AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002 and MSEW Version 3 software developed by Adama Engineering Inc. were used for analysis and checking. Appendix 1 includes the AASHTO requirements and assumed loads and parameters.

5.2.1 External stability of the MSE walls

Calculated FoS (Factor of Safety) for external stability of MSE walls, A, B and C including global stability, sliding, overturning and bearing capacity met the AASHTO’s requirement for both static and seismic conditions (Table 2).

5.2.2 Internal stability of the MSE walls

Internal stability was checked for the following conditions:

- Uncorroded reinforcing grid. This is representative of the conditions at the completion of construction.
- Reinforcing grid with AASHTO 100 year corrosion allowance.
- Corroded reinforcing grid and 15% increase to the bridge dead load due to extra overlay thickness.

Figure 20 shows the calculated factor of safety (FoS) of longitudinal wire strength against yielding. For the uncorroded conditions, FoS values meet AASHTO’s requirement. The FoS values of some of grid layers were slightly less than that required by AASHTO for the corroded wires.

Figure 21 shows the calculated FoS on pull out resistance for longitudinal wires. For the uncorroded conditions, FoS values meet the AASHTO’s requirement. For corroded conditions, the FoS values of some of grid layers were slightly less than that required by AASHTO.

The methodology of calculation of internal horizontal stress indicated on SSL drawings (RW-04, Design Notes, Section 4.6) is different than the AASHTO’s method (Section 11.10.6.2.1). SSL uses a coefficient of lateral stress varying linearly from coefficient of at-rest pressure (Ko =1-sinφ) at the top of the wall to coefficient of active pressure, Ka=(1-sinφ)/(1+sinφ) at 6m depth below top of the wall. AASHTO specifies a coefficient of lateral stress varying linearly from 2.5 x Ka at the top of the wall to 1.2 x Ka at 6m depth below top of the wall for wire grids. SSL methodology should calculate lower lateral stress and hence lower longitudinal reinforcing steel.
5.3 **Settlement Calculations**

5.3.1 **Ground settlement under MSE Wall B**

Settlement of MSE Wall B at the mid-point of the leveling pad was calculated using the information obtained from test holes TH08-17, 18 and borehole BH G09-1. The weight of the MSE wall, road fill and bridge dead load were allowed for in the calculations. The fine grained soils were assumed to undergo immediate and consolidation settlements. Compressibility parameters (Compression index, Cc, rebound index, Cr and pre-consolidation pressure, Pc) of the fine grained soils were derived from the available oedometer tests performed on three Shelby Tube samples in BH G09-1 at depths 4.3, 8.9 and 13.3m. Figure 22 compares the Atterberg limits and moisture content of the samples used for oedometer tests and those obtained in test holes near Wall B. Figure 23 shows the simplified layering and soil parameters used for calculation of settlements. Consolidation settlement in the range of 100 mm was calculated for the mid-point of the Wall B leveling pad when using the compressibility parameters interpreted from the oedometer tests in BH G09-1. SPT blow counts indicate very stiff to hard clayey silty soils. Pre-consolidation pressure estimated from correlations to SPT blow counts gave higher values than those from the oedometer tests (Figure 23-e). Immediate settlement using elastic moduli derived from correlations with SPT N-values gave a settlement of 30 mm for the same location (mid-point of Wall B leveling pad). Most of the total settlements, including immediate and consolidation settlement is expected to occur during construction of the wall and bridge.

The above settlement may be approximately broken down as follows:

1. Settlement under MSE wall, Self-weight only: 65% of total settlements
2. Settlement under MSE wall, Self-weight + Bridge Dead Load: 80% of total settlement
3. Settlement under MSE wall Self-weight + Bridge Dead Load + Road fill: 100% of total settlement

It should be noted that the above calculation is based on the available site investigation information and lab test results. No attempt was made to adjust the soil parameters or calculation methodology so a match could be achieved with the observed total settlements.

5.4 **Numerical Modeling**

The interaction of the MSE wall with the bridge abutments, the bridge deck and foundation soil is complex. Numerical modeling was carried out to provide insight into this interaction, and insight into the patterns observed in the field. More specifically this included:

- The effect of the presence of a loose zone behind the facing panel.
- The elevation mismatch between the broken wires and their respective connections on facing panels.
- The pattern of distribution of broken wires in the MSE walls under the abutments.
- The effect of the settlement of foundation ground on reinforcing grids.
• The pattern of abutment movements (displacements and rotation).

The two dimensional finite difference program FLAC version 6.0 (ITASCA 2008) was used.

Figure 24 shows the geometry of the FLAC model. Figure 25 presents the various components of the model i.e. foundation ground, backfill, reinforcing wire grids, facing panels, abutment wall, abutment footing, deck acting as a strut and elastomeric bearings. Table 3 to 6 present the assumed parameters for soils and structural elements. The geometry of the wall, wire grids and abutment are generally based on Wall B.

The analyses were carried out in the total stress mode in a chronological manner similar to the construction sequence. The general procedure used for analyses included the following steps:

• Set up model grid for foundation soils.
• Construct the MSE wall including the MSE backfill, retained soil and the structural elements in approximately 1.5m high lifts.
• Repeat the above step until the full height of the MSE wall and retained soil is constructed.
• Add the abutment footing, abutment wall, deck strut and elastomeric bearing. Apply bridge dead load to the top of abutment wall (see Appendix 1 for bridge dead load on MSE Wall B Abutment).
• Add road fill (which is the backfill behind the abutment wall and the bridge deck).
• Apply bridge live load, bridge brake load (see Appendix 1 for assumed loads, on MSE Wall B Abutment).
• After each construction stage, update the soil stiffness as a function of the confining stresses.
• Compile and summarize results of analyses.

5.4.1 Effect of the loose zone behind the facing panels

To assess the effect of the presence of a loose zone behind the facing panels (will be called loose zone), multiple analyses were carried out on the two models shown on Figure 26. The results presented in this section are for the condition after construction of bridge deck and road fill, unless mentioned otherwise.

The stiffness of the foundation ground and dense backfill was selected such to give vertical displacements in the range of the observed settlements at the location of the leveling pad and abutment footing. The stiffness of the loose zone was initially set to 20 times softer than the dense backfill. However, the soil stiffness was assumed to be stress dependent (Table 3) and varied as construction progressed in the model. Figure 27 presents the distribution of shear modulus in the model after application of bridge dead load and road fill.

Figures 28 and 29 show the distribution of vertical and horizontal stresses, respectively. The vertical stresses in the loose zone are about one order of magnitude smaller than those in the
adjacent dense zone. The horizontal stresses in the loose zone and the lateral pressure on the facing panels were also somewhat reduced as illustrated on Figure 29 and 30.

Figure 31 shows the distribution of axial forces (all tensile) in the grids. Presence of the loose zone increased the maximum forces in the grids but slightly decreased the axial forces at the connections to facing panels (Figure 32).

Figure 33 presents the accumulated vertical displacements after placement of the bridge dead and the road fill. Larger vertical displacements occurred in the loose zone with the maximum at about mid-height of the MSE wall. Figure 34 shows the development of vertical displacements in the model at different stages of construction. Figure 35 shows the magnified deformed shape of the abutment and facing panels. The presence of loose zone resulted in more abutment settlement and greater outward tilt.

Figure 36 & 37 show the magnified deformed shape of the reinforcing grids. The grids generally followed the soil displacements which resulted in them draping down in the loose zone. If the connection is restricted from rotating, the slope of the deformed shape of the grids, at the connection, would be an indication of potential strains induced to the grids while the sagging of grids may explain the elevation mismatch between the connection on the panels and elevation of the ends of the broken grids (Figure 38a). The presence of the loose zone increased both the slope and elevation mismatch; especially in the upper 6 rows of grid and is generally consistent with the observed pattern of wire breakage. Figure 38b shows that construction of the bridge and road fill increased the calculated elevation mismatch and grid slopes at the connection. This increase was greater for the upper 4 layers of the grids.

The following items were not included in the current numerical models:

- The compaction induced settlement and lateral stresses.
- Flexural stiffness of the grids.

The sensitivity of the results to the element size also was not assessed.

5.4.2 Effect of foundation ground settlement under the MSE wall

The effect of settlement of the MSE wall foundation soil on the deformed shape of the reinforcing grids was examined (Figure 39). The analyses indicated that foundation settlement has minimal effect on relative grid deformation or rotation (Figure 40).

5.4.3 Effect of the stiffness of the loose zone on the displacement pattern of the abutment

Parametric analyses with varying stiffness of the loose zone were carried out. A rigid foundation was assumed. Decrease in stiffness of the loose zone increased the settlement and tilting of the abutment (Figure 41).

5.4.4 Effect of the width of the loose zone on the displacement pattern of the abutment

Similar analyses were performed where the width of the loose zone was varied from 0 to 3m. Increasing the width of the loose zone increased vertical displacement of the abutment. The
abutment tilted outward (towards the center pier) when the loose zone encroached under its footprint, otherwise the abutment tilted inward. The horizontal strutting action of the bridge deck is beneficial in the stability of the abutment. Numerical analyses without the strutting action of the deck resulted in greater vertical settlement and tilting of the abutment.

5.4.5 Conclusions from the numerical modeling

- The presence of the loose zone
  - decreased the vertical stresses in the soil behind the facing panels significantly,
  - slightly decreased the horizontal stresses on the facing panels and slightly decreased the tensile forces in the grids at the connection to facing panels,
  - increased the maximum tensile forces in the grids and
  - increased the vertical displacements behind the facing panels. This intensified the sagging (draping shape) of the grids and increased the induced rotation of the grids at the connection. The increase in sagging and slope of grids at connection were higher in the upper layers (consistent with the observed wire breakage pattern).

- Foundation settlement has minimal effect on relative grid deformation or rotation.

- The pattern of movement of the abutment obtained from the numerical modeling is generally consistent with that observed in the field.

- The rotation and settlement of the abutment depends on the ratio of the loose bearing area to the dense bearing area as well as the stiffness ratio of the loose to the dense zone.

- The horizontal strutting effect of the bridge deck is beneficial in reducing movement of the abutments.

6 DISCUSSIONS

At Wall B the reinforcing wire grid failed at the connections to the facing panels resulting in collapse of a number of panels and some spilling of backfill onto the road below. The reinforced soil mass beyond 1.0 to 1.5 meter from the facing panels appears to have maintained its integrity. At Wall C there was no collapse of the facing panels, however after deconstruction, a significant portion of the longitudinal wires were found to be broken at their panel connection bends. Based on discussions with B&T and Levelton Consulting, it is understood that the longitudinal D20 deformed wires are embrittled at their connection bends. We understand the embrittlement reduces the tolerance to strains induced by the construction process.

1. Wall Design

**External Stability:** The SSL MSE wall design was checked for external stability using two sets of foundation soil parameters, 1) parameters provided in the Golder 2009 report and, 2) simplified
soil model and parameters contained in the Contract Special Provisions. The MSE wall design meets the AASHTO 2002 requirement for external stability for both sets of parameters.

Internal Stability: The SSL MSE wall design was checked for internal stability. The factor of safety for wire strength was greater than the AASHTO requirement when in an uncorroded state. For the fully corroded condition, after the design life of 100 years, the factor of safety is slightly less than the AASHTO requirement. Pull out resistance of the grids meets the AASHTO requirements in an uncorroded state but is slightly less than the AASHTO requirement in fully corroded condition. In this simplified analysis the effect of the horizontal strutting action of the bridge deck on the abutment is not considered. Numerical analyses using FLAC showed that the addition of the strut to the model reduced the demands on the reinforcing grids.

One concern regarding the design of the MSE walls B and C (in particular Wall C) is that the underside of the abutment footing is close to the uppermost layer of reinforcing grid. The settlement of the abutment footing under load will strain the wire grid in proximity to the panel connections. Notwithstanding, the geotechnical design of the MSE wall is considered adequate and the MSE wall failure does not appear to be related to the geotechnical aspects of the MSE wall design.

2. MSE Wall Construction

Lack of adequate fill compaction in a zone behind the MSE wall panels and the use of a coarse backfill material that may not freely pass through the reinforcing grid openings created loose pockets of fill with possible voids in this area. This loose zone does not provide adequate bearing support for the reinforcing grids when they are loaded from above. Therefore, the vertical loads from compaction (even if light in nature), from the self-weight of upper layers of fill, and from loading of the abutment footings were applied directly to the poorly supported grids causing downward movement, rotation, and bending of the wires. Compaction directly behind the panels also may have exerted dynamic forces on the wire grids and their connection to the facing panels.

The following is an excerpt from the book “Evaluation of the SSL MSE Plus retaining wall system” by Highway Innovative Technology Evaluation Center in 1999. HITEC 1999 emphasized that it is important for MSE walls to compact the backfill below the grids and in particular beneath the grid connections to prevent additional bending forces in the grids. At this site this did not take place.

“As with all MSE wall systems, it is important that the construction specifications under “Backfill Placement” be strictly enforced. Specifically the following must be enforced.

At each reinforcement level, the backfill shall be placed to the level of the connection. Backfill placement methods near the facing shall assure that no void exist directly beneath the reinforcing elements.

Failure to strictly enforce this specification may introduce bending forces in the grid reinforcement and significantly increases the difficulty of maintaining the required overall verticality.”
FHWA-NHI-00-043 (Mechanically stabilized earth walls and reinforced soil slopes design and construction guidelines, March 2001) also emphasizes on the compaction of the area behind the facing panels to prevent “chimney-shaped vertical void immediately behind the facing elements”.

It is concluded that the MSE wall construction methodology contributed to the failure of the MSE wall by adding bending strains to the wire grids. However, we believe that the MSE wall would have tolerated the construction induced strains without failure if the grid wires were sound and not embrittled near the connection to the facing panels.

3. Field Response of the Structures

**Settlement of the leveling pad**: Settlement of Wall B leveling pad was inferred to be about 21 to 38 mm from field measurements. Calculated immediate (elastic) settlements were 30 mm whereas those calculated using consolidation theory and oedometer tests were 100 mm. The site soil strata therefore appear to be stiffer than what was inferred from the consolidation theory and oedometer test results.

**Settlement of the pier**: The calculated settlement of pier spread footing is about 3 to 4 times greater than the inferred field measurement in the range of 10 to 12 mm. Again, this implies that the site soil stratum is stiffer than what was inferred from the oedometer tests.

**Movements of West abutment (on MSE Wall B)**: Vertical movement in the range of 49 to 74 mm and tilting in the range of 0% to 0.4% towards the central pier were surveyed. FLAC analysis resulted in calculated vertical movements of typically about 50 mm and footing tilting of about 0.5% towards the pier. The effect of breakage of the wires was not considered in the numerical analyses. The calculated values are, however, in relatively good agreement with the site observations.

**East abutment (on MSE Wall C)**: Vertical movement in the range of 48 to 66 mm and tilting in the range of 1.6 to 3.4% towards the pier were inferred from site measurements. No calculations or modeling was performed for Wall C. Extrapolating from the MSE Wall B modeling, the inferred vertical movements are in the range of calculations but the measured tilt seems greater than calculated values. Some of the additional tilt could be due to smaller setback of the footing behind the facing panel and breakage of the grid wire connections which was not simulated in the FLAC model.

**The pattern of elevation mismatch between the broken wires and facing panels**: The observed elevation mismatch results from the deformations within the MSE walls, compaction loading and bridge loading. Also, construction photographs indicate that in some cases, the fill layers were placed lower than the elevation of panel connections. These issues result in draping of the grids where it spans between compacted backfill and the connections on the facing panel over the loose and possibly voided area. Upon breakage the ends of the broken longitudinal wires would deflect under the weight of the soil above it.

**Wire breakage pattern under the abutments**

Mapping of the breakage pattern showed that most of the broken wires were in the upper grid layers. Numerical analyses indicate that the deformation of the wires is concentrated in the
loose zone. The sagging (which is an indication of elevation mismatch) and slope of the draping wires at the connections (which may be an indication of induced bending strains as the results of construction and service loading) were greater in the upper layers. This is generally consistent with the observed wire breakage pattern.

Properly designed and constructed MSE walls are generally flexible structures and should be able to tolerate global site settlements and deformations in the range of 100 mm to 200 mm without failure. This has been demonstrated by good behavior of MSE walls during major earthquakes (Figure 43). Since surveyed elevations of key structural components and of the finished road works show settlements well below these values, we conclude that overall site settlement did not cause the wall to fail. Numerical modeling also indicated minimal effects of settlements (increased to about 4 times those observed in the field) on reinforcing grids.

7 SUMMARY AND CONCLUSIONS

Some possible geotechnical concerns in design and construction of the MSE walls at the Westside Road Interchange Project are identified. However, MSE walls are generally flexible and typically tolerate such concerns without failure. It is postulated that the primary cause of the MSE wall facing panel detachment and wire breakage was not geotechnical, but a structural detailing and metallurgical problem.

NAESGAARD GEOTECHNICAL LTD.

Reviewed by,

Ali Amini, Ph.D., P.Eng.

Ernest Naesgaard, Ph.D., P.Eng.
### Table 1 - Generalized soil profile

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>Zone Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 1.1</td>
<td>FILL</td>
<td>asphalt with underlying gravel</td>
</tr>
<tr>
<td>3.2 to 6.3</td>
<td>plastic CLAY</td>
<td>stiff to very stiff brown, moist, CLAY, SILT, &amp; clayey SILT, w=33 to 41%, Wp=24 to 27%, WL=52 to 69%, SPT N-value 13 to 18</td>
</tr>
<tr>
<td>8.8 to 10.5</td>
<td>CLAY/SILT</td>
<td>Firm to very stiff brown CLAY &amp; SILT, w=29 to 44%, Wp=21 to 22%, WL=38 to 46%, SPT N-value 7 to 72</td>
</tr>
<tr>
<td>0 to 15.8</td>
<td>SILT</td>
<td>Very dense brown moist sandy SILT, SPT N-value 21 to 65</td>
</tr>
<tr>
<td>7.3 to 13.0</td>
<td>silty SAND</td>
<td>Dense to very dense brown silty SAND and SAND, SPT N-value 30 to &gt;100</td>
</tr>
<tr>
<td>&gt;2.6</td>
<td>SAND</td>
<td>Very dense brown moist SAND, some silt, and trace gravel and cobbles, SPT N-value 72</td>
</tr>
<tr>
<td>&gt;1.9</td>
<td>SILT</td>
<td>Hard brown clayey SILT to sandy SILT, SPT N-value &gt; 100</td>
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</table>

### Table 2 - Calculated factors of safety for external stability

<table>
<thead>
<tr>
<th>MSE Wall</th>
<th>FoS Bearing Capacity</th>
<th>FoS Sliding</th>
<th>FoS Overturning</th>
<th>FoS Global stability</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Static</td>
<td>Seismic</td>
<td>Static</td>
<td>Seismic</td>
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<tr>
<td>A</td>
<td>3.7</td>
<td>2.9</td>
<td>2.8</td>
<td>1.6</td>
</tr>
<tr>
<td>B</td>
<td>3.5</td>
<td>3.2</td>
<td>2.5</td>
<td>1.7</td>
</tr>
<tr>
<td>C</td>
<td>2.7</td>
<td>2.5</td>
<td>2.7</td>
<td>1.8</td>
</tr>
<tr>
<td>Minimum Required by AASHTO 2002</td>
<td>2.5</td>
<td>1.9</td>
<td>1.5</td>
<td>1.1</td>
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</table>
Table 3 - Assumed soil parameters in FLAC analyses

<table>
<thead>
<tr>
<th>Soil Parameters</th>
<th>Dense Zone</th>
<th>Loose Zone</th>
<th>Road Fill</th>
<th>Retained Soil</th>
<th>Foundation Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kN/m³)</td>
<td>22</td>
<td>18</td>
<td>22</td>
<td>21</td>
<td>20</td>
</tr>
<tr>
<td>Peak Friction angle (deg) (Note 1)</td>
<td>38</td>
<td>30</td>
<td>38</td>
<td>36</td>
<td>-</td>
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<tr>
<td>Dilation angle (deg)</td>
<td>5</td>
<td>-3</td>
<td>5</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Poisson's ratio (-)</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
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<tr>
<td>Shear Modulus (kN/m³)</td>
<td>Note 2</td>
<td>Note 3</td>
<td>Note 1</td>
<td>Note 1</td>
<td>2.30E+04</td>
</tr>
<tr>
<td>Constitutive soil model</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
<td>Elastic</td>
</tr>
</tbody>
</table>

Note 1: The strength parameters are best estimate values and not the project design values.
Note 2: Shear modulus is assumed to be stress dependent using the following equation:
\[ G = K \cdot (\sigma'_m)^{0.5} \]
where  \( k \) is a correlation factor and  \( \sigma'_m \) is the mean normal stress.
Note 3: Shear modulus of loose zone was assumed as a ratio of the dense zone. See text.
Table 4 - Properties of precast concrete panels

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross area (m²/m)</td>
<td>0.15</td>
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<tr>
<td>Young's modulus, E (MPa)</td>
<td>2.5 x 10⁴</td>
</tr>
<tr>
<td>Moment of inertia, I (m²/m)</td>
<td>2.30 x 10⁻⁴</td>
</tr>
<tr>
<td>Modified area (m²/m) (see Note 1)</td>
<td>0.001</td>
</tr>
</tbody>
</table>

Notes:

1- Facing elements cross section area were reduced to account for the effect of rubber pads on the overall axial stiffness of the wall facing elements.

Table 5- Properties of abutment footing

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross area (m²/m)</td>
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<tr>
<td>Young's modulus, E (MPa)</td>
<td>2.5 x 10⁴</td>
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<tr>
<td>Moment of inertia, I (m²/m)</td>
<td>5.30 x 10⁻³</td>
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<tr>
<td>Density (kg/m³)</td>
<td>2400</td>
</tr>
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</table>
Table 6 - Assumed parameters for wire grids in FLAC analysis

<table>
<thead>
<tr>
<th>Strip Row # From Bottom</th>
<th>Height from Wall Base (m)</th>
<th>No. of wires per panel width=1.8m</th>
<th>Longitud. Wire Type</th>
<th>Transverse Wires</th>
<th>Minimum apparent friction coefficient ((-)) (corroded/uncorroded)</th>
<th>Initial apparent friction coefficient ((-)) (corroded/uncorroded)</th>
<th>Transition Confining Pressure (kPa)</th>
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</thead>
<tbody>
<tr>
<td>8</td>
<td>4.69</td>
<td>5</td>
<td>D20</td>
<td>0.50D11</td>
<td>0.493/0.625</td>
<td>0.986/1.249</td>
<td>120</td>
</tr>
<tr>
<td>7</td>
<td>4.44</td>
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<td>6</td>
<td>3.80</td>
<td>6</td>
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<td>5</td>
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<td>1.50D11</td>
<td>0.164/0.208</td>
<td>0.329/0.416</td>
<td>120</td>
</tr>
<tr>
<td>3</td>
<td>1.51</td>
<td>6</td>
<td>D20</td>
<td>1.50D11</td>
<td>0.164/0.208</td>
<td>0.329/0.416</td>
<td>120</td>
</tr>
<tr>
<td>2</td>
<td>0.75</td>
<td>5</td>
<td>D20</td>
<td>1.50D11</td>
<td>0.164/0.208</td>
<td>0.329/0.416</td>
<td>120</td>
</tr>
<tr>
<td>1</td>
<td>0.12</td>
<td>5</td>
<td>D20</td>
<td>1.50D11</td>
<td>0.164/0.208</td>
<td>0.329/0.416</td>
<td>120</td>
</tr>
</tbody>
</table>

Notes:
- Modulus of elasticity, $E=2.1\times10^5$ MPa.
- Diameter of uncorroded longitudinal wire=12.8mm
- Diameter of corroded longitudinal wire=10.78mm
- Yield Strength, $f_y=517$ MPa
- Transition confining stress is the pressure beyond which apparent friction coefficient remains at its minimum value.

All the FLAC analyses presented in the report used uncorroded section properties.
Figure 1- Site location
Figure 2- General arrangement of Westside Rd. Overpass
Figure 3- Wall B after collapse
Figure 4- concrete panel and reinforcing grid connection typical detail
Figure 5- Westside Road bridge over HWY 97 - Looking South
Figure 6- Wall C- lock blocks placed against panels as a safety measure
Top: Front View
Bottom: Side View
Figure 7— Note the broken longitudinal wire (deformed 0.5” wire) and their mismatch elevation with the embedded connection loops.
Figure 8- Wall C, some of reinforcing grid sloped downward
Formation of a gap possibly due to rotation of the abutment

Figure 9- Wall C, the abutment seems to have rotated outward (towards the pier) causing a gap between the bearings and top of the abutment.
Figure 10- Wall C, the top of abutment footing was lower than the top of wall whereas the structural drawing at the top shows they should be at the same elevation.
Figure 11- Wall C, work plan for December 14, 2011 night shift was to remove yellow highlighted panels.
Figure 12- Wall C, vacuum excavation.

Figure 13- Wall C- Note backfill material arched above the grid opening can bend down the grid wires.
Figure 14- Wall C, a panel is being pulled out.

Figure 15- Wall C, after removal of the two panels in Column 5 and 6
Figure 16- Wall C, some of longitudinal wire were broken (note the rust) and some were cut.

Figure 17- Wall B, most of the concrete panels had been removed. Some backfill behind the panels were removed and covered with shotcrete.
Figure 18: Inferred Backfill placement procedure (Modified from SSL DWG. ERECTION SEQUENCE AND DETAILS, The background figure is taken from SSL DWG, - the colored areas are added)
Figure 19 - Borehole location
Figure 20—Calculated factor of safety on longitudinal wire strength for Wall A, B and C based on AASHTO 2002
Figure 21—Calculated factor of safety on pull out resistance for Wall A, B and C based on AASHTO 2002
Figure 22– Comparison of moisture content and Atterberg limits of the test holes near Wall B and oedometer tests in BH G09-1
Figure 23- Assumed soil layering, parameters and calculated total settlements for Wall B at the mid-point of leveling pad.
Figure 24 - Geometry of the FLAC model (X and Y coordinates are in meters)
Fixed in horizontal direction- to simulate Deck Acting as a Strut Slaved to the top of abutment wall in vertical direction

Spring to simulate Horizontal Stiffness of elastomeric bearings (Stiffness \( \approx 3800 \text{ N/mm/m} \))

Abutment Wall & footing

Precast Panels & Backfill Contact friction Angle 20\(^\circ\) with loose zone 27\(^\circ\) with dense zone

MSE Fill (Loose Zone) \((\phi=30^\circ, \text{ Dil}=-3^\circ)\)

MSE Fill (Dense) \((\phi=38^\circ, \text{ Dil}=5^\circ)\)

Bridge DL & LL & Brake

Road Fill \((\phi=38^\circ, \text{ Dil}=5^\circ)\)

Retained Soil \((\phi=36^\circ, \text{ Dil}=3^\circ)\)

Reinforcing Wire Grid

Elastic Foundation

Traffic load (17 kPa)

Notes:
- Precast panels, abutment wall, abutment footing and elastomeric spring were modeled using BEAM elements.
- Reinforcing grids were modeled using STRIP elements.
- Backfill soils (all the soil above the foundation ground) were modeled using Mohr-Coulomb Constitutive model.
- Foundation ground was modeled using ELASTIC constitutive model.

Figure 25 - Geometry of the FLAC model and summary of strength parameters for Wall B \((\phi=\text{friction angle}, \text{ Dil}=\text{dilation angle}, \text{ cohesion}=0 \text{ for all})\)
Figure 26 – Two FLAC models with and without the loose zone behind the facing panels. (The foundation soil is also modelled but not shown here.)
Figure 27– Distribution of shear modulus after bridge dead load and road fill (Numbers in the legend are in N/m², X and Y coordinates x10 in meters)
Figure 28—Distribution of vertical stresses after bridge dead load and road fill (Numbers in the legend are in N/m², X and Y coordinates x10 in meters)
Figure 29—Distribution of horizontal stresses after bridge dead load and road fill (Numbers in the legend are in N/m², X and Y coordinates x10 in meters)
Figure 30—Backfill horizontal pressure on the facing panels after bridge dead load and road fill. The foundation soil and retained soil are not shown for clarity.
Figure 31– Axial tensile forces in reinforcing grids after bridge dead load and Road fill (Numbers in legend are N/m, X and Y coordinates x10 in meters)
Figure 32– Axial tensile forces in reinforcing grids after bridge dead load and road fill
Figure 33—Distribution of cumulative vertical displacements after bridge dead load and road fill (Numbers in the legend are in meters, X and Y coordinates x10 in meters)
Figure 34—Cumulative vertical displacements at different stages of construction—see from bottom to top. (Numbers in the legend are vertical displacements in meters. Negative values denote downward movement)
Deformations are magnified 20 times

**Undeformed shape**

**No loose zone - Deformed shape**
Max. vertical displacement ~ 35 mm
Tilting inward (into the soil)
(Analysis wb174)

**With 1.5m wide loose zone - Deformed Shape**
Max. vertical displacement = 53 mm
Tilting outward (towards the pier)
(Analysis wb173)

Figure 35- Deformation pattern of the abutment and facing panels after bridge dead load and road fill
Figure 36- Deformation pattern of reinforcing grids after bridge dead load and road fill  (deformation of grids is magnified 10 times- Background graph is soil cumulative vertical displacements)
Figure 37- Exaggerated deformed shape of reinforcing grids after bridge dead load and road fill. Deformed shape of the grids is quantified by the grid slope at connection and maximum sagging of grids.
Figure 38a- Variation of draping shape of reinforcing grids with height from the base of MSE wall after bridge dead load and road fill
(See previous figure for definition of elevation mismatch and slope angle alfa)

- 1.50 m wide loose zone
- No loose zone
Figure 38b- Effect of bridge dead load and road fill on draping shape of reinforcing grids

- 1.50 m wide loose zone- End of MSE wall, before Bridge DL +Road Fill
- No loose zone- End of MSE wall, before Bridge DL +Road Fill
- 1.50 m wide loose zone- After Bridge DL +Road Fill
- No loose zone- After Bridge DL +Road Fill

Figure 38b- Effect of bridge dead load and road fill on draping shape of reinforcing grids
Connection with facing panel

Range of Settlement observed in the field at the Leveling Pad

Settlements increased by about 4 times the observed settlement of the Leveling Pad

- 1.50 m wide loose zone - Settlement of leveling pad in the range of field observation
- No loose zone - Settlement of leveling pad in the range of field observation
- 1.50 m wide loose zone - Increase settlement - Drop foundation stiffness by 0.25
- No loose zone - Increase settlement - Drop foundation stiffness by 0.25

Figure 39- Settlement profile under the MSE wall as the result of foundation ground settlement
Figure 40- Effect of foundation settlement on variation of draping shape of reinforcing grids with height from the base of MSE wall after bridge dead load and road fill

- 1.50 m wide loose zone- Settlement of leveling pad in the range of field observation
- No loose zone- Settlement of leveling pad in the range of field observation
- 1.50 m wide loose zone- Increase settlement- Drop foundation stiffness by 0.25
- No loose zone- Increase settlement- Drop foundation stiffness by 0.25
Figure 41- Effect of stiffness of the loose zone on the displacement pattern of the abutment footing (under bridge dead load and road fill). Rigid foundation ground is assumed.
Figure 42- Effect of the width of the loose zone on the displacement pattern of the abutment (under bridge dead load and road fill).
Rigid foundation ground in assumed.
Stiffness of the loose zone was assumed two times softer than the dense zone.
Figure 43- Photo showing the collapsed bridge after Izmit major earthquake in 1999 (taken from SoilTech 2000). Note that the approach MSE wall within a few meters from the fault line survived the earthquake without failure despite relatively large deformations. (Refer to Amini et al. 2010. Assessment of seismic performance of a Reinforced Earth wall using dynamic analysis, Canadian Geotechnical Conference in Calgary, 2010)
Appendix 1

Assumptions for External and Internal Stability
Analysis of MSE Walls A, B and C
Design Parameters for wall B

- **Welded wire mesh**: \( f_y = 517 \text{ MPa} \) (from SSL Drawing RW-01)
- **Reinforced soil**: \( \phi = 34^\circ, \gamma = 22 \text{ kN/m}^3 \) (from SSL Drawing RW-01)
- **Retained soil**: \( \phi = 31^\circ, \gamma = 21 \text{ kN/m}^3 \) (inferred from SSL Drawing RW-01)
- **Foundation soil**:
  - Desiccated Clay: replaced with compacted granular material with minimum 2.0 m thickness.
  - Fissured CLAY: \( \phi = 21^\circ, \gamma = 18 \text{ kN/m}^3, C = 5 \text{ kPa} \) (thickness = 4.0 m)
  - Layered Silty CLAY and Clayey SILT: \( \phi = 28^\circ, \gamma = 18 \text{ kN/m}^3, C = 10 \text{ kPa} \) (thickness > 10.0 m)
    (from Golder report - preliminary geotechnical assessment - August 21, 2009)
- **Design life**: \( = 100 \text{ years} \) (from SSL Drawing RW-01)
- **Bridge dead load**: \( = 181 \text{ kN/m} \)
  - Note: Dead load was increased from 157 kN/m (from SSL Drawings) by 15% due to extra deck overlay thickness, according to B&T.
- **Bridge live load**: \( = 62 \text{ kN/m} \) (from SSL Drawing RW-01)
- **Braking load**: \( = 6.9 \text{ kN/m} \) (from SSL Drawing RW-01)
- **Traffic surcharge**: \( = 17 \text{ kPa} \) (from SSL Drawing RW-01)
- **AASHTO 2002 requirement for factor of safety**.
  - Safety factor (Overturning): Eccentricity at the base < \( L/6 \)
  - Safety factor (Sliding): \( = 1.5 \)
  - Safety factor (Pullout): \( = 1.5 \)
  - Safety factor (Bearing capacity failure): \( = 2.5 \)
  - Safety factor (Wire strength): \( = 2.08 \)
    - Note: Maximum tensile stress should be less than \( 0.48x f_y \). This results in an equivalent factor of safety of 2.08 against yielding
    - Note: The above safety factors were consistent with those on SSL drawings.
    - Seismic stability: F.S. >= 75% of static F.S. (All failure modes)
- **Equivalent coefficient of horizontal acceleration of design earthquake 1 in 2475 year return period=0.14 g**
  (from Golder preliminary geotechnical assessment - August 21, 2009)
- **Corrosion rates**:
  - Galvanization (from AASHTO 2002 - consistent with SSL drawings)
    - 15 mm/per year (first two years)
    - 4 mm/per year (After)
  - Carbon steel
    - 12 mm/per year

Note: Thickness of galvanization coating was assumed 0.086 mm (this is the minimum thickness required by AASHTO)
Design Parameters for wall C

- Bridge dead load: $=224$ kN/m  Note: Dead load was increased from 195kN/m (from SSL Drawings) by 15% due to extra deck overlay thickness, according to B&T.

- Bridge live load: $=85$ kN/m  (from SSL Drawing RW-01)
- Braking load: $=6.9$ kN/m  (from SSL Drawing RW-01)
- Traffic surcharge: $=17$ kPa  (from SSL Drawing RW-01)

Other assumption same as Wall B

Design Parameters for wall A

- Retained soil: $\phi=34^\circ$, $\gamma=21$ kN/m$^3$  (inferred from SSL Drawing RW-01)
- Traffic surcharge: $=17$ kPa  (from SSL Drawing RW-01)

Other assumption same as Wall B
Site Plan

Modified from SSL drawing, RW-02 Wall A
Elevation of Wall A – Including abutment

Modified from SSL drawing, RW-09 Wall A

NOTE: ALL THE W11 and W20 ARE DEFORMED WIRES (D11 and D20).
Elevation of Wall B – with abutment

Modified from SSL drawing RW-06 Wall B

NOTE: ALL THE W11 and W20 ARE DEFORMED WIRES (D11 and D20).
Elevation of Wall C – with abutment

Selected section for design check

Modified from drawing RW-05 Wall C

NOTE: ALL THE W11 and W20 ARE DEFORMED WIRES (D11 and D20).
Selected section - Wall A for design check

"W" on SSL drawings is changed to "D" on this sketch as deformed wires were actually used at this project

Information obtained from drawings RW-09 + RW-12 + RW14 Wall A

Westside Road Interchange Project, MSE Wall Failure, Kelowna, BC

April 23, 2012
Selected section with abutment footing - Wall B for design check

"W" on SSL drawings is changed to "D" on this sketch as deformed wires were actually used in this project.

Assumed dimensions of abutment:

- a: 818 mm
- b: 575 mm
- c: 400 mm
- d: 575 mm
- e: 400 mm
- f: 804 mm
- g: 1114 mm

Information obtained from
SSL drawings RW-06 + RW-13 + RW-16 Wall B and B&T Figure SK001
Selected section with abutment footing - Wall C for design check

Assumed dimensions of abutment:

- a: 308 mm
- b: 575 mm
- c: 400 mm
- d: 575 mm
- e: 400 mm
- f: 778 mm
- g: 1114 mm

Information obtained from SSL drawings RW-06 + RW-06 + RW-08 Wall C and B&T Figure SK002

"W" on SSL drawings is changed to "D" on this sketch as deformed wires were actually used in this project.