Mitigating Environmental Impacts on the Sea-to-Sky Highway through Innovative Structural Concepts and Details

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ABSTRACT

In preparation for the 2010 Winter Olympics, the Sea-to-Sky Highway from Vancouver to Whistler has undergone major improvements with respect to safety, reliability and capacity. MMM Group has worked closely with the design-build team to mitigate environmental impacts through the selection of structural concepts and details that reuse existing structural elements, optimize the use of construction materials and reduce the structural footprint. Examples presented in this paper where these principles were successfully implemented include (a) the retention and reuse of existing bridge substructure elements; (b) the reuse of an existing bridge as a temporary detour crossing; and (c) the innovative combination of cast-in-place concrete and mechanically stabilized earth walls in stacked and tiered arrangements.

As a commitment to local residents in Squamish, noise mitigation measures were provided through the urban Squamish corridor. This was achieved in part through the use of open-graded friction course (OGFC) pavement. The coarser gradation and increased air voids of OGFC pavement have noted noise reduction features and improved safety features but require special attention to bridge deck surface drainage and pavement finishing.

1. INTRODUCTION

Figure 1 – The Sea-to-Sky Highway

The Sea-to-Sky Highway (Figure 1) is located in British Columbia's Coast Mountains, serving as the principal transportation route for the numerous communities from Vancouver to Whistler (Figure 2). In addition, this provincial highway will be a vital link between these two host cities for the 2010 Winter Olympic and Paralympic Games. With the games serving as impetus, the highway has recently undergone major improvements with respect to safety, reliability and capacity. Specific improvements to the 95 km corridor included (1) highway realignment to provide improved sightlines, consistent driving speeds and shorter travel times; (2) highway widening to provide passing lanes, pull-outs and wider shoulders; (3) new highway safety features including median barriers, highly reflective pavement markings and rumble strips.

MMM Group was responsible for the design of all major structures in three of the project design sections; a total of thirteen bridge crossings and over 45 retaining walls. MMM Group worked closely with the design-build team in the development of structural concepts and details that reduced environmental impacts while meeting the design criteria and construction schedule. Such measures typically included the reuse of existing elements and the reduction in construction materials and structural footprints.

This paper will first provide a brief history of the highway and a summary of its recent improvements. In a subsequent section, the paper highlights several constructed examples where environmentally-compatible design concepts and construction methods were developed. A final section will provide background and bridge design details relating to a noise-reducing open-graded asphalt that was used in urban locations along the corridor.

2. HIGHWAY HISTORY AND PROJECT BACKGROUND

Figure 2 – Aerial View of the Construction Corridor

The southern portion of the highway from Horseshoe Bay to Squamish was built in 1959 with the extension to Whistler completed in 1966. Through the 1980s and 1990s, numerous studies focused on both expansion to the existing corridor and alternate routes through the mountains. With the success of the Olympic bid on July 2, 2003, the BC Ministry of Transportation and Infrastructure (MoT) initiated an ambitious program of improvements to the existing highway. Early work on a 0.9 km test section in 2004 provided insight on constructability, geotechnical and traffic management issues. Later in 2004, design-build (DB) construction was initiated for two 7 km sections from Sunset Beach to Lions Bay (DB2) and Culliton Creek to Cheakamus Canyon (DB11).

With the successful completion of these early projects, the MoT bundled the remaining 65 km of highway into a single $600M Design-Build-Finance-Operate (DBFO) contract. This public-private partnership (the first of its kind in BC) included a four year design-build period and a twenty-five year operations and maintenance period for the entire length of highway, including sections upgraded under earlier contracts. In addition to the highway realignment and widening, the DBFO project required construction of over 40 new bridges and 110 retaining walls, as well as upgraded drainage features, rock excavation and stabilization and environmental enhancements. The DBFO contract was awarded in 2005 to the S2S Transportation Group, a consortium that included Peter Kiewit Sons (construction), Miller-Capilano (operations and maintenance) and the Macquarie Group (financing). The design-build team was comprised of several local consultants, working closely with Peter Kiewit Sons (PKS) in all transportation-related disciplines.

MMM Group was the design manager and structural design consultant for the 7 km section from Sunset Beach to Lions Bay (DB2), completed in 2005. In the subsequent DBFO contact, MMM Group was the design consultant for the 10 km section through Squamish (DB8) and the structural design consultant for the 6 km “green-field” route above Horseshoe Bay (DB1).

By all measures, the improvements to the Sea-to-Sky Highway are considered to be a success. Each of the early packages was delivered on time and budget with minimal traffic disruption or public complaints (Ref. 1). The DBFO project is finishing in 2009 with substantial completion achieved in all payment sections. This project has been the recipient of numerous accolades and awards including the 2009 Lieutenant-Governor’s Award for Engineering Excellence, the highest honour bestowed by the Consulting Engineers of British Columbia.

3. ENVIRONMENTALLY-COMPATIBLE STRUCTURAL CONCEPTS AND DETAILS

A significant component of the project requirements was the protection and enhancement of the environment. These issues were primarily and explicitly addressed through project deliverables relating to, as example, terrestrial wildlife, fisheries habitat, permitting and compensation. Because these issues were the responsibility of the environmental consultant, they are not discussed further in this paper.

From the structural consultant's perspective, environmental considerations are often integrated within the structural design process and project criteria. For the DBFO project, all structures were designed in accordance with the CAN/CSA S6-00 Canadian Highway Bridge Design Code (Ref. 2) and the BC MoT's Standard Specifications for Highway Construction (Ref. 3). MSE walls were designed in accordance with AASHTO's 2002 *Standard Specifications for Highway Bridges* (Ref. 4) and the FHWA's *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines* (Ref. 5). Detailed project-specific structural design criteria were included in the Concessionaire's Agreement. Within these criteria, the environmental requirements were often reflected in the language of structural durability and are summarized as follows:

Design Life: New structures shall have a minimum design life of 75 years. Modified structures shall have a minimum design life of 50 years. Design calculations for corrosion and other time-related durability calculations require consideration for a design period of 100 years.

Corrosion Rates: For structural elements in non-aggressive soil: (a) galvanization loss of 15 micrometres/year for first two years and 4 micrometres/year for subsequent years; (b) carbon steel loss of 12 micrometres/year after zinc depletion.

Deck Protection System: All new bridge decks to include (a) 100 mm asphalt overlay; (b) waterproofing membrane; (c) high performance (silica fume) deck concrete; (d) epoxy-coated rebar in top reinforcing mat of bridge deck.

Beyond the contractual requirements for both environmental protection and structural durability, the design-build team had an obligation to develop design concepts and construction methods to suit the myriad other project requirements and constraints. In this regard, a common perception is that environmental concerns are relegated to a lower priority due to competing cost and schedule requirements. While this may be the case in some circumstances, there are numerous project examples where the final design concept or construction methodology produced compatible incentives with respect to both construction costs and environmental impacts. The following subsections highlight constructed examples that had these shared benefits.

3.1 Mamquam Blind Channel Bridge

The Mamquam Blind Channel Bridge is located in the Squamish (DB8) section, two hundred metres south of Cleveland Avenue. The final concept for the new crossing is a four-lane, two-span continuous 61m long structure (Figure 3). Located on a tidal slough, this new bridge replaced a two-lane, three-span structure constructed in 1965 (Figure 4).

Figure 3 – New Mamquam Blind Channel Bridge (Looking North-West)

The superstructure of the new bridge is comprised of a cast-in-place concrete deck made composite with partial-depth precast concrete deck panels and ten precast and prestressed concrete I-girders. The bridge foundations vary from a spread footing on rock at the south abutment (Figure 5) to extended concrete-filled steel pipe piles at the pier and north abutment. Potential liquefaction and embankment lateral spreading were challenging design issues and required extensive timber densification at the north abutment.

Figure 4 – Existing Mamquam Blind Channel Bridge (Looking South-West)

Figure 5 – South Abutment of Existing Mamquam Blind Channel Bridge (Looking South)

Like many of the bridge crossings along the corridor, the Mamquam Blind Channel Bridge made extensive use of precast concrete. In addition to a contractor preference for cost and schedule benefits, the use of precast concrete can minimize the spill risks associated with handling fresh concrete on a bridge site. With a main span length of 38 m, this crossing is amongst the longest spans on the corridor to use precast concrete girders. In addition, the bridge concept called for the use of partial-depth precast concrete deck panels to span between the girders and support a cast-in-place concrete topping (Figure 6). This deck construction approach is often cited for its environmental benefits in reducing the risk of spills and debris falling into a sensitive watercourse.

Early design concepts called for the continued use of the existing 40-year-old structure by either (a) building a twin structure; or (b) widening the existing structure. However, the particularly constrained right-of-way at each bridge approach required a new four-lane bridge centered on the existing alignment, pivoted slightly about the south abutment control point. This geometric constraint precluded the twinning option entirely and presented significant geometric challenges for the widening option. A detailed structural review of the widening option also indicated that the existing bridge would require extensive strengthening including seismic upgrades and had significant uncertainties relating to the existing timber piles at both the pier and north abutment. Furthermore, the steel girders were coated with red-lead paint, a toxic and carcinogenic coating no longer permitted on bridge structures.

After a detailed cost comparison of several variations of both the widening and replacement options, the contractor selected the full bridge replacement option. While a reuse option is often cited as the preferred environmental choice, the replacement option in this case provided significant environmental benefits including (a) improved design life; (b) the permanent elimination of one in-stream pier; (c) reduced in-stream work at the remaining pier; and (d) removal of the girders, thus eliminating the toxic paint exposure. Furthermore, the selected option still included the retention and reuse of several existing bridge components, described as follows:

Figure 6 – Precast Concrete Deck Panels

Reuse of Existing Bridge as Temporary Detour: The existing bridge was used as a temporary detour crossing while the first stage of new bridge construction was completed (Figure 7). Once the first stage of new bridge construction was completed, traffic was shifted to the new structure. Then, the existing bridge was removed and the second stage of new construction was completed (Figure 8). Given the pivoted highway alignment, the temporary traffic laning during the second stage construction gradually narrowed over the length of the bridge. To accommodate this pinch geometry, the bridge sidewalk (in the final configuration) was partially utilized to accommodate the Stage 2 southbound traffic. The anchored concrete barrier separating vehicular and pedestrian traffic in the final configuration was installed at a later construction stage. This traffic laning strategy also required a permanent 1.1m wide extension to the bridge deck and a temporary sidewalk extension to accommodate pedestrians during the construction period. While this approach carried both additional costs and risks, it avoided the expensive and environmentally intrusive work for a temporary detour crossing.

Figure 7: Aerial View of Stage 1 Construction

Retention of Existing Timber Piles: The embedded portions of all 139 existing timber piles were left in place. Recommended by the environmental consultant, this approach minimized the disturbance to the fisheries habitat.

Figure 8: Aerial View of Stage 2 Construction

Reuse of South Abutment Footing: The existing south abutment was comprised of a reinforced concrete wall and footing bearing on a rock outcrop (Figure 5). The new south abutment design concept called for a similar foundation, extended to the west to accommodate the wider structure. The option of reusing the existing south abutment substructure was investigated and considered geometrically feasible. A detailed materials testing program was conducted that included a half-cell potentials survey, concrete powder chemical analysis and compressive tests on concrete cores. Results from these tests indicated that a further 75-year design life could not be achieved without significant future repairs. For this reason, the option to structurally reuse the south abutment was abandoned. However, rather than an entire and intrusive demolition of the existing south abutment, the existing footings were left in place to serve as a working floor for the new abutment and wingwall footings (Figure 9). Furthermore, the structural geometry was adjusted so that the existing abutment wall could be left partially in place to (a) avoid risky demolition adjacent to the channel; and (b) enable the existing abutment to serve as an in-situ construction cofferdam during high tide periods.

Figure 9 – Reuse of South Abutment Footing

3.2 Mamquam River Bridge

The Mamquam River Bridge is located in the Squamish (DB8) section, 100 metres north of Centennial Way. The final concept was the rehabilitation of the existing two-lane structure (Figure 10) and the construction of a new parallel structure immediately downstream (Figure 11). In the final configuration, the existing bridge will provide two northbound traffic lanes and the new twin bridge will provide two southbound lanes. A single sidewalk will be located on the outer edge of each structure.

Figure 10 – Existing Mamquam River Bridge (Looking North)

Figure 11 – New Twin Mamquam River Bridge (Looking South-East)

The new twin crossing is a two-lane (Figure 12), three-span continuous 90 m long bridge. This structure consists of a cast-in-place concrete deck made composite with nine precast and prestressed concrete box girders, all founded on concrete-filled steel pipe piles. Piers for the new structure were located to match the existing bridge pier location. The new bridge superstructure soffit is 1.5 m higher than the existing (Figure 13) to provide the larger hydraulic opening required by present-day river discharge estimates.

The existing crossing is a two-lane, three-span 90 m long bridge as well. Constructed in 1977, this structure is comprised of steel girders and composite concrete deck founded on pile-supported wall-type piers and spread footings at the abutments. Rehabilitation of the existing structure included the addition of a new high-density concrete deck overlay, new deck joints and traffic barriers. A detailed structural strength review was completed to ensure the bridge can accommodate the increased dead load of the new deck overlay and paving.

To keep and upgrade the entire existing structure at this particular location had significant project cost and environmental benefits relating to the reduction in permanent materials. Furthermore the continued use of the existing bridge ensured that no temporary detour crossing was required and provided a convenient short-term working platform for girder installation and materials handling. These construction conveniences reduced the requirements for equipment access on the gravel river bed, thus reducing the risks to riparian and aquatic habitat.

Figure 12 – Aerial View (Looking South)

Figure 13 – View from Deck (Looking North)

Figure 14 – New Stawamus River Bridge (Looking South-East)

Figure 15 – Existing Stawamus River Bridge (Looking North-East)

3.3 Stawamus River Bridge

The Stawamus River Bridge is located in the Squamish (DB8) section, two hundred metres south of Valley Drive. The final concept for the new crossing is a five-lane, single-span 34 m long structure (Figure 14). This new bridge replaced a two-lane, single-span structure constructed in 1968 (Figure 15). The new structure consists of a cast-in-place concrete deck made composite with 24 precast and prestressed concrete box girders, all founded on concrete-filled steel pipe piles at each abutment. A potential liquefiable soil layer was mitigated through rapid impact compaction.

Full bridge replacement was considered the only viable option at this location given (a) the particularly substandard hydraulic opening of the existing structure; and (b) the complex geometric tie-in requirements for the intersections immediately beyond each end of the bridge. Similar to the Mamquam Blind Channel Bridge however, the substructure was kept in place to serve a new role in the bridge system and reduce environmental impacts. By leaving the existing abutments in situ, the disturbance to the riparian and fisheries habitat was greatly minimized. Furthermore, the existing abutments were left in place to provide scour protection for the retained approach fills (Figure 16), significantly reducing the requirement for riprap scour protection.

Beyond the limits of the existing abutments, steel plates were installed behind the steel pipe piles providing additional scour protection and further reducing the riprap requirements (Figure 17). The steel plates were designed so that they are vertically supported from each pile and provide the required soil retention while not contributing in-plane stiffness to the global seismic bridge response.

Figure 16 – Retained North Abutment and Steel Scour Protection Plates

Figure 17 – Backfill Side of Steel Scour Protection Plates

3.4 Garibaldi Pedestrian Overpass

Figure 18 – New Garibaldi Pedestrian Overpass (Looking West)

The Garibaldi Pedestrian Overpass is located in the Squamish (DB8) section, two hundred metres south of Garibaldi Way. The new bridge superstructure is composed of seven simply-supported spans with lengths ranging from 24 m to 34 m (Figure 18). The new overpass replaced a five-span pedestrian overpass constructed in 1979 (Figure 19). The new overpass consists of a single precast and prestressed concrete girder supported from each dapped end on a CIP pier cap with a circular column on a spread footing. Ground improvements included preloading and the removal / replacement of a liquefiable layer. In addition, the simply-supported superstructure configuration was selected to accommodate potential long-term settlements.

Figure 19 – Existing Garibaldi Pedestrian Overpass (Looking North)

The full reuse of the original piers and girders was not possible because of the widened highway, increased vertical clearance and unavoidable right-of-way constraints for the approach ramps on each side. However, the reuse of each reinforced concrete abutment (Figure 20) was feasible with respect to both design life and geometry. A detailed materials testing program was conducted that included a half-cell potentials survey, concrete powder chemical analysis and compressive tests on concrete cores. Results from these tests indicated that a further 75-year design life can be achieved with only minor future repairs and maintenance.

In order to geometrically tie-in the new highway crossing span (Span #4) with the existing abutments, the approach ramps were re-oriented so that spans #2, #3, #5 and #6 were parallel to the highway (Figure 21). By doing so, the lengths of these approach ramps could be adjusted to meet (a) the required vertical clearance over the highway; and (b) the deck slope requirements as specified in the BC Building Access Handbook (Ref. 6). These requirements specify a maximum deck slope of 1(V):12 (H) with 1.5 m long landings at 9m intervals. This variable deck profile was achieved by varying the top flange thickness of the precast girder, a task completed by the precast concrete supplier thus eliminating all on-site cast-in-place concrete work on the bridge superstructure.

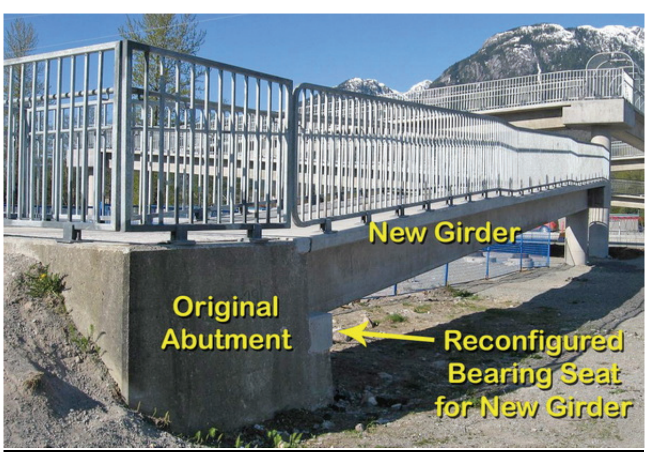

Figure 20 – East Abutment

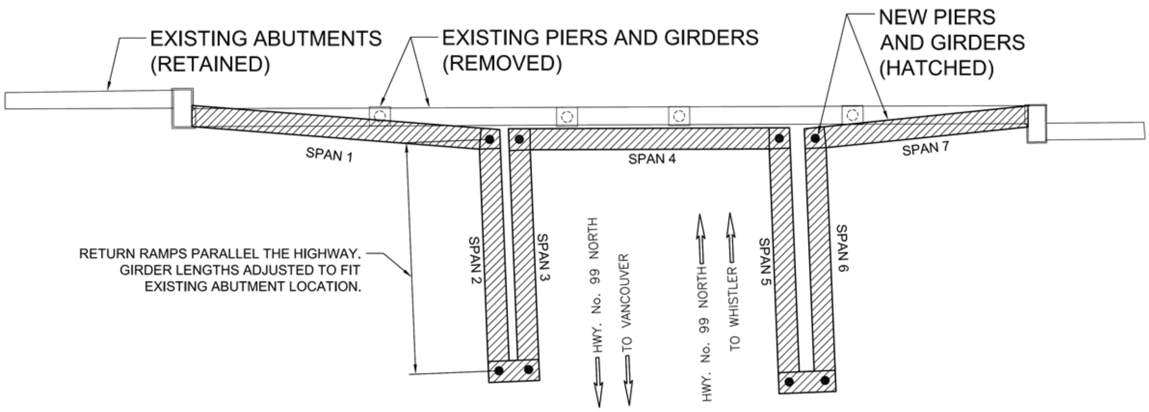

Figure 21 – Garibaldi Pedestrian Overpass - Bridge Layout

3.5 Retaining Walls

Various types of retaining walls are used extensively on the Sea-to-Sky corridor, both in combination and as replacement to bridge structures. Wall concepts, illustrated in Figure 22, include (A) reinforced concrete mass pour wall; (B) reinforced concrete cantilever wall; (C) reinforced concrete anchored wall; (D) mechanically stabilized earth; or most commonly (E & F) a combination of the aforementioned four types. While occasionally located up-slope of the highway (B & F), the preferred approach is the downslope wall (A, C and D) which eliminates the risk of debris falling on the travelling public during construction. Wall lengths on the corridor can extend for over a kilometer and the maximum overall wall height is 26.5 m (Figure 23).

While the structural principle of reinforced soil has been in use for several centuries, the modern mechanically stabilized earth (MSE) wall concept has only been in wide-spread use since the 1970s. An MSE wall relies on layered tension elements, embedded in the backfill at regular intervals to maintain a stable reinforced soil block (Figure 24). The MSE reinforcing strap lengths are typically 70%-100% of the wall height, depending on the foundation conditions and slope stability requirements. This large foundation footprint can be a major drawback to the system.

MSE Wall Use on the Sea-to-Sky Highway: The predominant MSE retaining wall system used on the corridor is a system with a wire-mesh facing panel. However, a concrete-faced system is also used regularly and was contractually required where (a) the MSE wall height is greater than 9 m; (b) the MSE wall is visible to highway traffic; or (c) the MSE wall is supporting a bridge structure. The MoT limits the height of concrete-faced MSE walls to 12 m. However, variances were obtained at specific locations for MSE wall heights up to 20m. Both the wire-mesh and concrete-faced panel systems are visible in Figure 23. With respect to the embedded soil reinforcing straps, all MSE retaining wall systems on the corridor use galvanized steel.

Structural Concept Selection – Retaining Wall vs Bridge: In lieu of a downslope retaining wall, a common alternative concept is a half-bridge structure with a median wall to support the fills for the inner highway lanes (Figure 25). The selection between these two options is a function of several site specific factors, most notably: wall height, geotechnical conditions and geometry, equipment access and availability of resources. Many of these factors are contractor-specific or can only be reliably assessed with full site pioneering. For these reasons, a design-build project delivery is often better positioned to make an informed and optimized decision between these structural concept options.

Over the duration of the project, the wall option was increasingly preferred by the contractor for its significant cost and schedule benefits. The reported schedule benefits can be partially attributed to the

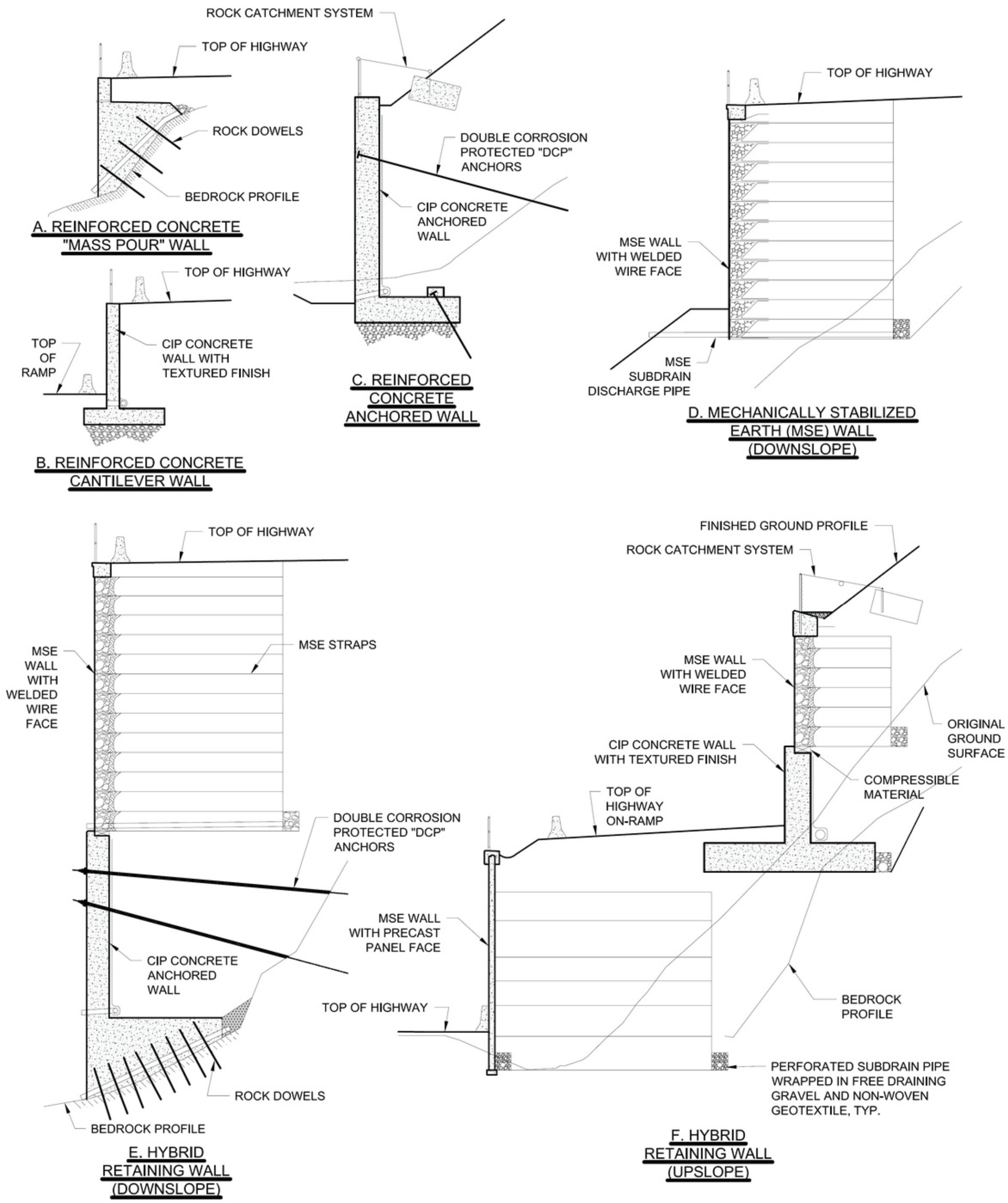

Figure 22 – Typical Retaining Wall Combinations

large-scale roadworks component to the project and the broad availability of equipment and resources suited for both roadworks and MSE wall construction. Furthermore, the surplus of excavated rock material along the corridor was well suited for processing and reuse in MSE wall construction. At project completion, the contractor was reporting that wall heights up to 15 m were competitive, provided the wall was founded on bedrock and global slope stabilization was not a significant issue.

MSE Retaining Wall – Reduced Footprint: As noted, a significant disadvantage to the application of an MSE wall can be its large foundation footprint. This geometry can present both construction complications and environmental impacts due to increased excavation and temporary shoring. This geometry can be reduced for walls founded on rock (or equally competent material) by using uneven reinforcement strap lengths (Ref. 7). For typical cases on the corridor, strap lengths were reduced to approximately 50% of the wall height for the bottom half of the wall, when founded on rock.

Figure 23 - Completed Hybrid Retaining Wall (RW1025)

Retaining Wall Optimization – Hybrid Walls: A second common strategy to minimize the foundation footprint was the construction of an MSE wall on top of a cast-in-place reinforced concrete starter wall or foundation. This option is not only effective in reducing the overall wall footprint and its associated excavation / shoring but can also (a) smooth out an undulating bedrock profile for MSE wall construction; and (b) allow for the use of taller overall retaining walls while still abiding by the MSE height limits specified by the MoT. Numerous hybrid wall configurations were developed to meet the specific requirements at each wall site. Two such examples are shown in configurations E and F in Figure 22 and an as-built photograph of the configuration F is shown in Figure 26.

Figure 24 - MSE Wall Construction (RW993)

For such hybrid walls, the applied earth pressure loadings acting on the concrete starter walls are a function of the geometry and flexibility of the

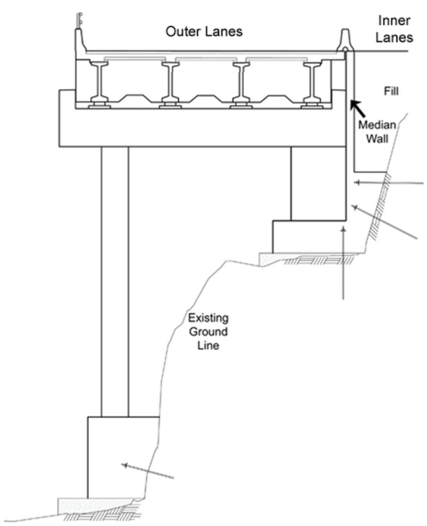

Figure 25 - Typical Half-Bridge Cross-Section (6900 Structure)

Figure 26 - Tiered and Stacked Retaining Wall (RW87)

system's components. The earth pressure coefficient methods provided in the AASHTO Standard Specifications (Ref. 4) are known to be conservative in such applications. As a design optimization measure therefore, the geotechnical consultant prepared a series of design loading curves for various structural configurations, based on rigorous computer-based (FLAC) analyses.

Concluding Comments on the Use of Retaining Walls: The Sea-to-Sky Project has demonstrated that the use of retaining walls in lieu of bridges is increasingly preferred with respect to project cost and schedule. In addition, the project has also shown that custom and refined hybrid wall designs developed at a specific site can reduce the overall wall footprint. By doing so, excavation, shoring and new construction materials were minimized. In this regard, the incentive for reduced construction costs and environmental impacts were fully compatible.

3.6 Culvert Head Walls

Culvert end treatments usually take one of two forms: (a) a culvert extension through the roadside embankment; or (b) a structural headwall at the culvert end, retaining the road embankment fill. A culvert extension is often the preferred option due to its simplicity and lower cost. However roadway geometry and property constraints can preclude its use. In addition, for streams with valued fisheries habitat, a shorter culvert with head walls is considered to have fewer permanent environmental impacts. While culvert head walls can take many different forms, they are most commonly built with cast-in-place or precast concrete.

Each of the existing culverts in the Squamish (DB8) section required replacement and/or an extension to accommodate increased hydraulic flows and highway width. Many of these water courses were classified as having highly productive salmon habitat and therefore, these culvert locations required headwalls. The contractor selected the proprietary Deltalok retaining wall system and the MoT permitted its use in select locations up to 5m in retained height. This wall system is a battered MSE wall that uses polymeric geogrid soil reinforcement and an engineered geotextile soil bag as its facing element (Figure 27). The stacking bag system is relatively simple to install and can be field-customized to suit a range of culvert geometry.

The facing bags are perforated so that dense vegetation grows over the face of the wall (Figure 28). In addition to the environmental benefits of this "green" facing, a dense vegetative covering is required to protect the bag from ultraviolet damage and provide long-term durability for the wall system. The climatic and backfill conditions at each of the Squamish culvert locations were considered well suited to develop this protective vegetative layer. However, there were concerns over the viability of a fully vegetated cover within the riparian zone. This issue was addressed by stacking riprap along the face of the wall within the zone of concern.

Figure 27 - Deltalok Wall Before Vegetation (Dryden Creek Culvert, Inlet Headwall)

Figure 28 - Deltalok Wall After Vegetation (Dryden Creek Culvert, Inlet Headwall)

4. USE OF OPEN GRADED FRICTION COURSE ASPHALT ON BRIDGE DECKS

The Sea to Sky Highway project design criteria called for the placement of an open-graded friction course (OGFC) pavement through the Village of Lions Bay (DB3) and the District of Squamish (DB8). This design requirement was part of a commitment to local residents in populated areas to reduce the noise levels produced by highway vehicle tires.

4.1 OGFC Introduction

OGFC pavement is an asphalt wearing course that, compared to typical dense pavements, has a coarser gradation with little or no fine aggregate and higher air void content. North American usage was initiated in Oregon in the 1930s and it is presently used in over 26 American states (Ref. 8). Due to its poor performance in colder climates and areas with widespread snow plough and tire chain use, OGFC is more commonly used in milder jurisdictions. Notable reasons for the use of OGFC include (a) improved pavement drainage resulting in reduced vehicle hydroplaning; (b) improved vehicle skid resistance; (c) improved wet and nighttime visibility; and (d) noise mitigation. With respect to the reduced noise levels, the California Department of Transportation (Caltrans) <u>Open Graded Friction Course Usage Guide</u> (Ref. 9) specifies that *"test results have indicated that open graded mixes are typically 3-5 dBA quieter than dense graded asphalt concrete pavements*". This document also advises that "*limited studies have indicated that noise levels on OGFC pavements will increase about 1 dBA every 3 years*". For this reason, and until further research is completed in this area, Caltrans is not recommending the use of OGFC exclusively for noise reduction benefits.

4.2 OGFC in BC

Early use of OGFC in BC dates to the 1960s in the Sicamous area and ongoing use in the Municipality of Saanich. In the 1990s, the BC MoT undertook an extensive and detailed in-service study on test sections in Nanaimo, the Coquihalla Highway, Terrace, 150 Mile House and Nakusp (Ref. 10). Amongst the numerous findings, this study concluded that (a) pavement quality was maintained within the still short evaluation period; (b) a 5 dBA noise reduction was achieved and follow-up testing indicated only minor degradation that fell within the margin of error; (c) skid resistance improvement was of up to 20% within the first year of construction; (d) with appropriate pavement detailing and regular maintenance, the OGFC was able to maintain its superior drainage properties without clogging.

4.3 OGFC on Bridge Decks

There is little known research or design guidance presently available on the placement of OGFC on bridge decks. Until more data is available therefore, most jurisdictions discourage its use on bridge decks. For example, the Caltrans Usage Guide (Ref. 9) specifies that, "*OGFC should not be used to overlay a bridge deck without special approval*". Three often cited concerns relating to the use of OGFC on bridge decks include (a) difficulties with handwork; (b) fatigue cracking in tension zones; and (c) drainage. Each of these concerns is described further as follows:

Handwork: The BC MoT study (Ref. 10) advises that "*handwork is quite difficult*" and "*mistakes are difficult to correct*". This finding is echoed by the Oregon Department of Transportation (ODOT) that specifies, "*Open graded wearing courses are not recommended in urban areas with curb and sidewalk or where significant amounts of handwork or feathering will be required during construction*" (Ref. 11). Such handwork is often required on bridge decks, particularly around drains and joints.

Fatigue Cracking: The ODOT advises that *"the open graded mix is not in the tensile zone of the pavement structure. The open graded mix is more susceptible to fatigue cracking due to reduced tensile strength of the mix*" (Ref. 11). On a continuous multi-span bridge, a tension zone in the pavement structure is unavoidable.

Drainage: The Washington State Department of Transportation (WSDOT) continues to investigate the appropriate applications for OGFC after several cases of "*pavement lives of less than ten years, and as short as three to four years"* (Ref. 12). For the interim, the WSDOT Pavement Guide (Ref. 13) recommends the use of OGFC only in applications where placed directly on a free draining base and not as an overlay to a lift of dense asphalt paving. This requirement is also emphasized in the MoT study which advises that, "*Unimpeded edge drainage is critical to the success of OGFC pavements*" (Ref. 10). To provide edge drainage, the MoT report recommended the use of a 100mm wide longitudinal channel along the edge of pavement to collect and direct water from the OGFC layer.

4.4 OGFC Use on Squamish River Bridges

An OGFC top wearing surface was required on each of the three river crossings in the Squamish (DB8) section. The project design criteria required an aggregate gradation as summarized in Table 1 and 18% air void content.

Table 1 – OGFC Aggregate Gradation

Sieve Designation	Percent Passing
16.0 mm	100
12.5 mm	95 – 100
9.5 mm	50 – 70
4.75 mm	15 – 30
2.36 mm	5 – 15
0.075 mm	2 – 5

In light of the findings and recommendations from other jurisdictions, the design considered the pavement detailing to mitigate the known OGFC vulnerabilities. Each of the previously-itemized OGFC issues was addressed for the project and summarized as follows:

Handwork: Current BC practice calls for the elimination of bridge deck joints wherever possible to eliminate a common maintenance issue. This approach had the beneficial consequence of also eliminating difficult OGFC handwork. Similarly, deck drains are discouraged due to both maintenance and potential environmental discharge concerns. For the Squamish river bridges, deck drains were located off the bridge decks in all but two locations.

Figure 29 – Longitudinal Drainage Channel in Front of Concrete Barrier

Fatigue Cracking: Pavement tension zones will form over the piers on the two continuous structures: the Mamquam Blind Channel and the Mamquam River Bridges. In both cases, this issue is partially mitigated by the relatively short span lengths and very stiff, composite concrete cross-section. Without conclusive research data, however, the pavement performance in these areas remains uncertain and these bridges will serve as case-studies.

Drainage: Consistent with the recommendations from the BC MoT study (Ref. 10), each of the bridge decks has positive transverse drainage to a 100 mm wide longitudinal channel located in front of the concrete barrier (Figure 29). The channels discharge longitudinally to either a bridge deck drain or a roadside catch basin beyond the end of the bridge.

5. CLOSURE AND ACKNOWLEDGMENTS

Examples from the Sea-to-Sky Highway Improvement Project have demonstrated that construction incentives can often be compatible with environmental interests. Innovative engineering solutions can lead to the maximum reuse of existing structures, the optimal utilization of construction materials and the significant reduction in structural footprint. Such measures can result in mutual benefits for construction costs and environmental impacts.

There is little known research or documented experience relating to the placement of OGFC pavement on bridge decks. Based on guidance from both BC MoT research and other jurisdictions, design details to extend the pavement design life included considerations for unimpeded pavement drainage and minimized pavement handwork.

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