REPLACEMENT OF A 721 FOOT TIMBER TRESTLE

WITH A RETAINING STRUCTURE

AND GRANULAR FILL

MILE 118.93 YALE SUBDIVISION

CANADIAN NATIONAL RAILWAY

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ABSTRACT

The bridge at Mile 118.93 Yale Subdivision is a steel and timber structure used by six railroads with a daily average traffic of forty trains and is located in Greater Vancouver, British Columbia. Constructed in 1969 it connects the Port of Vancouver with CN’s North American network and is comprised of a 150 foot steel frame structure combined with a 721 foot timber pile trestle.

In 1993 the timber trestle suffered extensive damage from a fire that swept through 36 of the 61 bents. At the time, temporary repairs were undertaken to re-establish traffic. In time, decay of the timber caps and stringers necessitated a full replacement of the bridge.

The trestle is flanked within several feet by Front Street to the west, CP to the east, and an elevated portion of Vancouver’s LRT system also immediately to the east. Access for construction materials and equipment posed a significant challenge in addition to the logistical problems created by having to obtain work windows on adjacent Front Street and from several railroads.

In 2003, CN designed a replacement structure consisting of two types of retaining structures: an H-pile and concrete lagging wall and a mechanically stabilized earth retaining structure. Transition from the retaining structure to the existing steel span was achieved with two pre-stressed pre-cast concrete spans on concrete caps and steel H-pile foundations.

The mechanically stabilized earth retaining structure under railway loading is the first such installation on CN property.
This paper highlights the reasons for selecting the varied components of the replacement structure. It addresses the constructability issues, deals with the geotechnical concerns and the foundation preparation techniques employed prior to construction of the retaining walls.

Key Words: mechanically stabilized earth, retaining structure, timber trestle replacement
1 INTRODUCTION

The ballast deck timber trestle located at mile 118.93 of the Yale Subdivision in New Westminster (Greater Vancouver), British Columbia is on one of the most critical links of CN’s system, connecting the Port of Vancouver with CN’s North American network (Figure 1). It provides the necessary approach grade to the west end of the Fraser River Swing Bridge. This 721 foot long timber trestle was built in 1969 with a two degree curve at the north end and an eight degree curve on the south portion of the bridge (Figure 2). It carries an average of 40 trains per day and is used by six different freight and passenger railroads.

The trestle is flanked within several feet by the Canadian Pacific Railway to the east, Front Street to the west, and an elevated portion of Vancouver’s Light Rapid Transit system also to the east. Front Street is a busy primary bypass route for the city of New Westminster carrying a substantial amount of commercial truck and commuter traffic (Figure 3).

A fire engulfed 36 of 61 bents (a bent is composed of a row of five or six piles with a horizontal cap on top of them) in 1993 significantly charring the exterior of the bridge timbers. However, damage was not extensive enough to warrant replacing the structure at the time. Temporary repairs were undertaken at the time in order to re-establish traffic. As time progressed, there was an increase in the number of decay pockets being reported at the cap and stringer levels resulting in a need for a full replacement of the trestle.

Consideration was given to replacing the timber trestle with conventional bridge spans on concrete pier caps supported on deep foundations. However, as a result of the
strict operating restrictions from several railroads, the site’s physical constraints, and the existing soil conditions, less conventional methods of replacing the bridge were analysed. Retaining structures offered advantages when working within confined limits and with limited work blocks as well as decreasing the possibilities of fouling both road and rail traffic when placing deep foundations.

2 CONVENTIONAL BRIDGE

CN has been very successful in replacing existing timber trestles with conventional bridge designs. Pre-cast concrete superstructures sit on substructure elements composed of driven or augured steel or concrete piles, topped with pre-cast or cast-in-place concrete caps (Figure 4).

Typically, timber bents have a span length of 12 to 14 feet. The proposed replacement foundation piles are placed mid-way between the existing bents and are designed to accommodate spans of double or triple the existing span length. Foundation piles are driven through an open timber deck, or outside the confines of the existing deck. The procedure of driving piles through a ballasted timber deck requires considerable preparation and extensive out-of-service track time. With the substructure piles installed, the pre-cast concrete caps are set on the piles and grouted in place. The existing deck and pile bents are then removed and the pre-cast concrete spans are placed on the caps.

The conventional bridge replacement option consisted of 36-foot double voided concrete box girders, situated on cast in place pier caps. These caps were to be supported by either two 36-inch diameter concrete caissons or by four 24-inch diameter concrete filled steel pipe piles. The caissons or pipe piles would be embedded 30 feet and 20 feet
respectively below grade. The concrete caissons and pipe piles would have to be placed outside of the confines of the existing deck.

The issues pertaining to the geometric and physical constraints with respect to the proximity of Front Street and CP’s track were found to hinder the construction of a conventional bridge design. Retaining structures were found to be better suited for dealing with the issues found at this site. A conventionally built bridge would involve obtaining extensive work blocks from both the municipality for access to Front Street and from several affected railroads. These would be necessary to facilitate access for construction materials and equipment. Selection of a retaining structure was found to lessen the severity of impact on both railway operations and on commercial truck traffic.

3 RETAINING STRUCTURES

3.1 Options Considered
A steel rigid frame structure over Front Street connects to the 61 bent ballast deck timber trestle. The north end of the bridge links to an existing tied back sheet pile wall approximately 9 feet in height.

The possibility of driving sheet piles and filling in the 721 ft. trestle was briefly reviewed, but the depth required for the walers could potentially encroach onto the adjacent CP track envelope and Front Street. Another issue was the amount of time Front Street and CP would have to be disrupted in order to drive sheet pile walls. In addition, geotechnical drilling data revealed the probability of encountering cobbles, boulders, and timber logs at the depths required for the necessary embedment. Driving sheet piles under these circumstances and attempting to backfill with the existing trestle overhead would be difficult.
Five classes of retaining structures were considered as possible replacements for the existing trestle:

- Geo-Web Fill
- Wire Wall
- Concrete Pre-Cast T-Wall
- Terraclass System – Mechanically Stabilized Earth (MSE)
- Tied back H-Pile and Lagging

The geo-web, wire wall, and concrete pre-cast T-walls were found to have disadvantages as possible solutions for this situation. Their shortcomings included placement inflexibility around the existing timber bents, and limitations pertaining to compatible fill type. They were also found to be considerably more expensive to install than a reinforced earth retaining wall of the embedded strip type.

CN’s familiarity with the use of MSE walls has only regarded retaining structures that were located outside the zone of influence of any train loading. There were concerns with the limited rail bed width, the extensive curve on the bridge, and the fact that the bridge was located in an earthquake zone.

Tied back retaining walls required the placing of H-piles, soil anchors, and walers. With the restricted work area, clearance from the adjacent CP track and the limited track time that was available, this option did not show much promise when first considered.

### 3.2 Geotechnical Investigation and Foundation Remediation

Borehole and test pit logs revealed a sequence of at least two fills overlying dense till in the area of the trestle. Both standard and Becker penetration tests were performed around
the trestle. They indicated very low Standard Penetration Test (SPT) blow counts to a depth of 10 feet below the CP grade. Equivalent SPT “N” values ranging between 2 and 4 were typical, with some zones having values of less than one. The southern part of the trestle contained fills separated from the till by logs underlain by loose, organic sandy silt. The fills were composed of loose sand and gravel with cobbles. Where sandy silt was found, the lower fill also contained smaller wood waste fragments.

The water table averaged 7.5 feet below grade at the south end of the bridge (Bent 1) and 10.5 feet below grade at the north end (Bent 61).

Analyses to determine the global stability of a typical design section of MSE wall, seismic design criteria, assessment of liquefaction potential, liquefied strength of foundation soils, and post earthquake stability were undertaken. The results were not favourable on account of the soft layers of soil and organic matter overlying the till.

The southern end of the bridge exhibited loose fill containing logs and sandy silt overlying till, while the northern end was predominantly loose fill. Both ends of the bridge demonstrated higher than allowable differential settlement. The allowable differential settlement criteria are 1% longitudinally and 3% transversely. These settlement criteria were critical to prevent excessive deformation of the reinforcing strips contained within the fill.

Stability was the next topic analyzed. Liquefaction of the loose fill and sandy silt below the water table would be expected between the 1/100 year return period for earthquakes (Level I- Serviceability Limit State) and the 1/475 year return period for earthquakes (Level II- Ultimate Limit State). This fact made it apparent that the MSE
wall would not be able to meet AREMA’s seismic performance standards with the loose fill in place.

Remedial action was required to improve the foundation to a point where stability would not be compromised in the presence of a 1/2400 year return period earthquake (Level III- Survivability Limit State). A 10 ft. depth of poor soil was removed in order to implement soil improvement procedures. Figure 6 depicts the excavation limits, under the MSE wall, required for the removal of existing soft material, and subsequent replacement with well-compacted granular material. The compacted granular replacement fill yielded a higher density soil stratum under the trestle, and much lower settlements under load. The new granular fill had a much higher resistance to liquefaction. The resulting factor of safety of static stability was greater than 1.5, remaining above 1.0 under peak ground accelerations up to the 1/2400 return period earthquake event. Thus, the MSE wall was found to be compliant with the AREMA criteria for post earthquake stability of the foundation.

The physical constraints present at this site made construction of the embankment unique. There has never been such a narrow MSE embankment constructed where most of the length of soil reinforcement strips was overlapped with the strips from the wall on the opposite side of the embankment.

The internal stability analysis and the actual wall design was undertaken by the MSE designers. The approach used was to design the walls on each side of the track independently, assuming no effect from the opposite wall or its overlapping soil reinforcement. It was felt that this assumption would be generally conservative and a higher order analysis (FLAC - Fast Lagrangian Analysis of Continuum) was facilitated to
verify this assumption. In addition, the base design analysis was carried out using a proprietary program featuring a limit state equilibrium.

### 3.3 Selection of Structure Types
An H-pile and concrete lagging tied-back wall was selected for the north half of the retaining structure due to the high costs of soil remediation in this 363 foot section (Figure 5). Proceeding north from Bent 30 to Bent 61, Front Street begins to rise relative to the grade at CP. In order to sub-excavate the weak soil from under the bridge, installation of temporary shoring would be required to support the Front Street embankment. By driving H-piles and installing concrete lagging, the weak soil could be left in place and any resulting settlement would be acceptable.

In addition, as a result of the rise in elevation of Front Street, a retaining wall was only required on one side (CP side) of the timber trestle as fill material could be placed against the Front Street sub-grade.

Piles were driven 8 feet into the till at 6 foot intervals and staggered to avoid the 12 foot spacing of existing bents. Tie back soil anchors were installed from the CP side and extended under Front Street. An anchorage detail running through the web and flange of the pile was developed precluding the use of walers because they would foul the CP clearance envelope.

A double-sided MSE wall was selected to be constructed from Bent 4 to Bent 30. This 318 ft. long section varied from 15 to 20 feet in height on the CP side and from 12 to 20 feet in height on the Front Street side (Figure 6).

With double the 100 year required AASHTO thickness of reinforcing strips and with historical corrosion rates generally slower than AASHTO rates used in the design of
MSE walls, there is a reasonable expectation that 100 years of service life will be attained. Maximum vertical displacements due to consolidation were calculated to be 1 3/8” near the base of the wall. As this zone falls inside the MSE volume, much of this consolidation occurs as the wall is being built. The estimated consolidation at track level was calculated to be only 5/8”. Vertical live load surcharge was taken as 2100 psf. And a lateral load of 300 plf was applied at the top of rail.

The constricting proximity of Front Street to the south end of the trestle, Bents 1 to 4, prohibited the use of an MSE wall at this location. Accordingly, a more conventional solution was implemented for the final 40 foot approach to the rigid steel frame structure. This was accomplished with the installation of two pre-stressed pre-cast concrete spans on pile foundations (Figure 7). The first pier (B1) was placed at approximately 16 feet to the north of the rigid frame bridge, while the second pier (B2) was placed 24 feet north of B1 and was enveloped by the MSE wall. The spans and H-pile foundations were designed to resist the effects of earthquakes, lateral loads associated with the 8-degree curve and longitudinal forces related to tractive effort and braking. The concrete pier caps cast in place at the top of the H-piles provided an adequate bridge seat that was just clear of oncoming heavy truck traffic on Front Street.

Preparatory work was necessary in order to construct the piers and install the spans. This included the removal of two sections of the ballasted deck that was replaced with an open deck. The open deck configuration provided access to drive piles, place formwork and pour concrete for the pier caps.

The resulting 721 feet of replacement structure was to consist of a tied back H-pile wall, an MSE wall, and two new pre-cast pre-stressed concrete spans (Figure 8).
4 CONSTRUCTION OF THE RETAINING STRUCTURE

4.1 Site Preparation
The first phase of construction began with preparation of the jobsite. CN negotiated with the Greater Vancouver Regional District (GVRD), the City of New Westminster and CP Rail to obtain access and convert the surrounding area into a construction site for the eight month duration of the project. An agreement with the GVRD was arranged to encroach on Sapperton Park located adjacent to CP’s track next to the trestle. A portion of the walkway and the grassy area of the park were fenced off and used for a material and equipment storage site, with the provision that CN would restore the park to its original state once the project was complete.

Crossing planks were installed for a length of 900 feet along CP’s track to provide a smooth surface for equipment working next to the trestle. In addition, CP provided flagging protection for the duration of the project. Discussions with CP also included scheduling trains to provide sufficient time for a construction window that would be productive.

Negotiations with the City of New Westminster included a variance on the noise by-law to permit working at night, as well as road closures on Front Street at various times during the construction. Front Street is a corridor heavily used by commercial vehicles, large trucks, and commuter traffic. Consequently, daytime closures during the week were unacceptable to the City. The City restricted the use of Front Street as a staging area to nights and weekends.

All utilities in the area were located to ensure nothing was damaged during the excavation and pile driving operations.
4.2 Sub-excavation of Poor Soil
As the soil below the timber trestle was weak and contained buried timber, sub-excavation was required to remove the poor soil between Bents 4 and 30. An excavator was used to dig the soil out from between the pile bents by excavating to the dense till layer, an average depth of 10 feet (Figure 9). This material was then hauled off site for disposal. The excavation was immediately backfilled with 3” minus granular material and compacted in 6 inch lifts to a 95% Standard Proctor Density. Keeping the excavation open for more than 30 minutes could cause slope failures and undermine Front Street or the CP track as the water table was often identified at 7-8 feet below grade. The backfill was placed to an elevation level with the top of rail on the adjacent CP track.

As discussed previously, the soil conditions between Bent 30 and Bent 61 were not a concern because a tied back H-pile wall was constructed to replace this section of the timber trestle.

Settlement was not a concern north of Bent 30 because there are no reinforcing strips in the backfill as there are with the MSE wall. In addition, the gradient of the track and the natural ground converge in a northward direction decreasing the depth of fill required and reducing the load on the earth below.

4.3 Construction of the Tied-back H-pile Retaining Wall
In order to maintain current levels of rail service, the construction sequence of the H-pile retaining wall had to be completed in the following order:

1. drive H-Piles
2. drill and grout soil anchors
3. install lagging and backfill
4. remove track, ballast and bridge deck
5. install final lagging and backfill
6. replace track panel and ballast

4.3.1 H-pile Installation
Sixty-three H-piles were driven at six foot centers and positioned just to the outside edge of the pile cap on the existing timber trestle. The piles were HP14 X 117 with the longest pile having a length of about 33 feet. The height of the wall at Bent 30 was 15 feet above the adjacent CP track reducing to about 11 feet high at Bent 61. Initially, a vibratory pile driver (Model Ice-316 with a weight of 5,500 lb) was used to drive the piles through the weak overburden into the dense till below. A diesel hammer (Model D30-32 with a rated energy of 91,088 ft-lb) was then used to drive them to a minimum depth of 8 feet into the till.

A template for pile driving was constructed by driving two temporary piles and attaching two horizontal beams to them. Lugs were welded to the inside of the beams to hold the H-piles in position while driving took place.

A portion of the web and flange of the H-pile was removed by torch cutting to accommodate the tie-back anchor. This area of the pile was strengthened by welding two semi-circular sections of pipe to each side of the web and to the inside of both flanges.

4.3.2 Soil Anchor Installation
The soil anchors used to tie back the H-pile wall are Double Corrosion Protected (DPC) steel thread bars with a diameter of 1 3/8 inches and yield strength of 100ksi. They were installed in two sections and joined together with a steel coupler that was protected from corrosion by placing a rubber heat shrink sleeve over the connection. The anchors were designed to have their entire 24 foot bond length in the till layer, with the top of the bond
zone at least 3 feet into the till. The free stressed portion of the anchor, which extended through the loose overburden to meet the anchor assembly at the H-pile, was 36 feet long.

Cased boreholes were drilled at a declination of 25° with an Odex drill using a downhole hammer with rotary casing (Figure 10). A 5 ½ inch OD steel casing was inserted into the borehole as the drill progressed and stopped just beyond the till/overburden interface. Drill cuttings were removed by forcing air through the drill rods and bit. The DCP anchors were installed using plastic centering devices to keep the anchor in the center of the borehole and casing. A ¾ inch diameter plastic pipe was inserted into the borehole to the drilled depth to facilitate pumping of the grout. The casing was removed at 6 foot intervals as the anchors were filled with grout. As the anchors have lower overburden pressure, it was important to control the grouting pressure to avoid ground fracture and impacting the asphalt road surface above as well as damaging a nearby water main.

After the first section of the anchors were installed and allowed to set up, proof testing was conducted. Each anchor was loaded in increments of 25 % of the design load of 54 kips to a value of 133% of the design load. Each load was held for a duration of three minutes and all anchors tested satisfactorily.

4.3.3 Installation of Concrete Lagging
Prior to the installation of the lagging, a trench 20 inches deep was excavated along the H-piles. A 6 inch perforated PVC drainpipe was wrapped in filter fabric and situated in the bottom of the trench that was then backfilled with drain rock. The drainpipe was eventually connected into the City’s storm sewer system.
Pre-cast concrete lagging segments measuring 31 inches high and 7 inches thick were inserted between the H-piles. The first segment was supported on a steel bracket that was welded to each H-pile at the bottom of the trench. In order to ensure a proper fit with the piles, it was important to ensure that the lagging was installed level and at the corresponding design elevation.

As the pre-cast segments were installed, the fill was placed and compacted to a Standard Proctor Density of 95%. The inside of the lagging was lined with filter fabric to prevent soil from migrating through the joints. Each panel rested on neoprene padding to minimize excessive wear by preventing stress concentrations at the concrete contact points. The lagging and granular fill was installed until a minimum headroom prevented suitable compaction efforts to achieve the 95% Standard Proctor Density (approximately 6 feet below base of rail). To offset the effects of settlement of the fill on the DCP anchor, a 6 inch diameter pipe was positioned over the anchor where the new fill material was placed. If settlement occurred, the pipe would take up the downward displacement rather than the soil exerting pressure on the anchor. Once the lagging and fill reached the maximum height prior to removing the bridge deck, the anchors were temporarily locked off at 70% of their design load to retain the active pressures of the new fill.

### 4.3.4 Removal of Bridge Deck and Stringers

To complete the upper portion of the retaining wall, several blocks on rail traffic were required to perform the work. During the first 5 hour work block, a trial 35 foot section of the bridge was removed as a test case to ensure that all procedures and processes were properly developed to convert the bridge to a fill and conventional track structure.
In preparation for the closure, sufficient fill was hauled and stockpiled at the site. The complete 721 feet of track length of CWR was segmented into 35 foot sections. Thirty five feet was chosen as it was the maximum length of bridge deck that the largest available crane could safely lift. A 180 ton crane, excavators, compactors and various other tools and equipment were on site ready to start the conversion process. The track closure (CN mainline and CP spur line) was arranged on a Sunday night to minimize the impact to rail traffic and road traffic on Front Street. The appropriate flagging protection was arranged for the closure of Front Street.

At the beginning of the work window the crane lifted the 35 foot track panel off of the bridge and set it down on the CP track. All of the crane work for this project was severely constrained by the overhead light rapid transit line. At this time two excavators scooped up the ballast from the bridge deck. Next a cutting wheel was attached to one of the excavators to cut the 4 inch thick deck planks longitudinally along the center of the bridge (same principle as a pizza cutter). Slings were wrapped under the stringers and as the 180 ton crane lifted the 35 foot section of the stringers and deck it folded in half. The crane then placed it in an awaiting dumpster where it was immediately hauled off site for disposal in order to provide room for the remainder of the construction. The pile caps and about one foot of the piles were cut off prior to placing the granular fill. The remainder of the concrete lagging and backfill was installed; the track panel set in place, ballast dumped and the track was surfaced. The balance of the H-pile/lagging retaining wall was completed in a similar fashion. Two track and road closures were needed: one of five hours to complete 70 feet and another of 12 hours to complete the last 258 feet of the H-pile wall.
4.4 Construction of the MSE Retaining Wall

The MSE retaining wall was built on both sides of the existing bridge structure from Bent 4 to Bent 30. Construction began by pouring a concrete footing on an alignment just to the outside of the existing timber piles. When the footing was cured, the first row of pre-cast concrete retaining wall panels was placed. On the back of each panel there are six connections where galvanized steel reinforcing strips are bolted and extended into the granular fill (Figure 11). Each strip is 2 inches wide, 3/16 inch thick and 14”-9” long. The reinforcing strips were flexible and could be manoeuvred around the existing piles where necessary. After a row of panels and steel strips were set into position, granular fill was placed in 8 inch lifts and compacted to a Standard Proctor Density of 95%.

Subsequent rows of panels were installed in an interlocking fashion by inserting Neoprene bearing pads between the segments. The clearance under the existing bridge created a problem for equipment to place the granular fill, so a dump truck with a conveyor system attached to the rear was utilized to shoot the gravel under the structure. The fill was compacted with several passes using 1000 pound vibratory plate tampers. Filter fabric was also used to line the inside of the MSE panels.

Similar to the H-pile and lagging portion of the wall, the concrete panels and fill were installed as close to the bottom of the stringers of the timber structure as possible. One night time track closure of 16 ½ hours was required to complete the remaining 6 vertical feet of MSE retaining wall and convert the timber trestle to a fill and conventional track structure. This was accomplished in a similar manner to the H-pile and lagging portion of the retaining wall. The top of both wall sections was finished off with a pre-cast concrete cap equipped with inserts to bolt on hand the railing.
4.5 Transition Bridge Structure
To provide a transition between the existing steel frame bridge that crosses over Front Street and the MSE retaining wall, a concrete bridge on H-pile foundations was constructed. Two piers were constructed by driving two rows of five H-piles (HP12x74) in each row. One of the piers was contained within the MSE wall to provide a suitable transition to the filled structure.

As the original structure was a ballast deck timber trestle, the ballast deck at the pier locations was converted to an open deck to provide access for driving piles. The distance of 36 inches from top of pile cap to base of rail had to be maintained when the ballast deck was removed. This was accomplished by installing 4 timber stringers that were 18 x 32 inches with a four inch notch cut out of the bottom. Timber ties measuring 10 x 8 inches were set on top of the stringers to which the rail was reattached.

Once the steel H-piles were driven, a cast-in-place concrete cap was constructed on each pier. During the track closure to complete the MSE retaining structure, two pre-cast concrete spans were installed. A 120 ton crane was used to lift the temporary timber spans from the bridge, then the existing timber pile bents were cut off at ground and removed. The span from the MSE wall to the first pier was a double voided concrete box girder and the span from the first pier to the existing steel frame bridge over Front Street was a 23 inch thick concrete slab. Both were set in place with the 120 ton crane. Placement of the concrete spans and completion of the full length of the MSE wall was achieved in a track/road closure of 16 ½ hours.
5 CONCLUSIONS
There were several challenges that had to be overcome on this project. First, was deciding what type of structure to construct. Once a bridge was ruled out due to cost and site constraints, several retaining structures were evaluated to determine the best overall solution. A combination of the H-pile and lagging wall and the MSE wall were a good fit for the site conditions, construction sequencing, as well as being the most economical solution.

Secondly, the biggest challenge with respect to construction was the logistics of equipment and material on site. The constraints imposed by Front Street on one side of the bridge and CP’s track and Sapperton Park on the other side posed significant hurdles to surmount. Working on weekends and evenings/ nights was essential to minimizing the disruption to rail traffic and the substantial commercial and commuter traffic on Front Street.

Construction of this structure has provided CN with a reliable, cost effective solution to the replacement of a timber trestle that has reached the end of its useful life (Figure 12).
Table of Figures

Figure 1: Location Map ........................................................................................................ 23
Figure 2: Original Ballast Deck Timber Trestle ............................................................... 23
Figure 3: Showing the Tight Proximity of Front Street .................................................. 24
Figure 4: Concrete Span on Caissons ............................................................................. 24
Figure 5: Cross-section of Tied Back H-pile Retaining Wall ........................................ 25
Figure 6: Cross-section of MSE Retaining Wall ............................................................. 25
Figure 7: Cross-section of Concrete Spans ...................................................................... 26
Figure 8: Plan View of the Three Structures ................................................................... 26
Figure 9: Photo Showing the Sub-excavation .................................................................. 27
Figure 10: Photo of Drilling Soil Anchors ...................................................................... 27
Figure 11: Photo Showing the Reinforcing Strips of the MSE Wall ............................... 28
Figure 12: Photo Showing the Completed Structure ....................................................... 28
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Figure 1: Location Map

Figure 2: Original Ballast Deck Timber Trestle
Figure 3: Showing the Tight Proximity of Front Street

Figure 4: Concrete Span on Caissons
Figure 5: Cross-section of Tied Back H-pile Retaining Wall

Figure 6: Cross-section of MSE Retaining Wall
Figure 7: Cross-section of Concrete Spans

Figure 8: Plan View of the Three Structures
Figure 9: Photo Showing the Sub-excavation

Figure 10: Photo of Drilling Soil Anchors
Figure 11: Photo Showing the Reinforcing Strips of the MSE Wall

Figure 12: Photo Showing the Completed Structure