Deltaport Causeway Overpass: How to Construct a Complex Bridge Overpass on a Narrow Causeway in Challenging Soil Conditions

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ABSTRACT

The Deltaport Causeway Overpass in Vancouver B.C. was the centerpiece of a $45 million upgrade of the transportation infrastructure at Canada's busiest container port terminal. The project included the design and construction of a curved overpass located on a narrow causeway in an area of highly sensitive soils. The aim of the project was to improve vehicle access to the terminal by separating road and railway traffic at a critical bottleneck junction, and to contribute an additional 200,000 container units of annual capacity at the port.

This paper describes the technical challenges and the engineering solutions that were used to design the structures to accommodate the tight geometric constraints of the site while ensuring minimal impact to terminal operations. The key technical challenges included:

1. The design of very slender bridge columns due to the close proximity of the rail tracks. The innovative design used small-diameter reinforced concrete columns with an externally-bonded fiber-reinforced polymer (FRP) wrap. The FRP wrap was designed to confine the concrete core, and ensure the columns had sufficient ductility to meet the structural design capacity;

2. The design of expanded-base concrete 'Franki' piles founded within a zone of stone-column ground improvement. Franki piles were the preferred piling system because the depth to bedrock precluded the installation of deep-pile foundations. The Franki piles were constructed by driving a zero-slump concrete mix out the bottom of a steel casing to form the load-bearing compression and tension bulbs;

3. The design of a state-of-the-art lightweight-fill solution for the bridge-approach embankments using expanded polystyrene (EPS) blocks, aka "geofoam". The lightweight properties of EPS allowed the approaches to be constructed at a relatively shallow depth, and limited the weight applied to the load-sensitive foundation soils.

1 INTRODUCTION

The design and construction of the Deltaport Causeway Overpass was part of a $45 million design-build project for the Port of Vancouver's Deltaport Terminal Road and Rail Improvement Project (DTRRIP). The aim of the DTRRIP project was to upgrade the Port's infrastructure and increase the container handling capacity of the terminal. WSP led the design team and was the prime consultant providing design management, civil and structural engineering design, and construction inspection services.

2 DELTAPORT CAUSEWAY OVERPASS PROJECT

2.1 PROJECT OBJECTIVES

The project objectives were to increase the throughput capacity of the terminal, improve the flow of trucks and trains accessing the port, reduce vehicle idle times, and improve safety. This was to be achieved by constructing a new overpass to separate road and railway traffic at a critical bottleneck junction at the entrance to the port (see Figure 1) with the goal of contributing an additional 200,000 container units of annual capacity at the port.

The project goals were to remove the at-grade rail crossing and replace it with an overpass, and realign the roadways and rail tracks leading to the port. A key project requirement was for road and rail access to remain fully functional throughout construction.

2.2 PROJECT DESCRIPTION

The DTRRIP project included the design and construction of:

- A bridge overpass spanning over the rail tracks
- Three expanded polystyrene (EPS) filled approaches enclosed by precast concrete facing panels
- Roadway upgrades and realignments
- Rail track relocations
- New and relocated utilities

Figure 1: Deltaport Causeway Overpass, Vancouver, BC

There were multiple stakeholders involved in the project; including the project owners, Port of Vancouver and the BC Ministry of Transportation and Infrastructure. As well as the two terminal operators, Terminal Systems and Westshore Terminals, and three rail companies, Canadian Pacific Railway, Canadian National Railway, and British Columbia Railway.

2.3 PROJECT GEOLOGY

Deltaport Terminal is located on a 150 m wide man-made causeway in the Fraser River Delta approximately 60 km south of downtown Vancouver (see Figure 2). The causeway was constructed in the 1960s over intertidal mudflats using sand dredged from the seabed. The subsoil underlying the causeway is loose to compact native river sand to a depth of about 50 m, which was assessed to be potentially liquefiable under the design-level earthquake. Below the sand is clayey silt to a depth of about 100 m where competent till-like soil is present.

The key geotechnical considerations for the project were to (a) minimize long-term ground settlements, (b) ensure adequate seismic performance, and (c) accommodate the construction schedule and staging requirements. The general design approach was to locate foundation elements as shallow as possible and to employ structural configurations that were resilient to the anticipated ground settlements. The use of shallow foundations kept the piles and footings above the potentially liquefiable subsoils and reduced the potential for differential settlements between the bridge and approach embankments.

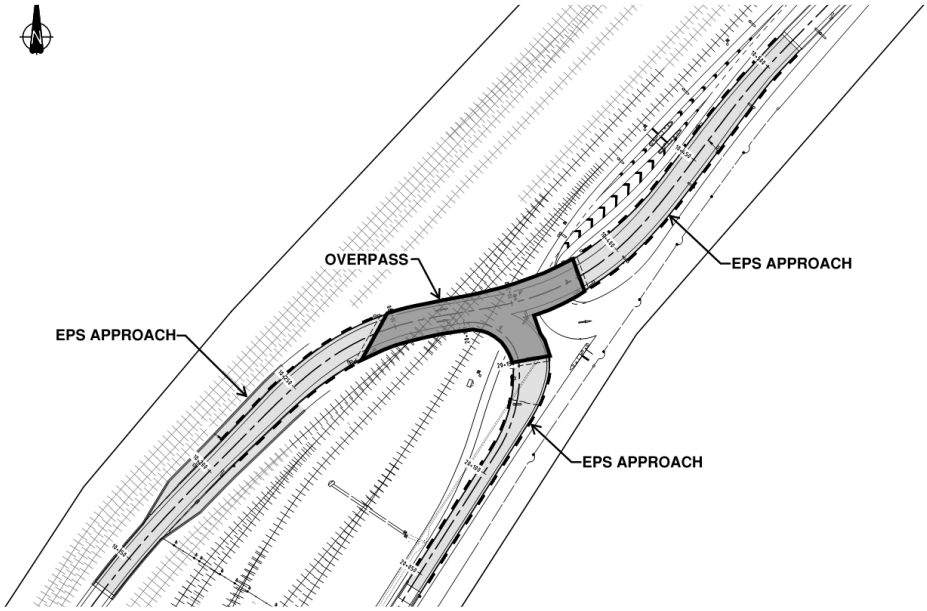

Figure 2: Deltaport Causeway Overpass Plan

2.4 SITE SEISMICITY

Liquefaction of the sand underlying the causeway was expected for the 1 in 475-year earthquake, and site-specific analysis was carried out to develop the response spectra for the site. Several ground motions were used for the site-specific response analysis, including the 1971 San Fernando, 1989 Loma Prieta, and

1949 Western Washington Earthquakes. From these records, the response spectra was developed with the peak ground acceleration estimated to be 0.6g.

To estimate the lateral ground deformations under the 475-year earthquake, numerical analyses were carried out using the finite-difference software program FLAC (Fast Lagrangian Analyses of Continua). The behaviour of the sand under dynamic loading was simulated using the 'UBC-Sand' model. This model, which was developed at the University of British Columbia, was capable of simulating the stress-strain behaviour of potentially liquefiable sand subjected to seismic loading. The model also included a prediction for the triggering of liquefaction.

2.5 SITE CONSTRAINTS

The project site was located on a narrow man-made causeway at the entrance to the port with sensitive marine habitats on both sides. The causeway provided access for both rail and road traffic and was a highly congested area with multiple rail lines, several traffic lanes, and numerous maintenance access roads (see Figure 3)

The new overpass was designed to fit between the realigned roadways and the new rail lines. Consequently, the available space for the new structures was severely constrained, and the resulting geometry of the overpass was extremely complex. The main span of the overpass was located on an 'S' curve section of the roadway with compound-curves and transitioning superelevation. The overpass was also designed to accommodate the bifurcating roadway alignment for an off-ramp located at mid-span of the structure (see Figure 3).

Figure 3: Aerial View (Looking West)

3 DESIGN INNOVATIONS

3.1 CONCRETE BRIDGE COLUMNS WITH FIBER-REINFORCED POLYMER WRAP

The bridge columns supporting the main span of the overpass were designed as slender reinforced concrete sections. The columns were 500 mm (20") in diameter with an externally-bonded fiber-reinforced polymer (FRP) wrap. The FRP wrap provided confinement for the concrete core, and was designed to ensure the columns had sufficient ductility to meet the structural design capacity.

The FRP wrap was 'Tyfo SEH-51A' by Fyfe Company, a glass fiber-reinforced fabric mesh bonded to the concrete column using an epoxy paste (See Figure 4). The principal tensile fibers, which were oriented transverse to the column's vertical axis, provided additional confinement for the concrete column – functioning in a similar way to that of the spiral reinforcing bars. Each bridge column was wrapped with two layers of FRP (12 columns in total).

The design approach for the bridge substructure elements was based on the seismic design principle of ductile columns and capacity-protected foundations. Under earthquake loading, the columns were designed to form plastic hinges at the top and bottom of the columns in order to dissipate energy and to limit the force transferred into the pile foundations. The piles themselves were designed for the maximum force that resulted from the plastic overstrength capacity of the column hinges. This design approach ensured that any damage to the columns occurred in an easily-accessible location, and, if required, could be readily repaired after an earthquake.

Figure 4: Application of FRP Wrap to Bridge Columns

The bridge columns were supported on a reinforced concrete crash wall which was 760 mm (2'6") thick. The crash wall was designed to protect the overpass from a rail car tipping over due to a low-speed derailment. The railway design guideline, AREMA, specified the minimum crash wall thickness of 760 mm, and also stipulated that any increase in column diameter would require a corresponding increase in the crash wall thickness. This was a fundamental constraint for the design because the close proximity of the rail envelope to the crash wall dictated that the maximum allowable diameter for the bridge columns was 500 mm (see Figure 5).

During the design phase several design options were considered for the columns, including (a) encasing the concrete columns in a permanent steel casing, (b) reducing the concrete cover to the reinforcing bars, and (c) using larger-diameter concrete columns. Each of these options, however, had its drawbacks and the reinforced concrete column option with FRP wrap was ultimately selected because it was the most advantageous from a design perspective and it was the most cost effective.

Figure 5: Bridge Columns and Crash Walls

The permanent steel casing was not a viable option because it would have over-stiffened the columns. The columns were designed to form plastic hinges and to remain relatively flexible during the design-level earthquake. The use of a steel casing would have prevented plastic hinges from forming and resulted in a significant increase in the number of foundation piles.

Reducing the concrete cover to the reinforcing bars in the columns was also considered. The design team examined the possibility of using stainless steel reinforcing bars with reduced concrete cover and a protective coating on the columns. However, the use of stainless steel bars in the plastic hinge zone would have compromised the column ductility, and the reduced cover did not comply with the Project Agreement.

The use of larger-diameter columns was also considered, however, this was not a viable option given the geometric constraints of the site and the limitations on the crash wall thickness. Although increasing the column diameter seemed at first to be the most obvious solution to the design team, the limitations imposed by the proximity of the rail tracks made it clear that using larger-diameter columns was not a feasible solution.

As such, the concrete column option with an FRP wrap was deemed the most suitable. The FRP wrap was applied to the columns after they had cured, and all told, construction of the wrap and the protective coating was completed in less than one week. The FRP wrap proved to be a simple and cost effective solution that ensured the columns had sufficient strength and ductility to meet the structural design capacity (see Figure 6).

Figure 6: Bridge Columns with FRP Wrap

3.2 FRANKI PILES

The main span of the bridge was founded on concrete 'Franki' piles. The Franki pile system was first developed in the early 1900s by Belgian engineer Edgard Frankignoul and utilizes a concrete bulb attached to the base of a vertical shaft. The depth to bedrock at the Deltaport site precluded the use of deep-pile foundations, as such, the relatively-shallow Franki piles were the preferred piling system.

The 9 m long Franki piles were constructed by driving a zero-slump concrete mix out the bottom of a steel casing to form the load-bearing compression and tension bulbs (See Figure 7). This type of piling system was well suited to the site given that the subsoils were comprised primarily of sand – which allowed for easy formation of the bulbs.

The Franki piles were founded within a 15 m deep zone of stone-column ground improvement. The stone-columns comprised a grid pattern of equally-spaced vertical columns consisting of crushed aggregate of various sizes. The stone-columns were designed to strengthen the soil surrounding the piles and to provide the piles with the required vertical and lateral support under both service and earthquake loads (See Figures 8 and 9).

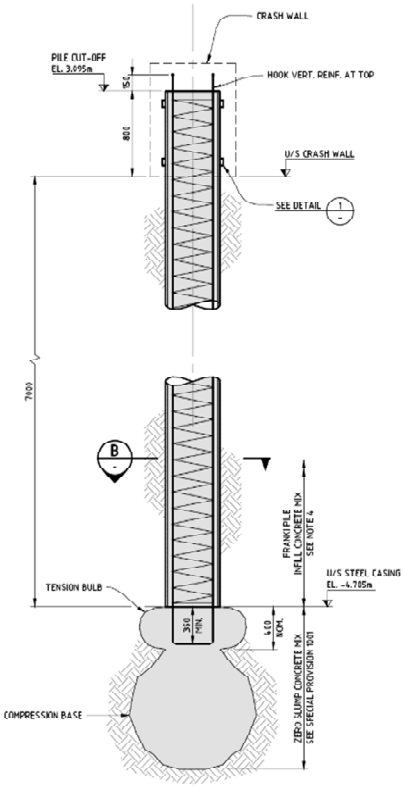

Figure 7: Franki Pile

The interior piles at each pier were designed as typical Franki piles with a single compression bulb. The two outermost Franki piles were designed with an additional tension bulb to resist uplift forces during

the seismic event. The tension bulbs were formed after the compression bulbs had been constructed by pulling the steel casing upwards while driving the zero slump concrete mix out the base. The tension bulbs were formed with vertical reinforcing bars extending into the concrete mass to provide connectivity with the pile shaft.

The Franki piles were analyzed as a beam element with the soil structure interaction modelled by assigning soil 'p-y' springs along the length of the pile shaft. The top of the pile was cast into the reinforced concrete crash wall and this allowed the piles to be modelled as a fixed-headed piles. The column forces were applied to the model as a series of point loads. The final Franki pile design was a composite steel and concrete section where the steel casing was included in the structural capacity of the pile system.

The Franki piles were installed on site without any major complications, and the overall piling system, in conjunction with the ground improvement, resulted in significant project savings. If traditional bridge piles had been used at the Deltaport site, such as driven steel pipe piles or drilled shafts, they would have been installed to a much greater depth at a much higher cost.

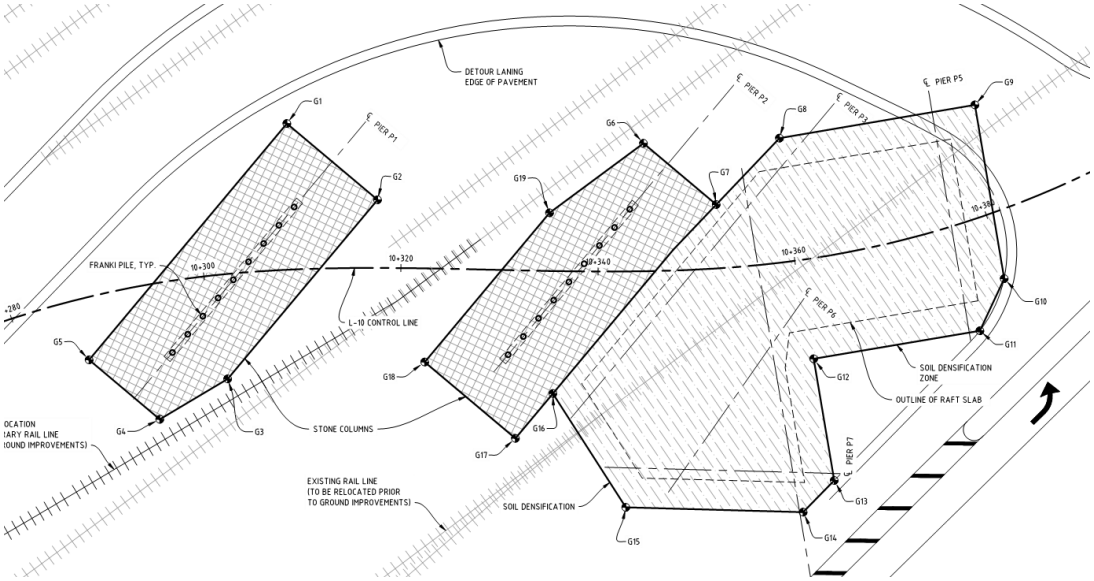

Figure 8: Ground Improvement Plan

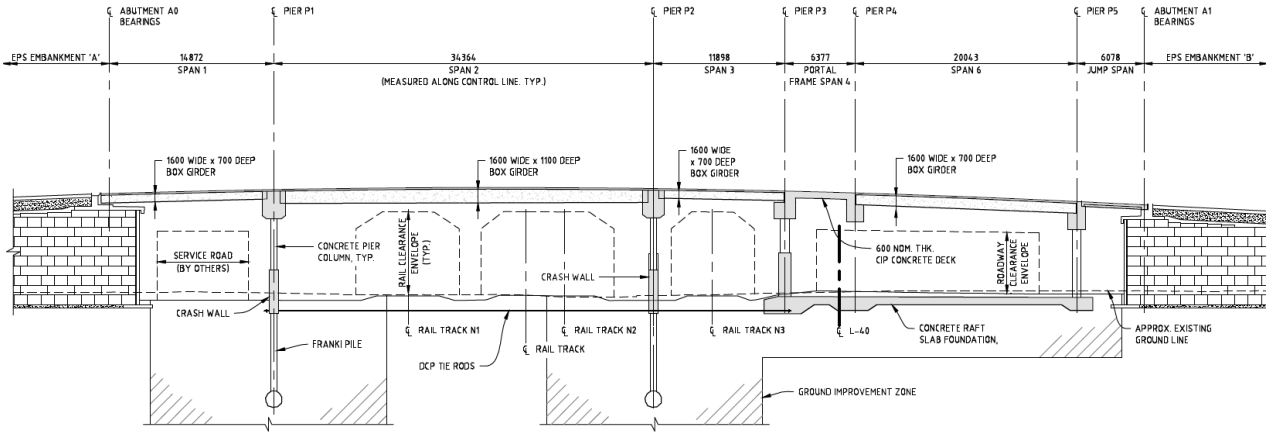

Figure 9: Ground Improvement Elevation

3.3 EXPANDED POLYSTYRENE APPROACH EMBANKMENTS

The three bridge approaches were constructed using lightweight-fill expanded polystyrene (EPS) blocks, aka "geofoam" (see Figure 10). The use of EPS allowed the embankments to be constructed at a relatively shallow depth and limited the weight applied to the load-sensitive foundation soils. The lightweight-fill approaches weighed only one-seventh of a traditional granular-fill approach, and were a more cost-effective solution than lengthening the bridge or constructing large areas of ground improvement.

EPS was used to construct the approaches at Deltaport for two main reasons:

1. To limit the load increase on the foundation soils and therefore prevent excessive long-term ground settlements.
2. To reduce the overall weight of the approach fills and therefore limit the inertial seismic loading on the foundations soils.

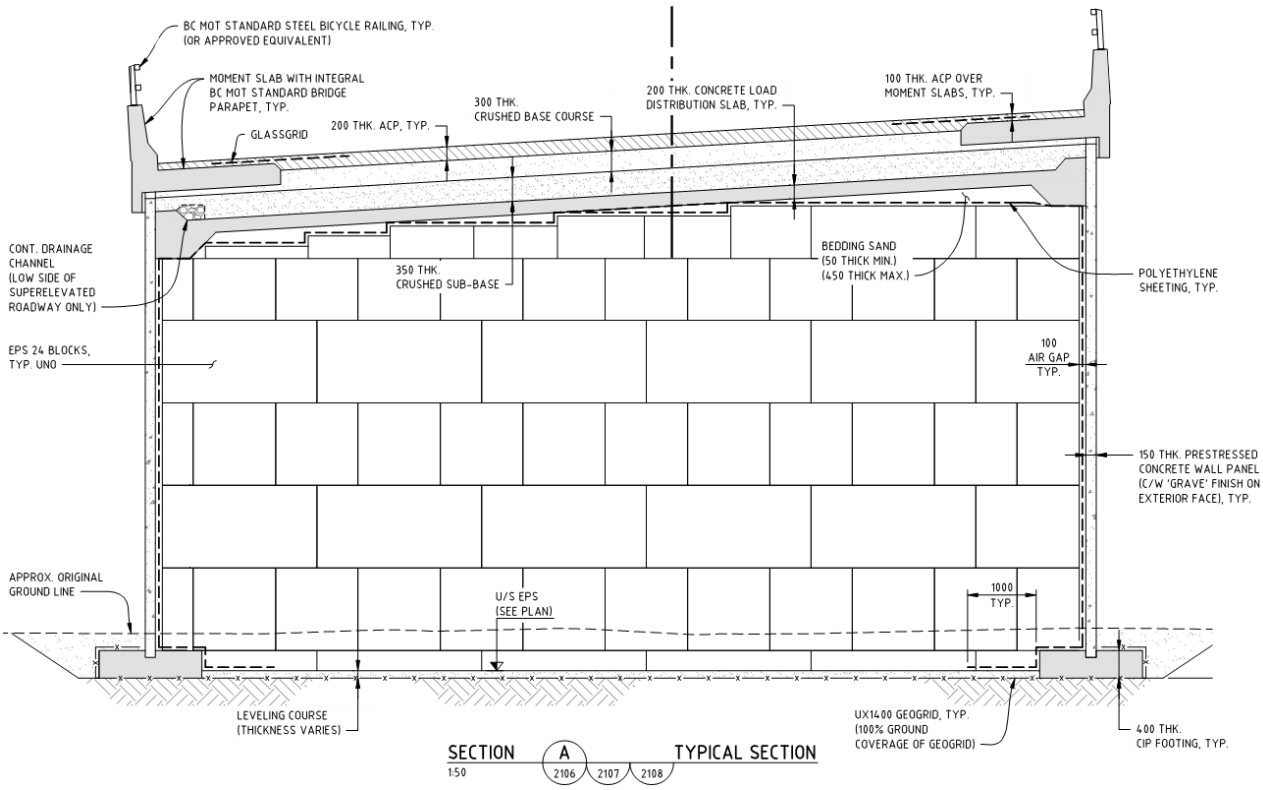

Figure 10: EPS Embankment Cross Section

Constructing traditional granular-fill approaches at the Deltaport site was not a viable option. The weight of the fills would have caused excessive ground settlements and there was insufficient time in the construction schedule to allow for pre-loading of the foundation soils. Furthermore, any settlement of the ground would have affected the rail lines which were located directly adjacent to the approaches.

The use of EPS has been growing rapidly in recent years with the increase in design-build projects and compressed construction schedules. Expanded polystyrene is a lightweight, closed-cell, rigid-plastic foam, comprised of 98% air and 2% polystyrene. EPS blocks are approximately 100 times lighter than soil and therefore offer several benefits over constructing traditional approaches.

The core of the EPS embankments comprised EPS 24 (EPS density of 24 kg/m^3). EPS 29 was used locally to support the bridge approach slab, and EPS 39 was used in locations where the blocks were subjected to higher loading conditions (see Table 1).

Table 1: EPS Material Properties

Material Property	Units	EPS 24	EPS 29	EPS 39
Density	kg/m^3 (pcf)	24.0 (1.50)	28.8 (1.80)	38.4 (2.40)
Compression Resistance (at 1% Strain)	kPa (psi)	66 (9.4)	75 (10.9)	103 (15.0)
Compressive Resistance (at 5% Strain)	kPa (psi)	140 (20.0)	170 (24.7)	241 (35.0)

The primary reference for the design of the EPS blocks was the DTRRIP Project Agreement. This document outlined the minimum design requirements and the material specifications for the EPS blocks. The design of the overall EPS embankment system was based on NCHRP Report 529 'Guidelines and Recommended Standards for Geofoam Applications in Highway Embankments'. The design of structural components such as the facing panels, concrete load distribution slab, and concrete roadside barriers, was carried out in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06.

One of the main criteria to be satisfied in the design of EPS embankments is the control of long-term ground settlements. The design approach adopted for the project was to limit the load increase on the foundation soils to less than 20 kPa. This was achieved by excavating the existing ground by a depth sufficient to offset the total weight of the embankment.

Figure 11: Exterior View of EPS Embankment and Span 6

The EPS pavement system placed over top of the EPS blocks was 1.3 m thick, and comprised asphalt concrete pavement (200 mm thick), crushed base course and sub-base (650 mm thick), concrete load-distribution slab (200 mm), and bedding sand (250 mm nominal thickness). The primary objective of the EPS pavement was to provide a durable, fully-bound pavement system that was capable of resisting long-

term traffic load-cycles and to disperse the truck wheel loads sufficiently to reduce the localized stress on the EPS blocks.

The sides of the EPS approaches were covered with precast concrete facing panels to protect the EPS blocks from fire and physical damage (see Figures 11, 12, and 13). The facing panels were 150 mm thick prestressed concrete panels ranging in height up to a maximum of 11 m. The typical panel was 3 m wide, and was cast with a textured finish to provide aesthetic relief. Prestressed panels were preferred over reinforced concrete panels for ease of construction and to reduce the chances of the panels cracking during transportation and erection.

The facing panels were designed in accordance with CSA A23.3, Design of Concrete Structures, Chapter 23 'Tilt–Up Concrete Wall Panels'. There is no design code available in North America specifically for the design of facing panels for EPS approaches, so the design principles for tilt up panels used in building construction were adopted for the design of the EPS panels. Tilt up building panels are typically used as load bearing wall elements that span vertically from the floor slab to the roof. Similarly, the EPS facing panels were designed to span vertically from the footings to the load distribution slab at the roadway level. The loads on the EPS facing panels were also similar in nature to those of tilt up panels, namely out-of-plane transverse loads, wind and seismic, in conjunction with a nominal axial load applied through the vertical axis of the panel.

The 'moment magnifier' method of analysis recommended in CSA A23.3 was used to design the panels for the combined effects of out-of-plane lateral loads and axial load, and to evaluate the panels for the secondary moments generated by the P-Δ effects. The panels were designed as simply supported flexural elements; the transverse load on the panels was from wind and seismic, and the axial load was from panel self weight and the dead load of the moment slab.

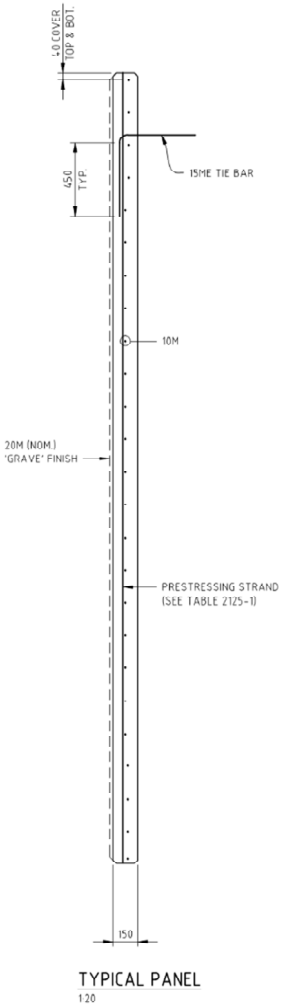

Figure 12: Typical EPS Facing Panel

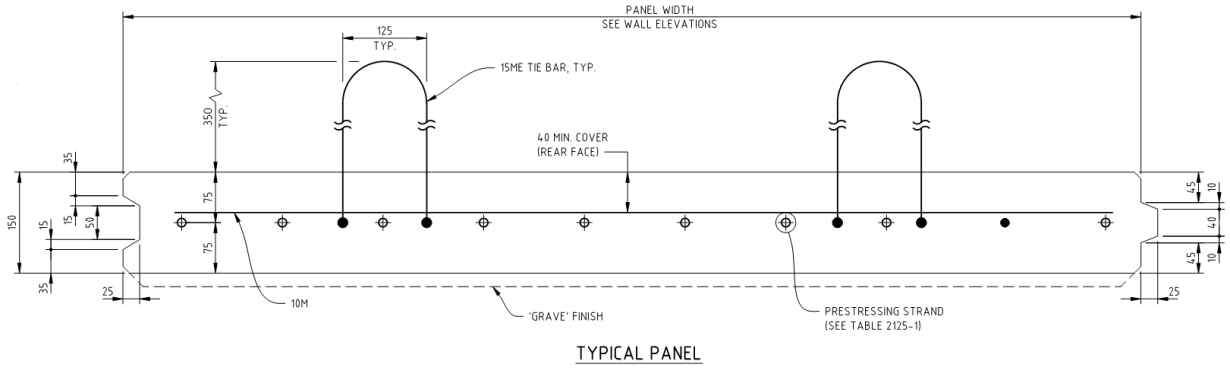

Figure 13: Typical EPS Facing Panel Cross Section

To protect the EPS blocks from fire, the facing panels were designed with a 2-hour fire rating. At joint locations, a shear key was cast into the ends of the panels and a fire-rated ceramic fiber blanket was installed in the joint (see Figure 14).

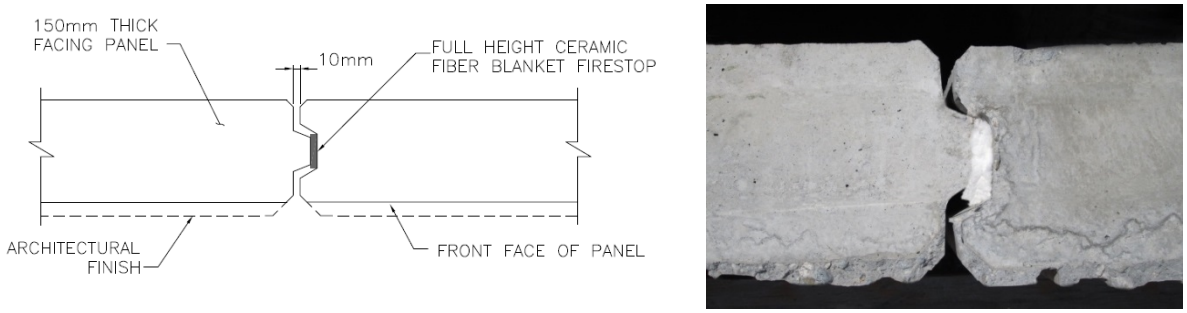

Figure 14: EPS Facing Panels Joint Detail

Under earthquake loading the EPS embankments were designed to respond as a uniform 'block'. This design approach was used to ensure that the integrity of the structure was not compromised by the expected horizontal and vertical ground deformations during the seismic event.

For the EPS approaches to respond as a uniform block, including the facing panels and footings, a tensile connection was provided at the top and bottom of the embankment. This was achieved by connecting the concrete load-distribution slab to the facing panels at the top of the embankment, and by tying the concrete footings together at the base. The top connection was made by connecting the load-distribution slab to the facing panels via hooked reinforcing bars projecting from the panels (see Figure 15). The bottom connection was achieved by installing uniaxial HDPE-geogrid between the concrete footings (see Figure 10). The EPS blocks formed the core of the uniform block, and the facing panels provided the tensile capacity on the side walls.

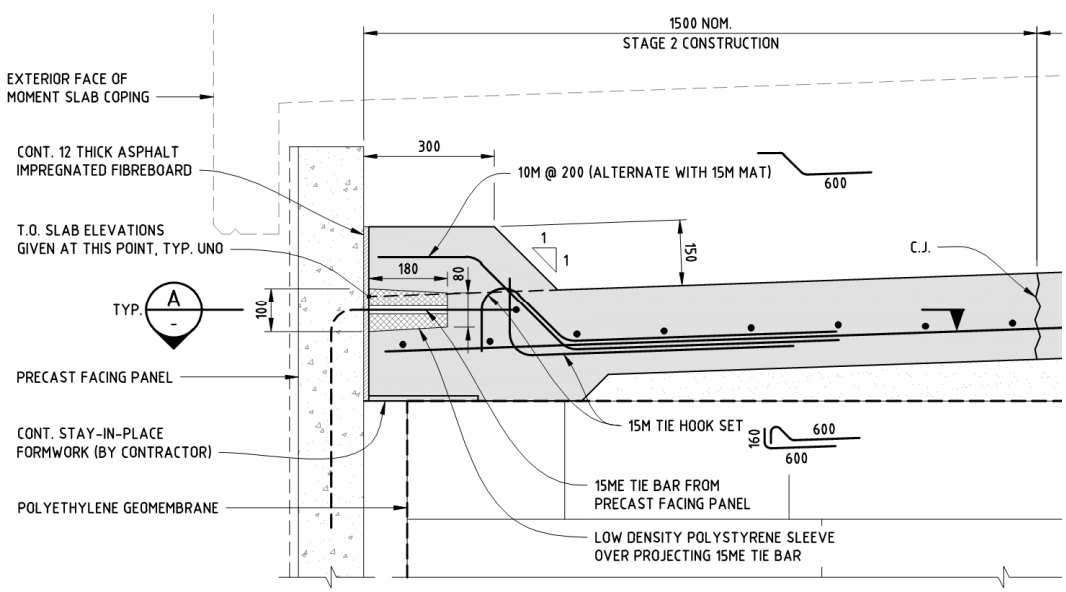

Figure 15: Concrete Load-Distribution Slab Connection

3.4 GEOMETRIC MODELLING USING AUTOCAD CIVIL 3D

The roadway geometry for this project was extremely complex due to the tight property constraints and the proximity of the existing port infrastructure. AutoCAD Civil 3D software was used to define and control the geometry for the layout of the new structures. The Civil 3D modelling capability saved countless design hours and was an accurate and highly efficient tool for controlling the geometry. The software features of Civil 3D enabled a detailed and dynamic model of the structural elements to be developed, which included piles, columns, cap beams, girders and bridge deck. Figure 16 below shows a simplified screen capture of the model.

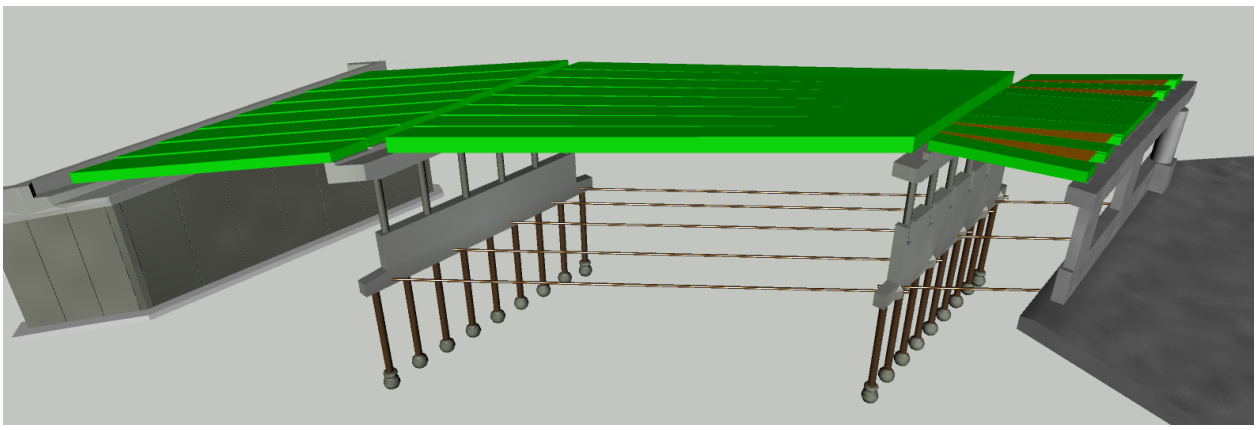

Figure 16: AutoCAD Civil 3D Model (simplified for visual purposes)

The compound curvature and the bifurcating roadway alignments resulted in a complex finished grade surface. The Civil 3D 'Corridor' feature was used to develop the roadway surface which had a high degree of superelevation transition at several locations. The Corridor feature was also used to generate the roadway surface in the area between alignments where the roadway geometry information was insufficient for structural design. For drawing production, the bridge deck elevations were defined using Civil 3D's 'Surface' feature which was dynamically linked to the elevation tables on the design drawings.

The bifurcating roadway alignment was located directly over the rail tracks and this required the box-girders to be splayed in plan (see Figure 17). The geometry was further complicated by the transitioning superelevation which varied over the width of the bridge. This resulted in the box girders being laid out in three planar zones; each zone having a unique horizontal and vertical geometry with a different superelevation.

Due to the splay, the box girders varied in length within each zone, ranging in length from 8 to 13 m. The girders in the middle zone had a constant longitudinal grade, however, all other girders had a unique longitudinal grade. This resulted in girder reinforcing details that were specific to each zone.

The ends of the box girders were also skewed relative to the piers. An innovative bearing arrangement was used whereby the ends of the girders on the upslope side were supported on two bearings, and the ends of the girders on the downslope side were supported on a single larger bearing. By using three bearings to support each girder it ensured that the girders beared evenly on the pier caps.

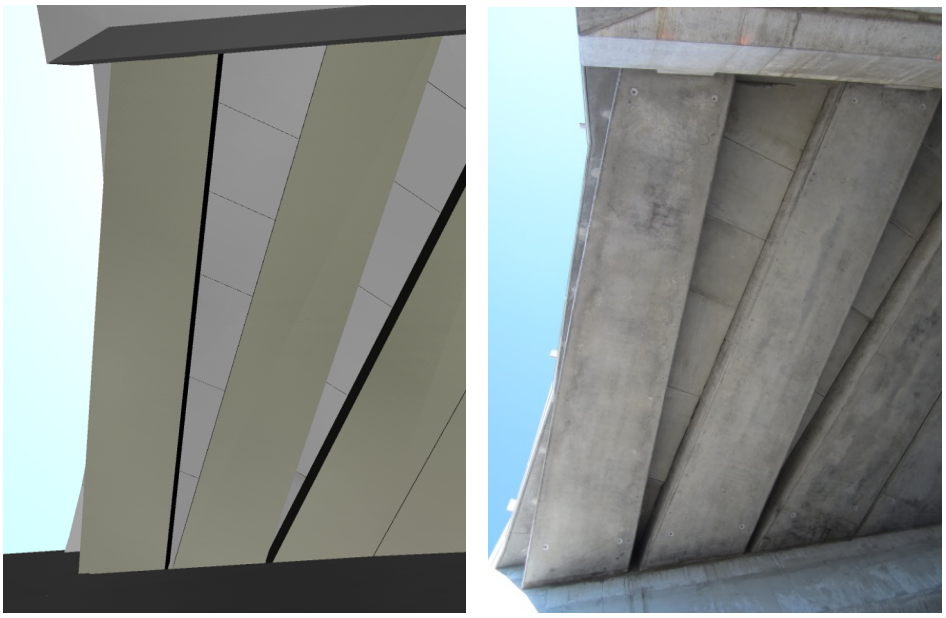

Figure 17: Underside of Bridge Deck Showing the Splayed Box Girders. AutoCAD Civil 3D Model (on the Left) and the As-Constructed Bridge (on the Right)

A consequence of using splayed girders was that the gap between each box girder was a trapezoidal shape. Precast concrete deck panels were used to span between the splayed girders and each panel had unique dimensions to accommodate the trapezoidal shape. A cast-in-place concrete topping was placed on top of the panels and girders to complete the concrete deck. Civil 3D's 'Grading Feature Lines' was used to generate the planar surfaces of the box girders, and the 'Volume Surfaces' feature was used to check the deck thickness. These features were used simultaneously to configure the box-girders, simplify the precast panel layout, and optimize the bridge deck thickness.

The portal frame structure was a complex, highly-interconnected system of cap beams with a cast-in-place concrete deck slab to support the roadway (see Figure 18). The cap beams were modelled in Civil 3D to provide elevation control at critical locations in order to prevent conflicts at the intersection points. The Civil 3D 'Profiles' feature was used to visually examine all intersecting elements and extract the necessary elevations. The soffit of the deck slab was developed as a planar surface using Civil 3D's Grading Feature Lines and Volume Surfaces. The planar soffit greatly reduced the geometric complexity of the deck slab and simplified its construction.

In addition to controlling the bridge geometry, Civil 3D was also used to control the geometry of the EPS approach embankments. The underside elevation of the EPS blocks was determined by excavating the existing ground by a depth sufficient to offset the total weight of the embankment. As such, a comparison was required between the finished roadway and the existing ground surface. Given the large quantity of EPS that was required for the approaches, it was critical to accurately model the EPS volume to optimize the design. Using Civil 3D's 'Volume Surface Comparison', the proposed founding elevation of the EPS blocks was readily compared to the existing ground surface to optimize the total volume of EPS. Civil 3D was an efficient tool for this calculation, and was more accurate and efficient than inputting survey points by hand into an Excel spreadsheet.

Figure 18: Comparison of the AutoCAD Civil 3D Model and the As-Constructed Bridge

The AutoCAD Civil 3D model was also provided to the Contractor to assist with construction set out and to identify constructability conflicts. Overall, the Civil 3D model greatly simplified the geometric layout of the structures during the design phase, and was a useful checking tool for the Contractor during construction.

4 ENVIRONMENTAL AND SOCIAL SUSTAINABILITY

The Deltaport terminal was located in a highly sensitive environmental setting. The terminal itself, and the causeway, were surrounded by sensitive marine habitats and highly productive agricultural lands. The newly completed overpass now provides a more efficient flow of rail and road traffic to and from the port. The project benefits include reducing congestion, wait times, emissions, vehicle idling, noise, and traffic, thereby leading to lower greenhouse gas emissions.

The design team was cognizant of the sensitive marine habitats surrounding the project site, and despite the congested and constrained site, all efforts were made to avoid any impacts to these areas with the design of the permanent works and throughout the duration of construction.

The EPS blocks used in the construction of the approaches were installed using lightweight tools and machinery. Two people were able to install a single block and much of the EPS on the Deltaport Project was installed in this way (see Figure 19). This reduced overall construction costs and reduced the environmental impacts of heavy earth-moving equipment that is typically required to build traditional granular-fill approaches.

The EPS embankments reduced the length of the bridge structure that would otherwise have been required. Life-cycle assessment studies of EPS embankments indicate that for low-height approaches, such as the ones used on this project, the overall carbon footprint is lower than that for an equivalent bridge span.

Figure 19: Installation of EPS Blocks

Another environmental protection measure that was employed during construction was the use of drainage percolation pits for the stormwater runoff. The drainage pits allowed all stormwater to be fully contained within the project site which precluded the requirement for ocean outfalls.

5 ACKNOWLEDGEMENTS

The Deltaport Terminal Rod and Rail Improvement Project was owned and managed by the Port of Vancouver. Construction was undertaken by a joint venture between Dragados Canada and Jacob Brothers Construction. All civil and structural design was carried out in-house by WSP, supported by specialist sub-consultants, Thurber Engineering for geotechnical engineering, and PBX Engineering for electrical engineering. The authors wish to thank the entire project team for their contribution to the success of the project.

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