# Gravina Access Project

## *Preliminary Bridge Structures Technical Memorandum*

## *Draft*

**Agreement 36893013**

**DOT&PF Project 67698**
**Federal Project ACHP-0922(5)**

**Prepared for:**

**State of Alaska**
**Department of Transportation and Public Facilities**
**6860 Glacier Highway**
**Juneau, Alaska 99801**

**Prepared by:**

**HDR Alaska, Inc.**
**712 West 12th St.**
**Juneau, AK 99801**

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# 1.0 Introduction

## 1.1 Scope

The Gravina Access Project was commissioned in 1999 by ADOT&PF to study ways to improve access from Revillagigedo Island to Gravina Island. Presently, ferries operated by the Ketchikan Gateway Borough serving Ketchikan International Airport provide access to Gravina Island. The scope of the Gravina Access Project included review of all of the previous studies, identifying and assessing additional potential modes and alternative alignments for improving access. A draft environmental impact statement will present the results of the alternatives development and impacts analysis.

This technical memorandum presents a review of different bridge types considered for the current Gravina Access Project hard link alignments. The memorandum also describes some of the engineering features of the various bridge types. It concludes with comparisons of the engineering features of the feasible bridge types, including cost comparisons based on deck area unit costs, and recommendations for further, more detailed study for a limited number of alternative bridge types. Based upon this information, it was determined that the post-tensioned concrete box girder bridge type is appropriate for consideration at all current hard link alignments. For purposes of comparing alternatives, this bridge type was developed to enough detail to establish conceptual cost estimates for each alignment. For more detailed quantity and cost estimates by alternative, please refer to *Preliminary Quantities and Cost Estimate Technical Memorandum (HDR, November 2001).*

If the recommended alternative to improve access to Gravina Island is a hard link, further studies will be necessary to determine the appropriate bridge type. These studies are identified within this technical memorandum. The result of the further studies will be a Major Bridge Type Selection Report which will document the conceptual bridge design and cost analysis.

## 1.2 Background

All of the sites identified in previous studies as potential crossing sites, as well as any other sites identified through public and agency scoping, were studied. The previous studies are *Tongass Narrows Crossing Phase I Site Selection*, EMPS-Sverdrup, 1981; *Gravina Road Corridor/Airport – Hard Link*, PEI, 1989; and *Ketchikan Alaska Tongass Narrows Crossing Preliminary Draft Environmental Impact Statement*, Montgomery Watson, 1994.
Eighteen crossing options were identified, of which 12 were bridges. They included three crossing types: bridge, tunnel, and ferry. Each site that appeared to be a candidate had a preliminary horizontal and vertical alignment developed to identify connection points (logical termini) to the transportation system on the Revillagigedo Island side of Tongass Narrows and to Ketchikan International Airport (Airport) on Gravina Island. Based on these alignments, a preliminary footprint was established to evaluate technical feasibility, environmental considerations, and conceptual costs.

## 1.3 Site Conditions

Ketchikan is located in southeastern Alaska at the extreme southern tip of the Alaska panhandle in the Alexander Archipelago. This coastal region of Alaska is accessible only by air and water and has a wet marine environment, with one of the highest annual rainfalls in Alaska. Figure 1 presents a vicinity map of the project area. Tongass Narrows separates the City of Ketchikan on Revillagigedo Island and Gravina Island where Ketchikan International Airport is located. Ferry service provides access to the airport. Except for the airport, Gravina Island is largely undeveloped and covered with lush vegetation.

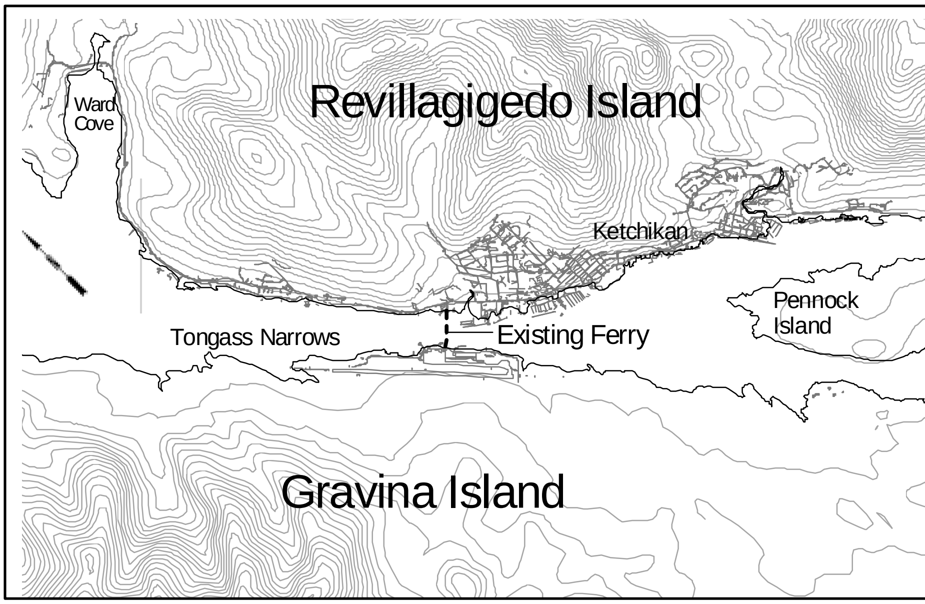


**Figure 1 – Vicinity Map**

**Scale 1:100,000**

Tongass Narrows is a long, narrow water body that is oriented northwest southeast and is approximately 19 kilometers (km) (11 miles) long in the study area. The channel width varies from about 460 meters (m) (1,600 feet [ft]) in the vicinity of the Airport to 2,000 m (6,500 ft) near Refuge Cove (0.5 km (0.3 mile) north of Ward Cove) and at the northern end of Pennock Island. Typical water depths in the channel at mean lower low water (MLLW) range from 25 to 60 m (80 to 200 ft) between Refuge Cove and Gravina Island. Marine currents within the Narrows generally trend from the southeast to the northwest during flood tides and some weak ebb tides, and reverse during strong ebb tides. The velocity ranges from less than 0.5 km/hour (0.3 miles/hour) to about 2.7 km/hour (1.6 miles/hour).

To the south, between Gravina Island and Revillagigedo Island is Pennock Island. Pennock Island is approximately 1 to 2 km (½ to 1 mile) wide by 5 km (3 miles) long and separates Tongass Narrows into the East Channel and the West Channel. Access to Pennock Island is by private boat or floatplane. The West Channel (between Gravina Island and Pennock Island) varies from 300 to 600 m (1,000 to 2,000 ft) in width; the East Channel (between Pennock Island and Revillagigedo Island) varies from 450 to 800 m (1,500 to 2,600 ft) in width. Typical water depths at mean lower low water (MLLW) range from 15 to 60 m (50 to 200 ft) in the West Channel, and 20 to 45 m (65 to 150 ft) in the East Channel.

## 1.4 Geography

Local topography consists of steep mountains plunging into Tongass Narrows. Elevations of the mountains reach about 300 m (1,000 ft) within 800 m (2,600 ft) of Tongass Narrows, and near-vertical cliffs exist along much of the coast of Revillagigedo Island. Because of the steepness of the mountains near the shoreline, much of the city is restricted to the corridor along the coast. Additional development exists to the south in the town of Saxman and community of Mountain Point, and to the north in Ward Cove and beyond.

Tongass Narrows in part is a glacially scoured fjord. Bathymetric contours indicate a relatively flat floor, with water depths ranging from about 30 to 60 m (100 to 200 ft). The diurnal tidal range is about 6.22 m (20.4 ft). The mean higher high tide in Tongass Narrows is 4.7 m (15.4 ft) and the lowest tide is -1.52 m (-5.0 ft).

The shoreline varies from beach type deposits of alluvial silt and sand to steep rocky areas. Much of the coastline on Revillagigedo and Pennock Islands is rocky. Gravina Island's eastern side has several silt and sand coastal areas. Within Tongass Narrows, the bottom conditions range from muddy substrate to rocky pinnacles.

## 1.5 Meteorology

The climate is predominantly cool maritime. The area experiences mild winters, cool summers, and heavy precipitation. Average annual precipitation is about 386 centimeters (cm) (152 inches [in]). Strong winds are common, especially in winter, and cloud cover is persistent. Average annual temperature is about 8 degrees Celsius (46 degrees Fahrenheit), with a mean January temperature of about 2 degrees Celsius (35 degrees Fahrenheit) and a mean August temperature of almost 15 degrees Celsius (59 degrees Fahrenheit). The area is a temperate rainforest. Vegetation is heavy and dense, consisting of western hemlock, Sitka spruce, and Alaska red cedar. The tree line is about 450 to 600 m (1,500 to 2,000 ft) above sea level, with sedges, mosses, and alpine forbs and shrubs at higher elevations. Many areas on lower slopes are subject to rapid surface runoff or spring seepage, and in valley bottoms the surfaces are covered with mosses, sedges, and other plants typical of muskegs.

## 1.6 Geotechnical

A preliminary geotechnical report, consisting of a review of existing literature and limited geotechnical engineering analysis, was compiled by Shannon and Wilson (May 2000). The purpose of the literature review was to identify the likely subsurface conditions at the potential bridge locations. The preliminary geotechnical analyses were then conducted to determine conceptual foundation recommendations. Potential channel cross-sections were developed to present a rough estimate of the anticipated depth and possible subsurface conditions that could be encountered in the various water crossing corridors. The cross sections show one of three conditions: 1) a thin veneer of sediments over bedrock, 2) a thick layer of glacial drift material overlying bedrock, 3) loose sands overlying glacial drift and bedrock. Generally, bridge foundations are envisioned to be drilled piers socketed into the bedrock.

The literature search also identified values of peak ground acceleration between 0.025g and 0.20g for the project area. A recent design for the Ketchikan Federal Building used a peak ground acceleration of 0.10g. An inactive fault may exist within Tongass Narrows and in the channels on either side of Pennock Island.

Most of the existing geotechnical data is near the shoreline, and there is essentially no subsurface data in the middle or deep parts of Tongass Narrows. Site specific geotechnical investigations are essential to establish an accurate geotechnical cross section and corresponding foundation design parameters for any bridge alternative selected.

## 2.0 Design Considerations

### 2.1 Marine Navigational Clearances

A primary consideration in designing a bridge crossing between Revillagigedo Island and Gravina Island is the effect on marine traffic in Tongass Narrows. The horizontal and vertical clearance requirements of marine traffic were investigated[1] and were incorporated into the design criteria[2] for the various bridge structures being evaluated.

Maximum and minimum vertical clearances under the bridge structures were established based on vessel traffic currently moving through Tongass Narrows. During the early phase of the project cruise ship operators in the Ketchikan area indicated that their future inventories included ships requiring a vertical clearance of 64 m (210 ft) above mean higher high water (MHHW).

In more recent discussions with cruise ship operators, it was determined that Lion's Gate Bridge in Vancouver, B.C., with a vertical clearance of 61 m (200 ft) above MHHW, is a primary limiting factor for cruise ships traveling to/from Vancouver, B.C. Therefore, it was determined that the maximum vertical clearance required for a bridge structure at Tongass Narrows is 61 m (200 ft) above MHHW. A 61 m (200 ft) vertical clearance would allow the passage of all cruise ships currently operating in the Ketchikan area.

This exceeds another height-limiting structure for vessels traveling to Southeast Alaska—the Seymour Narrows aerial cable crossing. The vertical clearance at Seymour Narrows is 56.4 m (185 ft). The Seymour Narrows cable could feasibly be raised to 61 m (200 ft), matching the vertical clearance of the Lion's Gate Bridge.

[1] *Gravina Access Project Reconnaissance of Vessel Navigation Requirements* (Glosten Associates, October 1999)
[2] *Gravina Access Project Design Criteria Technical Memorandum* (HDR, July 2001)

A second lower minimum vertical clearance of 36.6 meters (120 feet) above MHHW was considered. The lower vertical clearance permits the passage of all vessels up to and including those similar in size to the largest Alaska Marine Highway System vessels. A 36.6-m (120-ft) high bridge would result in the rerouting of large ships [i.e., ships requiring vertical clearances greater than 36.6. m (120 ft)] around Gravina Island.[3] Rerouting vessels of this size (primarily cruise ships) is not preferred by the City of Ketchikan due to potential economic impacts.

The different bridge vertical clearances under consideration directly affect the length of structure required to traverse Tongass Narrows. The project design criteria limits the grade on bridge approaches to a maximum of six percent. The eastern (Revillagigedo Island) approaches to the different bridge alternatives are in areas with steep slopes; therefore, the height of the bridge would not materially affect the length of the bridge from the east. The length of the western approaches, however, would be influenced by the vertical clearance chosen for the bridge. Assuming a six percent approach grade, a structure with 56.4 m (185 ft) of vertical clearance could be 76.2 m (250 ft) shorter than a structure with 61 m (200 ft) of vertical clearance. For a 14.6-m (48-ft) wide structure, the 56.4-m (185-ft) vertical clearance would result in a reduction in bridge deck area of 1,115 square (sq) m (12,000 sq ft), with respect to the 61-m (200-ft) high structure. A structure with 36.6 m (120 ft) of vertical clearance would be 406 m (1,333 ft) shorter than a structure with 61 m (200 ft) of vertical clearance, and its deck area would be 5,946 sq m (64,000 sq ft) less than a 61 m (200 ft) high structure. Cost of the structure is in general directly related to deck area. Thus the taller, longer bridges are the more expensive structures.

**Table 1. Comparison of Bridge Sizes by Vertical Clearance**

| | *Difference in Size Compared to Bridge with Vertical Clearance of 61 m (200 ft)* | |
|---|---|---|
| *Size Factor* | *Bridge with Vertical Clearance of 56.4 m (185 ft)* | *Bridge with Vertical Clearance of 36.6 m (120 ft)* [1] |
| Bridge Length Reduction[2] | 76.2 m (250 ft) | 406 m (1,333 ft) |
| Deck Area Reduction[3] | 1,115 $m^2$ (12,000 $ft^2$) | 5,946 m2 (64,000 $ft^2$) |

*1. A bridge with this vertical clearance would accommodate all Alaska Marine Highway System vessels.*
*2. Assumed approach grade = 6%.*
*3. Assumed bridge deck width = 14.6 m (48 ft).*

Requirements for horizontal clearances were established using the International Navigation Association (PIANC)[4] conceptual methods for channel widths. The conceptual channel width design is based upon a historical survey of ships that have passed through Tongass Narrows, projections of the type of ships anticipated to use the channel in the future, factors for channel bottom type and depth, visibility, and type of channel navigational aids. Based on this method, and the assumption that the large cruise ships would be restricted to one way travel through the bridge opening, the horizontal clearances were set at 168 m (550 ft) for one-way cruise ship passage, and 152 m (500 ft) for two-way passage of the Alaska State Ferry Columbia vessel type. All of the bridge layouts used for this study are based on these horizontal clearances.

[3] *Gravina Access Project Reconnaissance of Vessel Navigation Requirements* (Glosten Associates, October 1999)
[4] *PIANC Concept Design Method Email* (Glosten, January 2000)

Following the establishment of the horizontal clearances using the conceptual design method, AASHTO guidelines were used to develop allision frequency and force estimates. These data were used to determine pier foundation concepts for cost and alternative comparisons.

Computer simulations of a series of ship passages through Tongass Narrows were also initiated to develop a risk profile for ship passage. Called Monte Carlo simulations, the analysis is a means to assess the theoretical risk of a large cruise ship grounding in Tongass Narrows. Use of the Monte Carlo simulation for the Gravina Access Project is consistent with the request made by the U.S. Coast Guard Office of Bridge Administration to apply modern simulation methods to determine the horizontal clearance for any bridge crossing Tongass Narrows. It is also consistent with PIANC recommendations to use fast-time simulator techniques during preliminary design to estimate horizontal clearances.

In the Monte Carlo fast-time maneuvering simulations conducted for the Gravina Access Project, the risk of groundings or allisions has been determined for the natural channels at Charcoal Point, East Channel, and West Channel, and for potential bridge openings widths. The simulations were performed for large cruise ships and Alaska ferries transiting Tongass Narrows under the project's alternative bridge sites. The study estimated the probability distributions and statistics for 26,639 cruise ship transits and 45,550 Alaska ferry transits over 50 years. The reader is cautioned that the Monte Carlo simulation cannot account for implementation of extreme avoidance maneuvers and actions (other than commanding maximum helm angles) that might be ordered and attempted once the ship master or marine pilot recognizes that an accident is imminent. Potential extreme avoidance actions include combinations of the following: crash stops, twin-screw maneuvers (i.e., differential thrust), and use of the bow thruster and deploying one or both anchors.

The Monte Carlo study primarily confirms the need for pier protection for bridge alternatives. Secondarily, the study assesses the theoretical risk associated with current operations in Tongass Narrows to gauge the risk associated with proposed new bridges. Using the comparative risk approach, Table 2 presents the risk associated with the existing natural passages near Charcoal Point and in East and West channels respectively. Also provided are the relative risks associated with bridges with various effective horizontal clearances at Alternatives C3(a) and C4.

[5] *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges (February 1991)*

**Table 2. Comparative risk of potential groundings/allisions of large cruise ships operating in Tongass Narrows**

| *Location* | *Width (feet)* | *Probability of Exceedance* | *Normalized Risk Factor Relative to Natural Channel near Charcoal Point* |
|---|---|---|---|
| Charcoal Point | 687 | 0.001561 | 1.00 |
| East Channel | 477 | 0.013519 | 8.66 |
| West Channel | 476 | 0.016742 | 10.72 |
| C3(a) or C4 | 500 | 0.010502 | 6.73 |
| | 550 | 0.006197 | 3.97 |
| | 600 | 0.003707 | 2.37 |
| | 650 | 0.002246 | 1.44 |

[1] Width of natural channel between 5-fathom (30-foot) depth contours.

The natural passage risk in East Channel is 8.66 times greater than the risk of passage near Charcoal Point and the passage risk in West Channel is 10.72 times greater (or 24% greater than that in East Channel). It would require a bridge at Alternative C3(a) or Alternative C4 with an effective horizontal clearance of 687 feet to equal the passage risk near Charcoal Point but a bridge at that location with a 550 foot horizontal clearance would present less than half of the relative risk associated with the current passage of East Channel.

A “real time” simulation, conducted in a full mission simulator, replicates the actual conditions in a channel, and provides a marine pilot or ship master an opportunity to “pilot” a simulated vessel through the bridge opening. This is a method that can be used to calibrate the Monte Carlo simulation and to verify the conceptual PIANC channel design. The full mission/real time simulation is recommended to help verify the adequacy of an opening based on actual pilot input.

The U.S. Coast Guard is responsible for establishing navigation requirements, with the ultimate criterion being to meet the reasonable needs of navigation. The U.S. Coast Guard has stated that it must establish bridge clearances of navigable waterways in consideration of available studies, computer simulations, real time simulations, and consultation with the U.S. Army Corps of Engineers, the Federal Highway Administration, the design team, marine pilots, and shipping interests. The U.S. Coast Guard will provide the minimum navigational clearances needed for a bridge alternative, which may result in adjustments to the horizontal and vertical navigational openings used in this evaluation.

## 2.2 Aircraft Operation Constraints

Another consideration in designing a bridge crossing between Revillagigedo Island and Gravina Island is the possible encroachment of any structure on regulated airspace associated with the Ketchikan International Airport and floatplane operations[6] in Tongass Narrows.

[6] *Gravina Access Project Tongass Narrows Aviation Conditions Summary* (HDR, October 1999)

Aircraft operations associated with the Airport and the floatplane facilities in Tongass Narrows place constraints on the development of a bridge structure over Tongass Narrows. Ketchikan airspace is identified as Class E airspace; it is generally controlled airspace, permitting operations under Federal Aviation Regulations (FAR) for visual flight rules (VFR) and instrument flight rules (IFR), within certain parameters.[7] Ketchikan airspace is not controlled by a federally approved air traffic control tower, but it is monitored by the Ketchikan Flight Service Station (FSS), which provides air traffic advisories to aircraft operating within Ketchikan airspace.

Part 77 of the FAR, Objects Affecting Navigable Airspace, was developed to control the height of objects in the vicinity of an airport to ensure that airspace and approaches to each runway are protected from encroachment hazards that could affect the safe and efficient operation of the airport. The Part 77 airspace plan for Ketchikan International Airport[8] was investigated and alternatives were developed that would minimize impacts on the Airport's Part 77 surfaces.

The majority of aircraft operations in Ketchikan air space involve small single-engine floatplanes. During the summer tourist season, there may be as many as 500 floatplane operations in a single day. Under special VFR clearances, some floatplane operations are permitted below a 500-foot ceiling. The type and location of bridge structures under consideration for the Gravina Access Project were influenced by floatplane operations in Tongass Narrows[9]. For example, structure types that extend above the roadway deck will have a negative impact on floatplane operations.

## 2.3 Geometric Constraints

### *2.3.1 Vertical Constraints*

As discussed in Section 2.1, Marine Navigational Clearances, vertical clearances of 36.6 m (120 ft), and 61 m (200 ft) above MHHW were evaluated with the crossing alignments. Also, as discussed in Section 2.2, the FAA Part 77 surfaces and floatplane operations may constrain the location of a proposed alignment or bridge type.

### *2.3.2 Horizontal Constraints*

As discussed in Section 2.1, Marine Navigational Clearances, horizontal clearances of 168 m (550 ft) for one way cruise ship traffic, and 152 m (500 ft) for two-way Alaska State Ferry traffic were used in the evaluation of the crossing alignments.

### *2.3.3 Alignments*

Many access options were considered during an earlier phase of the project. Initial screening eliminated several of these options, based upon cost, travel time, and other environmental factors. The following sections describe the bridge alternatives currently under evaluation as reasonable alternatives for the Gravina Access Project. Profiles drawings of these are provided as

[7] *Ibid*
[8] *Gravina Access Project Tongass Narrows Aviation Conditions Summary* (HDR, October 1999)
[9] *Gravina Access Project Ketchikan Airspace Impacts Technical Memorandum* (HDR November 2001)

attachments in Sheet 1 through Sheet 11. The navigational and aviation impacts associated with each alternative are described in the *Navigation Opening Technical Memorandum* (HDR, August 2001) and *Aviation Impacts Technical Memorandum* (HDR, August 2001) for the Gravina Access Project.

***Alternatives C3(a): Airport Area to Signal Road***

The alternative C3(a) would start at Signal Road on Revillagigedo Island, traverse the hillside and gain elevation southward, turn southwestward to cross Tongass Avenue and Tongass Narrows, turn southward and parallel the runway. The vertical navigational clearance of the bridge would be 61 m (200 ft), and the main span horizontal clearance would be 168 m (550 ft). The bridge (Sheet 1) would be approximately 1.7 km (1.0 mi) long. The road would terminate at the boundary between the Airport Reserve and the Airport Development zones, and would include an exit to the Airport terminal.

***Alternative C3(b): Airport Area to Signal Road***

Like Alternative C3(a), Alternative C3(b) (Sheet 2) would connect to Signal Road on Revillagigedo Island, would traverse the hillside and gain elevation southward, turn southwestward to cross Tongass Avenue and Tongass Narrows, then turn southward and parallel the airport runway. The vertical navigational clearance of the bridge would be 36.6 m (120 ft) and the main span horizontal clearance would be 152.4 m (500 ft). The C3(b) bridge would be approximately 1.3 km (0.8 mi) long. The road would terminate at the boundary between the Airport Reserve and the Airport Development zones, and would include an exit to the Airport terminal.

***Alternatives C4: Airport Area to Cambria Drive Area***

Alternative C4 would connect to Tongass Avenue north of Cambria Drive, continue northward, traverse the hillside around the quarry, cross over Tongass Avenue and Tongass Narrows, then turn southward and parallel the airport runway. The Alternative C4 bridge would provide a vertical navigation clearance of 61 m (200 ft) and a main span horizontal clearance of 168 m (550 ft). The bridge (Sheet 3) would be approximately 1.7 km (1.0 mi) long. The road would terminate at the boundary between the Airport Reserve and the Airport Development zones, and would include an exit to the Airport terminal.

***Alternative D1: Airport Area***

Alternative D1 would start at Tongass Avenue near the airport ferry terminal, rise along the hillside behind the quarry, turn westward to cross over Tongass Avenue and Tongass Narrows, then turn southward and parallel the Airport runway, then touch down south of the terminal. The bridge (sheet 4) would provide navigational clearances of 37 m (120 ft) vertical and 152 m (500 ft) horizontal. The bridge would be about 1.0 km (0.6 mi) long. The road would terminate at the boundary between the Airport Reserve and the Airport Development zones, and would include an exit to the Airport terminal.

***Alternatives F3: Pennock Island***

Alternative F3 would cross Pennock Island, requiring two bridges: a bridge over the East Channel and a bridge over the West Channel of Tongass Narrows. Alternative F3 would start at Tongass Avenue south of the U.S. Coast Guard facility and cross the East Channel to Pennock Island. The East Channel crossing would provide 18 m (60 ft) of vertical clearance. The road would cross Pennock Island at grade and then use a second bridge over the West Channel providing 61 m (200 ft) of vertical clearance and 168 m (550 ft) of horizontal clearance. The East Channel and West Channel bridges (Sheet 5), would be 0.7 and 1.0 km (0.4 and 0.6 mi) long, respectively.

## 2.4 Structural Design Criteria

### *2.4.1 Superstructure Loads*

**Dead Load (AASHTO 3.3)**

**Density:**

| | |
|---|---|
| Pre-Stressed (P/S) and Reinforced Concrete: | $\Upsilon_c$ = 25.137 kN/m$^3$ (incl. reinf.) (160 pcf) |
| Steel: | Per AASHTO *Standard Specifications for Highway Bridges* |
| **Slab:** | Minimum 190 millimeters (mm) (7.5 inches [in]) without overlay (*thickness to be determined*); (65-mm [2 ½-in] Clear, top cover) |
| **Wear Course:** | 200 mm (4 inches) of asphalt total |
| **Barrier:** | Alaska Multistate Bridge Rail (TL-4) |

**Live Load**

| | |
|---|---|
| **Vertical Moving Loads – Truck Traffic and Lane:** | Per AASHTO *Standard Specifications for Highway Bridges except:* MS23 (HS-25 [HS-20 x 1.25]) |
| **Longitudinal Forces (AASHTO 3.9):** | Per AASHTO *Standard Specifications for Highway Bridges* |

**Impact (AASHTO 3.8)**

Per AASHTO *Standard Specifications for Highway Bridges*

**Vessel Collision**

Per AASHTO *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* and *Monte Carlo Navigation Simulation Technical Memorandum* (The Glosten Associates, October 2001).

**<u>Wind Load (AASHTO 3.15)</u>**

Design for wind and thermal loads will be according to AASHTO specifications. Due to the wind conditions that exist in Tongass Narrows high-level structures with considerable cross sections, such as cable stayed or suspension bridges will be more susceptible to wind than compact rigid structures, such as concrete box girders. Should a structure type be selected with a large cross section, additional wind studies may be warranted.

**<u>Earthquake Load (AASHTO 3.21 Division 1-A – Seismic Design)</u>**

A literature search of geotechnical data resulted in values of peak ground acceleration between 0.025g and 0.20g for the project area. Peak ground acceleration and other seismic criteria will be determined when a geotechnical investigation is completed.

| | |
|---|---|
| **Acceleration Coefficient:** | A= To follow with geotechnical investigation |
| **Importance Classification:** | IC = To follow |
| **Seismic Performance Category:** | Per AASHTO *Standard Specifications for Highway Bridges or ADOT&PF* |
| **Site Effects, Soil Profile Type I:** | To follow |

**<u>Thermal Forces (AASHTO 3.16)</u>**

**Temperature Range**

| | |
|---|---|
| Concrete Structures: | Temperature rise, 20° C (35° F)<br>Temperature fall, 25° C (45° F) |
| Steel Structures: | -34° to 49° C (-30° to 120° F) |

**Coefficients of Thermal Expansion (**a**)**

| | |
|---|---|
| Concrete: | 0.0000108 m/m/° C (0.000006 ft/ft/° F) |
| Steel: | 0.0000117 m/m/° C (0.0000065 ft/ft/° F) |

**<u>Uplift (AASHTO 3.17)</u>**

### *2.4.2 Substructure Loads*

Substructure loads will be the same as for the superstructure loads, except as follows.

**<u>Live Load</u>**

No impact from the footing level down.

**<u>Soils Density</u>**

To follow with geotechnical investigation.

**<u>Lateral Soil Pressure (AASHTO 3.20)</u>**

To follow with geotechnical investigation.

**<u>Live Load Surcharge (AASHTO 3.20.3, -.4)</u>**

600 mm (2 feet) of live load surcharge.

# 3.0 Substructures

## 3.1 Substructure

For purposes of this evaluation, it is assumed that the superstructure type is a post-tensioned concrete segmental box girder. It is the only bridge type common to all alignments. Because the seismicity of the site is relatively low, vessel collision forces were assumed to control the overall design of the substructure. This is likely to be the case whether the substructure is protected from vessel collision by sand islands or dolphins, or whether the foundations are designed to resist vessel collision forces directly. The actual cost of these various strategies and further study is needed to determine which is the most effective strategy. The strategy and resulting substructure configuration may depend on which alignment is chosen. Since one purpose of the conceptual design was to obtain order of magnitude, comparative costs, it was assumed that foundations would resist ship collision forces directly. An example of this design approach is shown in Sheets 6, 7, and 8.

Deep-water foundations may be constructed from either large diameter shafts (piles) or from caissons. The preliminary design was based on shaft foundations, since these are now more commonly used to support large bridges than are caissons. It may be appropriate to consider the caisson later, depending on the final alignment and the bridge type.

### *3.1.1 Vessel Collision Forces*

According to the AASHTO "Guide Specification for Vessel Collision Design of Highway Bridges" (February 1991) vessel collision forces depend on the deadweight tonnage of a ship and on its impact velocity. Because deadweight tonnage is a poor measure of the size of a cruise ship (because the weight of passengers and baggage is small relative to the weight of the ship), this was taken to be 75% of the loaded displacement. From Table 4 of the memo entitled "Reconnaissance of Vessel Navigation Requirements" future cruise ships visiting Ketchikan may have a displacement up to 64,500 tonnes (the Explores/Seas class).

The design methodology for vessel collision forces described in the AASHTO Guide Specifications is probabilistic, and the design is not necessarily governed by the largest ship, but by summation (averaging) over the spectrum of ships transiting the site, considering their types, weights, numbers, and probability of aberrancy. In lieu of making such a rigorous probabilistic calculation, an average vessel with a displacement of 35,000 tonnes (taken together with the total number of transits of ships of the Narrows) was assumed to govern the design (see Table 3 of "Reconnaissance of Vessel Navigation Requirements").

Vessel collision forces depend on impact velocity as well as ship size. The speed of vessels was taken to be:

**Table 3. Vessel Velocity**

| *Alignment* | *Velocity, knots* |
|---|---|
| A* | 12 |
| B* | 10 |
| C | 7 |
| D | 7 |
| F | 10 |

* No longer being considered.

After the conceptual design of foundations reported herein, Alignments A and B were dropped from consideration. Part of the rationale for this is the extreme cost of foundations in the deep water found along these alignments—as indicated in Section 3.1.3. The speed of vessels is least in the center of the Narrows, near the city and the airport. Ship collision forces were established for each of the alignments based on these speeds using the average vessel described above.

The vessel collision forces for each alignment were calculated using method II of the AASHTO Guide Specifications, assuming 500 transits each year of the average vessel. Other parameters used in the analysis were:

- Base rate of aberrancy = 0.0006
- Correction factor for bridge location = 1.0
- Correction factor for current = 1.3
- Correction factor for crosscurrent = 1.5
- Correction factor for density = 1.0
- Vessel length = 259 m (850 ft)
- Span length = 259 m (850 ft) (as for a concrete box girder bridge)
- Footing width = 30 m (100 ft)

The foundation design forces computed in accordance with the AASHTO Guide Specifications are:

**Table 4. Foundation Design Forces**

| *Alignment* | *Vessel Collision Force kN ( kips)* |
|---|---|
| A* | 109,000 (24,500) |
| B* | 90,800 (20,400) |
| C | 63,600 (14,300) |
| D | 63,600 (14,300) |
| F | 90,800 (20,400) |

* No longer being considered.

These forces correspond to a 19% probability of actual collision of an aberrant vessel with a pier, and an annual probability of aberrancy of 0.06. The projected return period of collisions of the design vessel with the bridge is 92 years. The design forces also correspond to a 1% chance of actual collapse in the event of a collision. Taken together, this all implies an annual rate of collapse of 0.0001 (i.e., a return period of 10,000 years), in accordance with the AASHTO Guide Specifications. If the bridge has a design life of 100 years, there is a probability of collapse of 1% during the life of the structure.

For conceptual design, the vessel collisions forces were computed for a 259 m (850 ft) span. For a 168 m (550 ft) span, the design forces will be slightly higher, because the geometric probability of collision with one of the piers will be increased. On alignment F, for instance, the vessel collision force increases to 93,700 kN (21,100 kips) for a span of 168 m (550 ft). This is a small increase, so that conclusions drawn for the larger span are generally applicable to the smaller span also.

### *3.1.2 Foundations*

**<u>Vertical Shaft Conceptual Design</u>**

Groups of 3.65 m (12 ft) diameter shafts were conceptually designed to resist the ship collision forces shown in Table 4. These forces correspond to a span length of 259 m (850 ft), which is suitable for a box girder superstructure. Dead loads corresponding to a concrete box girder were assumed for preliminary design (24,000 kN [55,000 kips] per span).

Shafts were assumed to be constructed of reinforced concrete, cast-in-place in steel shells, and socketed into bedrock. Designs were first prepared with vertical piles, where the ship collision forces are carried in bending of the piles. The number of shafts depends heavily on the depth of water. Conceptual designs were prepared for three water depths (corresponding to the average depth on three alignments):

**Table 5. Average Water Depth**

| *Alignment* | *Water Depth, m (ft)* |
|---|---|
| A* | 40 (130) |
| B* | 65 (215) |
| C, D, F | 20 (65) |

* No longer being considered

The designs are as follows:

**Table 6. Foundation Design (Vertical Shafts)**

| *Alignment* | *Pile Cap Size, m (ft)* | *Number of Shafts* |
|---|---|---|
| A* | 6.0x36.6x45.7 (20x120x150) | 20 |
| B* | 6.0x36.6x54.9 (20x120x180) | 24 |
| C, D, F | 6.0x24.4x36.6 (20x90x120) | 12 |

* No longer being considered.

The conceptual design of shafts includes 3% reinforcement (to resist bending) where they are fixed to the pile cap and socketed into rock. The shafts are assumed to be socketed 9.1 m (30 ft) into the rock, corresponding to a point of fixity 6.1 m (20 ft) below the rock line, as shown in the geotechnical report.[10] The axial loads in the shafts are carried primarily in end bearing, using an allowable bearing pressure of 3.8 Mpa (40 tsf). Steel shells to cast the shafts are assumed to be 50 mm (2 inches) thick (D/t = 72) (these are assumed to be non-composite with the shafts).

**<u>Battered Shaft Conceptual Design</u>**

Battered shaft conceptual designs were also prepared; although the constructibility of battered shafts is usually more difficult than construction of vertical piles, this concept is to be carried forward to final design. A smaller number of shafts (and a smaller pile cap) are needed if battered shafts are used because part of the vessel collision load is carried axially in the shafts, rather than in bending. Battered shaft conceptual designs are as follows, assuming an average batter of 1/12:

**Table 7. Foundation Design (Battered Shafts)**

| *Alignment* | *Pile Cap Size, m (ft)* | *Number of Shafts* |
|---|---|---|
| A* | 6.0x36.6x36.6 (20x120x120) | 16 |
| B* | 6.0x36.6x45.7 (20x120x150) | 20 |
| C, D, F | 6.0x24.4x24.4 (20x90x90) | 9 |

* No longer being considered

**<u>Spread Footings</u>**

On dry land the most appropriate foundation type will be a spread footing. Spread footings may also be a viable alternative in shallow water.

### *3.1.3 Foundation Cost*

Shaft costs were estimated as a function of depth using the unit costs of steel, concrete, reinforcement, and socketing developed for the replacement East Bay Bridge across San Francisco Bay. The main tower of the suspension span of this new bridge is also supported on large diameter shafts socketed into rock (through a depth of water of 24.4 m [80 ft]). The computed unit cost of shafts as a function of depth is:

[10] Phase 1 Geotechnical Report, Gravina Access Project, Shannon & Wilson, January, 2000.

**Table 8. Shaft Unit Cost**

| *Water Depth, m (ft)* | *Unit Cost, $/m ($/ft)* |
|---|---|
| 20 (65) | 13,500 (4,100) |
| 40 (130) | 11,500 (3,500) |
| 65 (215) | 10,800 (3,300) |

The unit cost of shafts decreases with increasing depth of water because the cost of socketing the shafts is amortized over a greater length of shaft.

Foundation square foot costs (dollars per square foot of bridge deck) were estimated for a concrete box girder bridge spanning 259 m (850 ft); the bridge was taken to be 14.6 m (48 ft) wide. The unit costs of concrete and reinforcement were based on those recently bid for several large U.S. bridges:

**Table 9. Foundation Costs (Concrete Box Girder)**

| *Water Depth m ( ft)* | *Vertical Shafts $/sq.m ($/sq.ft)* | *Battered Shafts $/sq.m ($/sq.ft)* |
|---|---|---|
| 20 (65) | 1,600 (150) | 1,300 (120) |
| 40 (130) | 3,700 (340) | 3,000 (280) |
| 65 (215) | 5,900 (550) | 5,000 (460) |

This table synthesizes the designs presented in Tables 6 and 7 for alignments A, B, C, D, & F with the water depth data shown in Table 5. The cost estimates are more generally applicable when presented in terms of water depth (which is the primary factor governing cost).

The costs of foundations for other bridge types, such as cable-stayed and arch bridges, may be different. These are discussed briefly in Sections 5.3 and 5.4, respectively. Variation in the bridge superstructure type and pier height impact substructure cost, though not significantly at this level of analysis.

The data presented in Table 9 make clear the remark made earlier about the extreme cost of foundations on alignments A and B, where the water is over 40 m (130 ft) deep on average. This is one reason these alignments were eliminated from consideration.

## 3.2 Piers

Piers are assumed to be hollow shafts of reinforced concrete. Local strengthening of the walls of the piers might be necessary near the waterline to resist vessel impact, depending on the size of the pile cap and the method of protection against vessel impact. Heavy confining reinforcement may be needed at the tops and bottoms of the piers, where plastic hinges might form during an earthquake. The corners of the cross-section may be enlarged and heavily confined to increase the ductility of the piers. Connections of the piers to a concrete box girder superstructure would, as far as possible, be monolithic. Depending upon the length of the structural units (between expansion joints), it may be necessary to support the superstructure on bearings at some piers, in order reduce the forces induced in the piers by creep and shrinkage of concrete. Connections to a steel plate or box girder superstructure would necessarily be through bearings—which is a disadvantage of this type of construction since bearings may require frequent inspection and maintenance. From cost data for other concrete box girder bridges, piers are estimated to contribute approximately $430/sq. m ($40/sq. ft) to the cost of the bridge, depending upon pier height.

### 3.3 Total Substructure Cost

The total cost of substructure (foundations and piers together) is obtained by adding $430/sq. m ($40/sq. ft) to the values shown in Table 9. This is for an assumed span of 259 m (850 ft), and for a concrete box girder bridge.

Assuming that pier costs—*per square meter of bridge*—are independent of span length and that foundation costs—*per footing*, not per square meter—are independent of span length (since vessel collision forces don't depend strongly on span length), the above data may be extrapolated to shorter spans as shown in Figure 2, for three depths of water. This extrapolation is useful for showing trends, but it may not accurately reflect the true cost at shorter spans.

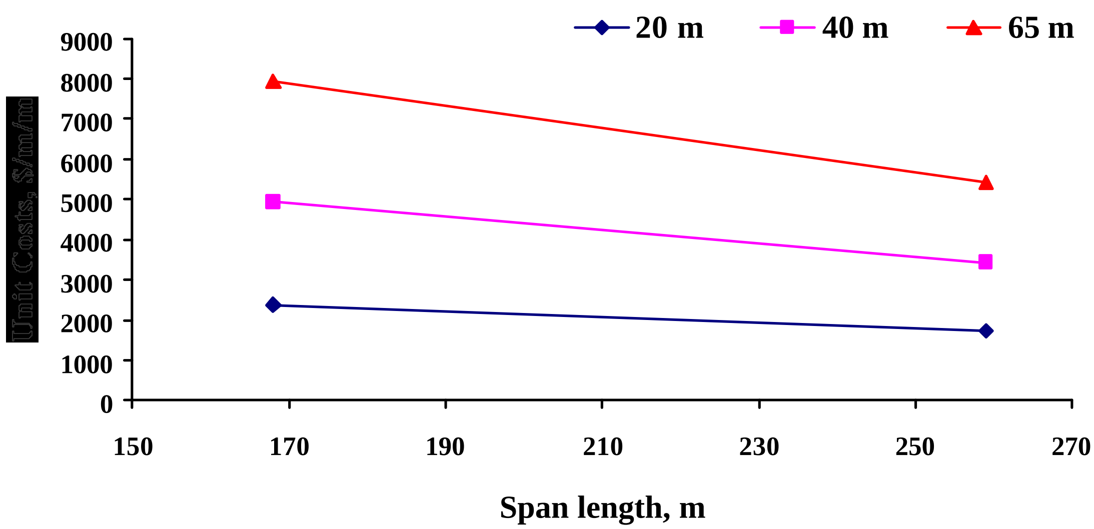


**Figure 2 – Substructure Costs, at Various Depths of Water**

## 4.0 Structure Types

Concrete and steel bridge types were considered when developing preliminary structure options. This section describes the various concrete and steel structures types examined for the Gravina Access Project.

## 4.1 Concrete Bridge Types

Concrete has two advantages that might make it preferable to steel for construction of a bridge at this site. The first advantage of concrete as a material is that the future inspection and maintenance requirements would be less for a bridge made from concrete than for one made from steel.

The second advantage of concrete is that it may be made predominately with local materials. Even if the raw materials were brought to Ketchikan, the concrete itself would be mixed and placed locally, and the reinforcing steel would be placed locally. This would be the case whether the bridge is precast or cast-in-place. Most of the labor needed to construct the bridge could be found locally. The weather in Ketchikan does not appear to present any real obstacle to concrete construction. Only routine precautions for cold-weather concreting would be necessary. In the case of cantilever construction, it is quite feasible to protect the work area from rain by erecting a cover over the form traveler. This was the technique used to construct the North Halawa Valley Viaduct in Hawaii, where the annual rainfall is about 200 inches per year (similar to Ketchikan). Winds are not usually a problem for construction of box girder bridges by balanced cantilever, since these structures are relatively heavy and stiff. And cable-stayed bridges can be effectively guyed during construction to ensure their stability.

Three concrete bridge types are well suited to the local conditions. These are a post-tensioned segmental box girder bridge, a cable-stayed bridge, and an arch bridge.

### *4.1.1 Post-Tensioned Segmental Concrete Box Girder*

For a box girder bridge, the project navigational requirements of a 168 m (550 ft) horizontal clearance dictate a main span in the range of 210 m (690 ft) or more. This is a large but feasible span for a post-tensioned segmental box girder bridge. The economical span length for this type of bridge is generally considered to extend to about 229 m (750 ft). The current world-record span is 301 m (988 ft); this belongs to the Stolma Bridge in Norway.

The width of the roadway dictates the width of the deck. In this case, two lanes of traffic plus shoulders and a sidewalk require a total width of 14.6 m (48 ft) between railings. This can easily be accommodated by a single cell box girder, without any transverse ribs or struts, thus simplifying construction. Based on rules-of-thumb, the cantilever arms should measure about 3.66 m (12.0 ft) from the side of the box. This is relatively small, so live load deflections or vibrations should be minimal.

Also based on rules-of-thumb, the depth of the structure would be about 5.3 m (17 ft) at mid-span (span/depth of 40/1, which is conservative) and 11.7 m (38 ft) at the piers (span/depth of 18). Although these dimension are quite adequate for preliminary cost estimates and selection of an alignment, they must be revisited at a later stage, and the structure finally proportioned by analysis. Structural depths of this size could easily accommodate utilities such as water, telephone, and petroleum pipelines. Examples of post-tensioned segmental box girder bridges are shown in Sheet 1 through Sheet 5.

The piers would likely be of box shape, with dimensions comparable to those of the superstructure. The transverse dimension in particular should match the width of the box, and is 7.3 m (23.9 ft). The longitudinal dimension of the pier is more difficult to establish, since the pier must be sufficiently strong to resist moments resulting from cantilever construction, but sufficiently flexible during service to avoid the build-up of excessive forces due to creep and shrinkage of concrete. The solution to these conflicting requirements depends strongly on pier height. For a box pier, the longitudinal dimension is likely to be about 6.0 m (19.7 ft). For piers which are not in deep water and not likely to be hit by ships, so-called double-leaf piers may be appropriate; as shown in Figure 3. This type of pier has great flexural strength to resist overturning moments, but is flexible enough to accommodate shortening of the bridge (due to creep and shrinkage) without inducing large forces in the structure. It is very suitable for long-spans like that proposed here. The individual walls of a double-leaf pier would not resist ship impact, however, so box type piers would be necessary for piers in deep water.

**Figure 3 – Double-Leaf Piers**

A segmental concrete box girder bridge would almost certainly be built by the balanced cantilever method, and cast-in-place using form travelers. For spans of the order of 210 m (690 ft) no other method of construction is really practical. Construction with form travelers is a mature technology; hundreds of structures of this type have been built around the world, including one in Alaska—the Gastineau Channel Bridge in Juneau, with a main span of 189 m (620 ft). Many U.S. contractors and specialty subcontractors have experience with this type of construction.

**Figure 4 – Form Travelers Installed on Bridge**

In the balanced cantilever method of construction the piers of the bridge are built first (at least before the superstructure is built). A "pier table" is then built on the top of each pier using more-or-less conventional falsework. This is a portion of the superstructure extending a few meters to each side of the pier. A pair of "form travelers" is then installed on the top of the pier table, cantilevering out towards the centers of the adjacent spans, as shown in Figure 4. Each traveler carries the formwork needed to cast a segment of the box girder, typically 5-6 m (16-20 ft) in length. When a new segment is cast, the form traveler is moved forward onto it, so that it again cantilevers out towards the center of the span. The two form travelers on either side of a pier work simultaneously, moving outwards from the pier in a *balanced* manner. In this way, the complete structure is built, one segment at a time, from each side of each pier, as illustrated in Figure 5. A complete span is formed by the cantilevers extending from adjacent piers.

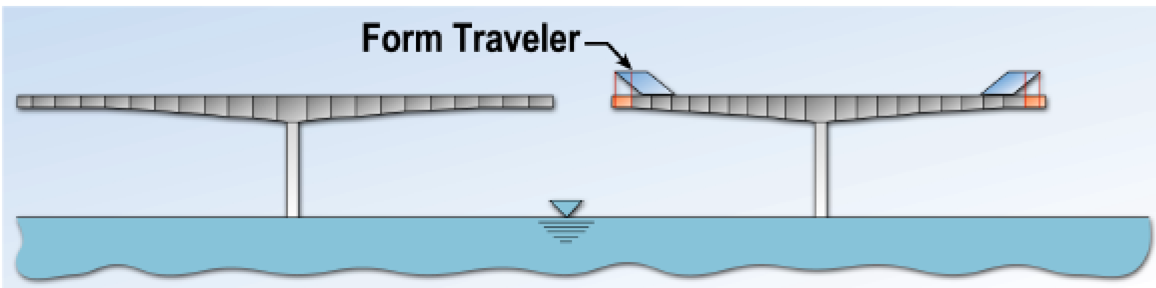


**Figure 5 – Construction with Form Travelers**

As each segment is added to the bridge it is connected to the already completed structure with "cantilever" (post-tensioning) tendons. When installed, each cantilever tendon runs from the tip of one cantilever to the tip of the other cantilever, over the top of the pier. The first cantilever tendons are short, since the first segments are close to the pier. When the form travelers approach midspan, the cantilever tendons become very long. These tendons are the primary load carrying elements of the bridge.

A closure segment is used to connect the cantilever arms from adjacent piers in order to complete each span, as shown in Figure 6. Short "span" tendons are placed through the closure segment in order to carry positive moments at midspan. The top slabs of segmental concrete bridges are typically post-tensioned transversely also; this is the primary direction of load transfer in the slab. Thus the top slab is compressed both longitudinally and transversely, which minimizes cracking and increases the durability of the bridge.

**Figure 6 – Construction of Closure Segment**

Construction with precast segments isn't feasible for the main span of the bridge; it's too large a span. Precast construction isn't likely to be economical for the approach spans either, depending upon the final span lengths and the overall geometrics of the structure. The currently proposed approach spans measure 152 m (500 ft), which is appropriate for their height. But this is beyond the span range typically considered practical for precast construction. Also, many of the proposed structures are along severely curved alignments, which makes precast construction more difficult.

Cast-in-place concrete box girder bridges can easily satisfy difficult geometrical requirements, either in the form of steep grades or horizontal or vertical curves of small radii. This is because they are built in segments; for each segment, the form traveler just needs to be repositioned to accommodate the tangential grade and alignment. (Temporary bents might be needed to support the superstructure during balanced cantilever construction if it *isn't* monolithic with the piers.)

When constructing by the balanced cantilever method of construction, most of the work is done from overhead, so the method is particularly well suited to construction over water, difficult terrain, or the environmentally sensitive areas that make up the project sites.

The construction of concrete box girders bridges by cantilevering is now routine within the United States. Weather conditions in Ketchikan present no special problems that are not faced within the Lower 48. Of all of the structure types considered, the segmental box girder structure type typically costs less to construct than the other structure types considered.

In a concrete box girder bridge, the superstructure is typically monolithic with the piers, at least for the piers flanking the main span. Thus, a minimum number of bearings are required. The superstructure may be made continuous over a length of 305 m (1,000 ft) or more, so that few expansion joints are needed. A concrete box girder bridge would then be relatively easy to maintain.

Segmental concrete box girder bridges, particularly cast-in-place bridges, have a good track record. A year 2000 survey of State Department of Transportation inspection and maintenance records by the American Segmental Bridge Institute included 164 bridges built in the U.S. in recent decades. Eighty percent of the bridges were considered to be in "good" condition or better.

Only 2 of the 164 bridges were rated as being in “fair” condition. All others were rated “satisfactory” or better.

A 1998 study by the National Cooperative Highway Research Program identified few reported problems with segmental concrete bridges in the U.S. The study concluded that properly designed and constructed segmental concrete bridges are highly durable.

Concrete box girder bridges are also quite resistant to earthquakes. With a minimum number of bearings, they are highly redundant, tough structures. Yet, the lateral-load-resisting mechanism is fairly easy to determine and the locations of maximum moment can be easily identified and reinforced.

This bridge type is considered appropriate for all alignments currently being considered.

***<u>Advantages</u>***

a. **Appearance –** Good, post-tensioned segmental concrete box girders are aesthetically pleasing with clean, simple lines.

b. **Constructability** - Relative ease of construction; well suited to construction over water, and difficult terrain.

c. **Construction Cost –** Low, least costly of the alternatives considered.

d. **Maintenance Cost –** Low, concrete structures have relatively low long-term maintenance costs when compared to steel structures.

e. Post-tensioning tends to close cracks, reducing effects of deicing chemicals thus increasing the durability of the deck.

f. **Environmental Impact –** Low, least impact on environmentally sensitive areas because the balanced cantilever method minimizes falsework requirements.

g. Structure type is inherently stable, torsionally stiff and has high earthquake resistance.

h. Horizontal and vertical clearance requirements are satisfied.

***<u>Disadvantages</u>***

a. **Aviation Impact –** Moderate, at alignments C3(b) and C4 the 61 m (200 ft) high bridge will protrude into the FAA Part 77 surfaces.

b. Economical and technical limitation of this structural type is a 229 m (750 ft) span length.

c. Very few post-tensioned segmental concrete box girder bridges (Gastineau Channel Bridge) have been constructed in Alaska.

### *4.1.2 Concrete Cable-Stayed*

A cable-stayed bridge would satisfy any requirement for a “gateway” structure. A main span of about 320 m (1,050 ft) would meet the navigational requirements of the project. On alignment F, a span of this size would cross most of the east channel of Tongass Narrows, between Pennock Island and Revillagigedo Island. As shown in Sheet 9, both the main pylon and the end pier are placed out of the deepest water in order to minimize the cost of foundations and the risk of ship collision. The structure proposed is an asymmetrical bridge, with a cable-supported backspan of only 200 m (660 ft). This span is over land, so a longer span is unnecessary. The short backspan can be made to “balance” the main span by using a heavier deck section in the backspan than in the main span. Also, an intermediate pier is provided in the backspan to anchor some of the backstays.

The bridge deck would consist of a reinforced concrete slab supported longitudinally by concrete edge girders. Transversely, the slab would be supported by prestressed concrete floor beams spanning between the edge girders. The edge girders themselves would be supported by the stays, hanging from the main pylon. The relatively stout concrete edge girders would allow a stay spacing of about 10 m (32.8 ft). Typical concrete cable-stayed details are shown in Sheet 10.

A diamond shape pylon is proposed, but other types of pylon are possible. However, the combination of central anchorage of the stays in the head of the pylon and edge support of the bridge deck gives the stays a transverse inclination. This increases the lateral stiffness of the deck, which is valuable in high winds. And it also increases the stability of the tower. Edge support of the bridge deck gives it torsional stability (which is required for an open deck section).

The pylons would extend about 130 m (427 ft) above the bridge deck, for a total height of structure of about 212 m (695 ft). This is shown in Sheet 9, for alignment F. Some flexibility is possible in the pylon height, to minimize aviation impacts. But this might adversely affect the economy of the bridge, since the height shown is nearly optimum for structural efficiency and cost.

Cable-stayed bridges of this type and size are readily constructible; there are many examples around the world. The construction of cable-stayed bridges by cantilevering from the piers is now fairly routine. Many U.S. contractors and specialty subcontractors have the experience to make construction of this type of bridge practical. Weather conditions in Ketchikan present no special problems. The cost of this type of bridge is likely to be comparable to that of a segmental concrete box girder bridge crossing the same alignment.

A cable-stayed bridge would present no special maintenance problems, other than the stays. Modern stay cables are very durable, however, and should last for many decades (say, 60 years). A one-time replacement of the stays during the life of the bridge should be planned for. The seismic resistance of cable-stayed bridges is quite good also.

One detrimental feature of a cable-stayed bridge is that the cable stays and towers would hinder floatplane operations in Tongass Narrows. The majority of operations in Ketchikan air space involve small single-engine floatplanes. During the summer tourist season, there may be up to

500 floatplane operations in a single day. Under special VFR clearances, some floatplane operations are permitted below 152 m (500 ft), well below the location of the towers and cable stays.

The approach spans to a cable-stayed main span would be approximately 80 m (262 ft) long. The unit cost of these structures would be about the same as for the (over land) segmental concrete box girder options discussed elsewhere.

This bridge type is considered appropriate for only the F3 alignment, at the west channel crossing. A concrete box girder bridge type would be used at the east channel in this case.

**<u>Advantages</u>**

a. **Appearance** – Good, cable-stayed bridges are aesthetically pleasing, signature type structures.

b. **Constructibility** - Relative ease of construction; well suited to construction over water, and difficult terrain.

c. **Construction Cost** – Low, one of the least costly of the alternatives considered.

d. **Maintenance Cost** – Moderate, concrete structures have relatively low long-term maintenance costs when compared to steel structures (see also paragraphs c. and d. Disadvantages).

e. Structure type is inherently stable, and has good earthquake resistance

f. **Environmental Impact** – Low, least impact on environmentally sensitive areas because the balanced construction method eliminates falsework requirements.

g. Horizontal and Vertical Clearance requirements are satisfied.

**<u>Disadvantages</u>**

a. **Aviation Impact** – High, the towers and cable stays will be extremely detrimental to float plane operations in Tongass Narrows.

b. Limited to Pennock Island alignments.

c. Cable-stays will need to be replaced every 60 years.

d. Cable-stays may be subject to vibrations induced by wind and/or rain. Many cable-stayed bridges have had to be retrofitted to minimize stay vibrations.

e. Very few cable-stayed bridges have been constructed in Alaska, and certainly nothing of this size.

### *4.1.3 Concrete Arch*

An arch bridge most readily meets any requirement for a "signature" structure. The bridge shown in Sheet 11 crosses the east channel of Tongass Narrows, between Pennock and Revillagigedo Islands, along alignment F. It would be a dramatic "gateway" to Alaska. The proposed structure has a main span of 500 m (1,640 ft) and a height of approximately 95 m (312 ft). These are world record dimensions for a concrete arch. We consider it feasible to build this structure (assuming that an alignment with competent supporting rock can be found); but it would be very expensive. Because of its extreme span and high cost, this bridge type was not selected for further consideration.

The structure spans from shore to shore of the east channel, so the foundations of the arch rib are out of the water and not subject to vessel impact. The arch rib itself must still be protected from vessel collision, however; so four (or more) dolphins would have to be built in the channel outboard of the navigation channel in order to protect the arch rib. The cost of these dolphins is included in estimates for the arch bridge.

The feasibility of the arch structure shown in Sheet 11 depends upon having competent rock to support it. Unfortunately, adequate geotechnical information is available at this time, so it is not possible to make a final determination of the feasibility of this concept.

The deck of a concrete arch bridge would consist of a concrete slab integrated with a box girder. The box girder would be supported at 50 m (164 ft) spacing by spandrel columns supported off the arch rib.

Although it would be an elegant structure, an arch bridge would be difficult to build and hence very expensive. Over water, the arch rib would have to be temporarily supported by stay cables running from towers erected either over the permanent foundations of the arch rib, or over the dolphins placed in the Narrows for protection from vessel impact. The arch rib would be either cast in place or formed from precast segments lifted into place by a barge-mounted crane. On top of the arch rib, the deck could be built on falsework. Over the navigational channel, the deck could be formed from precast segments lifted into place.

Bearings and expansion joints can also be minimized for this type of bridge so that its maintenance would be relatively straightforward. The seismic resistance of an arch bridge is less than that of a box girder bridge, but probably satisfactory in an area of moderate seismicity like Ketchikan.

The main span would be flanked by smaller concrete spans of approximately 100 m (328 ft). These approach spans are shown in Sheet 11.

**<u>Advantages</u>**

a. **Appearance –** Good, concrete arch bridges are aesthetically pleasing, signature type structures.

b. **Maintenance Cost –** Low, concrete structures have relatively low long-term maintenance costs when compared to steel structures.

c. Horizontal and Vertical Clearance requirements are satisfied.

***Disadvantages***

a. **Constructibility** - Difficult to construct, will require either falsework or a cable stayed erection tower to construct.

b. **Construction Cost –** High, one of the most expensive of the alternatives considered.

c. **Environmental Impact –** Moderate, temporary towers for construction may have some impact on environment.

d. A bridge constructed with the span length proposed will be of record length.

e. **Aviation Impact –** High, may be detrimental to aviation.

## 4.2 Steel Bridge Types

Steel structures can be economical solutions to long-span structures. In this environment and location, four structural types: box girder, plate girder, tied-arch, and movable truss may have potential. They must be evaluated for life-cycle cost, however, because of the need for coating renewal, maintenance, and corrosion concerns. These issues need to be considered in comparison with the concrete alternatives to establish the most overall cost-effective structure.

### *4.2.1 Steel Box Girder*

This concept uses a stiffened steel box superstructure. The depth of the girder varies to accommodate the relatively large spans. The main span length is of the order of 210 m (690 ft) to provide the required horizontal navigation clearance. The box girder section handles torsional stresses efficiently, and would therefore work well on a curved alignment near the airport.

Fabrication of the box girder would need to be done in a qualified steel fabricator facility. There are several on the West Coast. The girder could be fabricated and shipped to the site and erected in sections. The main unit could be floated in on a barge, then lifted into final position using jacks from above. Temporary falsework might be required during erection of the curved spans.

The deck could be either concrete, made composite with the steel box, or orthotropic steel with an asphalt concrete wearing surface.

This bridge type is considered appropriate for all alignments currently being considered.

**Advantages**

a. **Appearance –** Good, aesthetically, steel box girders have clean, simple lines.

b. Structure type is torsionally stiffer than a plate girder cross section.

c. Horizontal and Vertical Clearance requirements are satisfied.

**Disadvantages**

a. **Aviation Impact** – Moderate, at alignments C3(b) and C4 200 ft) high bridge will protrude into the FAA Part 77 surfaces.

b. **Construction Cost** – High, higher construction cost relative to other materials can be expected.

c. **Maintenance Cost** – High, steel bridges generally require more maintenance than do concrete structures.

d. **Constructibility –** Moderate, complex to construct, may require falsework towers to construct and bolted connections.

e. **Environmental Impact** – Moderate, temporary towers for construction, removal old paint and repainting of the steel structure may have some impact on environment,

### *4.2.2 Steel Plate Girder*

Three vertical steel plate girders, with either an orthotropic steel or composite concrete deck, constitute the longitudinal girder section in this concept. A main span length of 210 m (690 ft) is achievable by using a varying girder depth. The girders could be fabricated curved, to accommodate roadway alignment. Torsional forces are controlled by steel cross-frames and bottom flange lateral bracing.

As with the orthotropic box, the girder could be fabricated and shipped to the site in full-width sections, where they would be lifted into position with waterborne cranes or lifting frames on the partially completed bridge. Temporary falsework bents might be necessary for sharply curved spans.

This bridge type is considered appropriate for alignment F3 only.

**Advantages**

a. **Appearance –** Fair, aesthetically steel plate girders have clean, simple lines.

b. Horizontal and Vertical Clearance requirements are satisfied.

**Disadvantages**

a. **Construction Cost –** High, higher construction cost relative to other materials can be expected.

b. **Maintenance Cost** – High, steel bridges generally require more maintenance than do concrete structures.

c. **Constructibility** – High, complex to construct. Structure type, especially in curved sections, is not stable during construction, not as torsionally stiff as a box shaped cross, section and may require falsework towers to construct.

d. **Environmental Impact** – Moderate, temporary towers for construction, removal old paint and repainting of the steel structure may have some impact on environment.

e. **Aviation Impact –** Moderate, at alignments C3(b) and C4 61 m (200 ft) high bridge will protrude into the FAA Part 77 surfaces.

### *4.2.3 Steel Tied Arch*

A steel tied-arch bridge could be constructed to span up to the 210 m (690 ft) range required for navigation clearances. The edge girder for roadway deck is designed as the tension tie between the ends of the arch span. Because of the height of the arch, or alignment considerations this bridge type was not selected for further consideration.

Erection of the tied-arch bridge is more complicated than for the other concepts. A temporary erection tower or falsework would be required to construct arch ribs and the tension tie.

One negative feature of a tied arch bridge is that the arch and hangers would be detrimental to float plane operations in Tongass Narrows. A tied arch would have the majority of the superstructure high above the roadway, and, like the cable-stayed structure, would be a hindrance to floatplane operations.

**Advantages**

a. **Appearance –** Fair, steel tied arch bridges are aesthetically pleasing, signature type structures.

b. Horizontal and Vertical Clearance requirements are satisfied.

**Disadvantages**

a. **Construction Cost** – High, higher construction cost relative to other materials can be expected.

b. **Maintenance Cost –** High, steel bridges generally require more maintenance than do concrete structures.

c. **Constructibility –** Very high, complex to construct, will require either falsework or a cable stayed erection tower to construct, with some impact to the environment.

d. **Environmental Impact** – Moderate, temporary towers for construction, old paint removal and repainting of the steel structure may have some impact on environment

e. A tied arch does not work on horizontal curves, or where the structure height is restricted by airport clearances.

f. Limited to Pennock Island alignments.

g. **Aviation Impact –** High, the towers and hangers will be extremely detrimental to float plane operations in Tongass Narrows.

### *4.2.4 Movable Bridge*

A movable bridge concept was considered because it would allow cruise ships to pass through Tongass Narrows and it would not penetrate the Part 77 airspace to the degree that a high-level bridge would. Although the approaches to the lift span would not penetrate the Part 77 airspace, the lifting towers and would penetrate the airspace, as would the lift span when the bridge is in the open position.

A vertical-lift movable bridge is composed of a simple span structure with lifting towers at the supports. Each end of the span is elevated with cables that run from the span over sheaves in the towers. Both ends of the bridge are lifted simultaneously to ensure that the span remains level at all times. Because the spans of a movable bridge must be lifted, the lifted span is typically fabricated from structural steel. For spans in this range, a steel truss is the only structure type that is viable. For this movable bridge type, a 189-m (620-ft) simple span steel truss would be supported by braced steel towers at each end. The depth of the superstructure would vary from 15 meters (50 ft) at the supports to 20 meters (65 ft) at midspan.

To minimize traffic interruptions due to vessel traffic, a vertical clearance of 36.58 meters (120 ft) above MHHW was assumed for the bridge in the closed position. This clearance would allow Alaska Marine Highway System vessels to pass under the bridge without requiring an opening of the span. The maximum vertical clearance (i.e., when the bridge is in the open position) would be 61 meters (200 ft) above MHHW.

Because of cost and travel delays associated with the open/close cycles of a movable bridge, this type was not selected for further consideration.

**<u>Advantages</u>**

a. Horizontal and Vertical Clearance requirements are satisfied.

**<u>Disadvantages</u>**

a. **Construction Cost –** High, higher construction cost relative to other materials can be expected. The lifting mechanism will also increase the construction cost .

b. **Maintenance and Operational Cost –** Very high, movable bridges require significantly more maintenance than do concrete structures due to the maintenance required of the lifting mechanism and steel superstructure. The operation of the bridge will require a full time staff of movable bridge operators.

c. **Constructibility –** Highly complex to construct, may require falsework towers to construct, with some impact to the environment.

d. Auto and truck traffic may have up to a 30-minute wait while the bridge is opened for vessels

e. **Appearance –** Poor, unattractive structure.

f. **Aviation Impact –** Low to Moderate, the structure would not penetrate the FAA Part 77 airspace to the degree that a high-level bridge would, it may have some impact on floatplane operations.

# 5.0 Structure Costs

## 5.1 Methodology

Costs for a comparable project in Alaska in the scale of the Gravina Access Project are not available. Therefore, the project team attempted to use cost data from comparable projects in North America. After researching other projects it became apparent that pier protection would significantly affect the unit area structure costs. Conceptual design of piers and foundations was performed (See Section 3) and costs were established for foundations in depths of 20 m (65 ft), 40 m (130 ft), and 65 m (215 ft). Foundation costs for each alternative alignment were established by considering the depth of the channel at the pier and whether the pier would likely be subject to ship impact. Only piers immediately adjacent to the shipping channel, or in deep water were considered as candidates for ship impact.

Contingencies in the form of additional cost percentages are added to account for items unknown at this time. For example, a 25% contingency is added to the deep water foundation costs because of the lack of site specific geotechnical data. This is an accepted method of adjusting the cost estimate at this conceptual level of development. Other contingencies are also added to allow for the lack of detailed bathymetric and topographic data, no recent history of large bridge construction in Alaska, uncertainties of material market conditions at time of construction, and the potential cost of importing certain labor skills for the bridge construction.

## 5.2 Post-Tensioned Segmental Concrete Box Girder

### *5.2.1 Main Spans*

The cost estimate for these spans is based on data collected from the Benicia-Martinez Bridge, a segmental box girder bridge recently bid in the San Francisco Bay area, and the segmental box girder portion of the (now advertised) San Francisco-Oakland Bay Bridge, and other comparable projects. Based on these two projects, an average unit cost of superstructure of $2,150/ sq m ($200/sf) was used for estimating order of magnitude costs, and for comparison of alignments and structure types. For comparison, the bid unit cost of the superstructure of the Benicia-Martinez Bridge was approximately $2,050/sq. m ($190/sf) and the estimated unit cost of the superstructure for the San Francisco-Oakland Bay Bridge is $2,500/sq. m ($229/sf). Both of these bridges are long span (approx. 160 m [525 ft]) crossings over water, and a suitable basis for estimating the costs of a future Gravina Island Bridge.

An average unit cost of $2,150/ sq m ($200/sf) was calculated for the substructure from the conceptual pier and foundation designs described in Section 3. The foundation costs depend strongly on water depth, as discussed in Section 3.1.3. The unit cost mentioned is suitable for water depths up to about 20 m, and is appropriate for structures on the C, D, and F alignments. The high cost of the substructure relative to the superstructure (these are about equal) is characteristics of large water crossings. This ratio differs from that for typical segmental box girder bridges, where the substructure costs run about 30% of the total cost.

The unit costs used for the project cost estimate were escalated 25% for substructure contingencies and 10% for superstructure contingencies, and 10% for mobilization to obtain an average unit cost of $5,500/ sq m ($510/sf). The costs for the Gravina project are comparable to the $4,840/ sq m ($450/sf) of the Benicia-Martinez Bridge, and the estimated $4,600/ sq m ($431/sf) of the East Bay Bridge. The Confederation Bridge built several years ago across the Northumberland Strait in Canada cost about $3,800/sq m ($350/sf), but that is an extremely long structure, with economies of scale that the Gravina project will not enjoy.

### *5.2.2 Approach Spans*

The greater portion of any segmental concrete bridge would be over land, on Gravina and Revillagigedo Island. These approach spans will not be nearly so expensive to construct as will the main spans over the Tongass Narrows because they will be shorter (there is no navigation requirement) and because access to the work will be much easier. Also, simpler foundations can be used since ship impact loads needn't be considered.

The preliminary design of the approach spans assumes a typical span length of about 150 m (500 feet), which is common for cast-in-place segmental concrete box girder bridges. The estimated unit cost of these structures is about $2,700/ sq m ($250/sf). This breaks down approximately as follows: $1,800/ sq m ($170/sf) for superstructure and $900/ sq m ($80/sf) for piers and foundations. This ratio of superstructure and substructure costs is more typical of segmental concrete bridges than are the main span costs. With 20% allowed for contingencies and 10% for mobilization the unit cost of the approach spans is $3,500/ sq m ($325/sf).

## 5.3 Concrete Cable-Stayed

Costs for the cable-stayed bridge were based on unit costs for concrete, reinforcement, and stays bid for cable-stayed bridges of size comparable to that proposed here. The cost of the superstructure, main pylon and piers is estimated to be $23,400,000 over a length of 520 m (1,700 ft) and a width of 14.6 m (48 ft), which is $2,900/ sq m ($270/sf). Along alignment F, the main pylon and the end pier are both founded in approximately 20 m (65 ft) of water. These foundations are estimated to cost about $5,100,000 each to construct, assuming that they can directly resist vessel impact forces. Including the backspan piers, the total cost of the foundations is estimated to be $11,200,000, which is $1,400/ sq m ($130/sf). The total cost of the cable-stayed bridge is then $34,600,000, or $4,300/ sq m ($400/sf), without contingency and mobilization. Applying the contingency and mobilization factors given in the previous section, the total cost of the cable-stayed bridge is $43,600,000, or $5,400/ sq m ($500/sf).

The above costs assume that the main pylon and end pier foundations directly resist vessel impact forces. It is also possible that dolphins could be used to protect these foundations, so that they would be designed to resist primarily vertical loads. Preliminary designs show this approach to be more expensive, however.

The approaches to a cable-stayed bridge would most likely be segmental concrete box girders, costing approximately $3,000/ sq m ($280/sf) (with contingency and mobilization). These structures would be largely overland so the costs are significantly less than for the water crossings described in the previous section.

### 5.4 Concrete Arch

Costs for the concrete arch bridge were based on prices bid for the Crooked River Bridge, in Oregon, and on engineering judgment. The Crooked River Bridge is a concrete arch bridge across the Crooked River Gorge; it was completed in October 2000 and has a main span of 125 m (410 ft). Engineering judgment was used to extrapolate the prices bid for this bridge to the much larger span proposed here. The estimated cost of 500 m (1,640 ft) arch span itself is $59,300,000, without contingency or mobilization. Additionally, the arch ribs must be protected against vessel impact by four dolphins, estimated to cost $2,900,000 each. The total cost of the arch span is then $70,900,000, or $9,100/ sq m ($850/sf), *without contingency or mobilization*.

### 5.5 Steel Structures

As discussed earlier in this report the team was asked to limit detailed analysis to the three most applicable bridge types. Detailed analysis was not performed on the steel plate girder, box and tied arch structure types. Unit cost for these structure types were determined by referencing the unit costs for each type in the Washington State Department of Transportation's Bridge Design Manual. Unit costs are listed in the preliminary design section of that manual to facilitate planning level costs when comparing different structure types. The values listed in the manual were factored up by a ratio of the unit cost of a post-tensioned segmental box structure for this project (discussed earlier in Section 5), and tabulated value of the segmental box.

### 5.6 Concrete vs. Steel Structure Cost Comparison

Because of the limited scope of this study and in lieu of more detailed information to verify concrete vs. steel structure relationship, a comparison between concrete and steel structural types was made based on two other projects recently completed or currently under construction on the West Coast of North America. The location, climatic conditions and type of crossing have striking similarities with this project.

The most recent example is the New Benicia-Martinez Bridge across the Carquinez Strait in California. In 1989-90, the client (California Department of Transportation) commissioned the studies of four different structural types for a crossing over deep water with long pile foundations, in a seismic area, and with a busy shipping channel; these conditions are similar to those prevailing in the Gravina Access Project area. Four structural types were considered, i.e., a concrete box girder, a steel truss, a steel box girder and a cable-stayed structure. Each of those alternatives was prepared by a different firm. The cost estimator appointed by the client reviewed the proposed construction costs. The two most cost-effective alternatives were selected for the final design – **the lowest cost alternative was a prestressed concrete box girder** and the second lowest cost alternative a steel truss with concrete deck. Before the final design contract was awarded, the client abandoned the steel truss design alternative as undesirable and only the prestressed concrete box girder design was commissioned. The structure was successfully advertised in the summer of 2001 and the construction contract was awarded in the fall of 2001.

In 1994-95, T.Y. Lin International designed a prestressed concrete box alternative for the Tsable River Bridge for the new four-lane Vancouver Island Highway in British Columbia, Canada. While this bridge does not cross deep water and spans are relatively shorter, the bridge site is located in a very remote area of Vancouver Island, where climatic conditions are similar to the Gravina Access Project area. This 396-m (1,300-ft) long bridge, with four spans of 82, 118, 118, and 82 m (270, 387, 387 and 270 ft) respectively, piers of heights varying from 20 to 47 m (65 to 155 ft), and an overall deck width of 23 m (76 ft), is located in a seismically active area with a ground acceleration of .33g. It was prepared for bidding in two alternatives. Another firm designed the steel alternative. The client received five bids, two for concrete alternatives and three for steel. The contract was awarded **to the lowest bid for the prestressed concrete box** alternative. According to the client, the Ministry of Transportation and Highways, British Columbia "The bridge was bid and built for 10% less than the steel alternative and was completed on schedule with no claims."

This rather anecdotical evidence indicates the current trend in concrete vs. steel structure costs. This may or may not be confirmed at the time of this project construction.

## 6.0 Recommended Structure Type

This Preliminary Bridge Structures Technical Memorandum identifies and compares alternative structure types for various hard link alignments for the Gravina Access Project (see Table 10). The comparison of bridge types indicates that the preferred structure type, based on existing data, is a post-tensioned segmental concrete box girder for all of the alignments for both the main span and the approach spans (see Table 11). It has the lowest first cost, lowest annual operations and maintenance costs, and is rated equal or better that the other structure types in the other evaluation categories.

**Table 10. Comparison of Bridge Types**

| Bridge Type | Aviation Impacts: Part 77 | Aviation Impacts: Floatplane | Appearance | Environmental Impact | Cost Impacts (no contingency or mobilization): Constructibility | Cost Impacts: Dollars per Square meter (square foot) | Cost Impacts: % Higher than Type 1m | Maintenance |
|---|---|---|---|---|---|---|---|---|
| 1m—Post-Tensioned Seg. Concrete Box Girder – Main Spans | Mod. | Mod. | Good | Low | Low | $4,300 ($400) | 0 | Low |
| 1a—Post-Tensioned Seg. Concrete Box Girder – Approach Spans | Mod. | Mod. | Good | Low | Low | $3,300 ($300) | n.a. | Low |
| 2—Cable-Stayed Concrete | High | High | Good | Low | Low | $5,400 ($500) | 0 | Mod. |
| 3—Concrete Arch | High | High | Good | Mod. | High | $9,100 ($850) | 113 | Low |
| 4—Steel Box Girder | Mod. | Mod. | Good | Mod. | Mod. | $5,200 ($480)** | 20 | High |
| 5—Steel Plate Girder | Mod. | Mod. | Fair | Mod. | High | $5,600 ($520)** | 30 | High |
| 6—Steel Tied Arch* | High | High | Fair | Mod. | High | $6,500 ($600)** | 50 | High |
| 7—Movable* | Mod. | High | Poor | Mod. | Very High | $8,600 ($4,000) | 1000 | Very High |

*Mod. = Moderate*

** Sstructuretypes not recommended for further study..*

*** Costs from WSDOT Bridge Design Manual x 2.67*

**Table 11. Alignments vs. Bridge Types**

| Alignment | Concrete Segmental Box Girder | Concrete Cable Stayed | Concrete Arch | Steel Box Girder | Steel Plate Girder | Steel Tied Arch | Movable |
|---|---|---|---|---|---|---|---|
| C3(a): Airport Area to Signal Road | X | | | X | | | |
| C3(b): Airport Area to Signal Road | X | | | X | | | |
| C4: Airport Area to Cambria Drive Area | X | | | X | | | |
| D1: Airport Area | X | | | X | | | |
| F3: Pennock Island | X | X | | X | X | | |

Although the current comparison indicates that the concrete box girder is lowest cost, and recent anecdotal information from other similar structures recently constructed verifies this, the level of detail developed on the steel alternatives in this study is not adequate to eliminate the steel box or girder bridge type. When a final alignment is selected, alternate steel and concrete bridge types as indicated above should be compared as to construction cost, annual operation and maintenance cost, full life cycle cost, and other evaluation points to determine the most appropriate structure type for the selected alignment.

Based upon the information in this study, it is recommended that further study be accomplished when a final alignment(s) are selected. This work should include:

1. Additional detailed study and preliminary design for the concrete box girder alternative, developed to the TS&L level, including a detailed cost estimate sufficient to use for budgeting purposes.
2. Additional detailed study for the appropriate steel alternative, depending on alignment. This will be of sufficient detail to verify the present indication that the steel alternative is more costly for construction and life cycle costs.
3. Additional detailed study (real time simulation) of the navigational openings to verify the clearances necessary for Coast Guard Approval. In addition, assist the Coast Guard in surveying marine pilots and shipping interests to establish reasonable clearances requirements are needed in Tongass Narrows.
4. Perform site specific geotechnical exploration to establish foundation design parameters
5. Perform site specific topography and bathymetry to provide accurate base mapping for preliminary design.
6. Develop a Major Structures Selection Report to document the work accomplished and provide the final recommendation of structure type for the Gravina Access Project.

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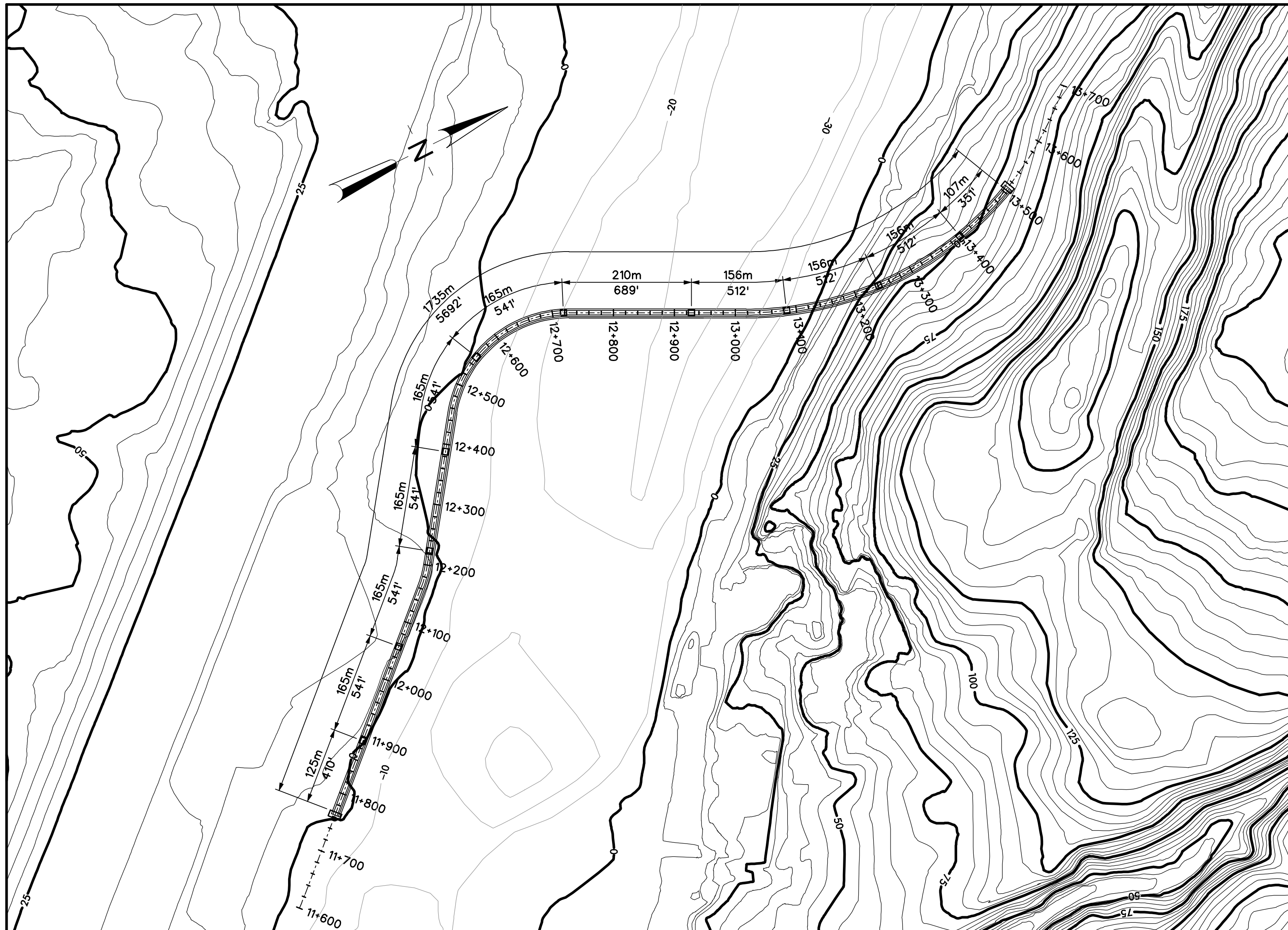


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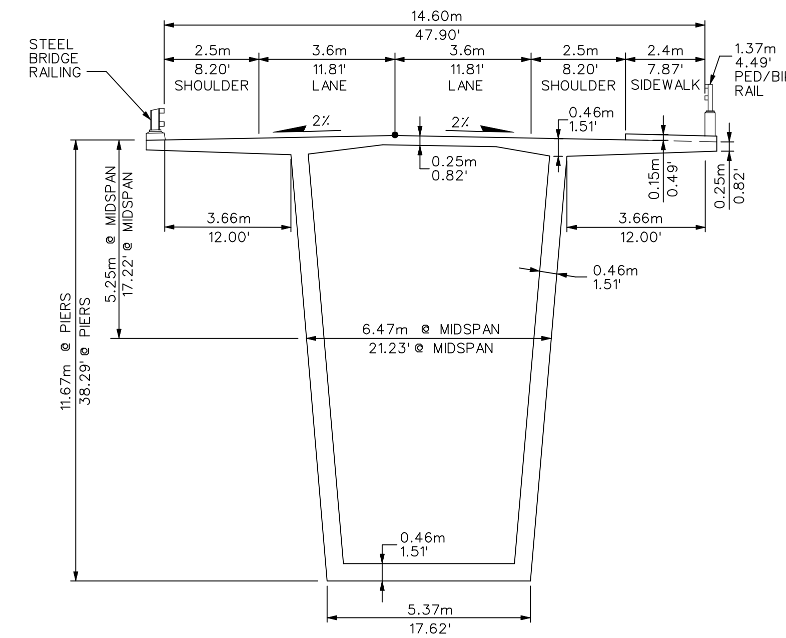


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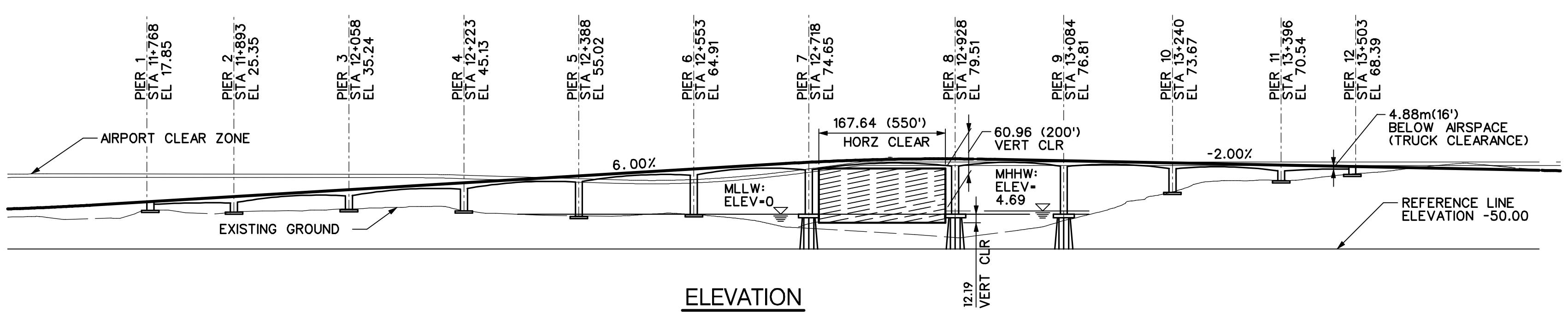


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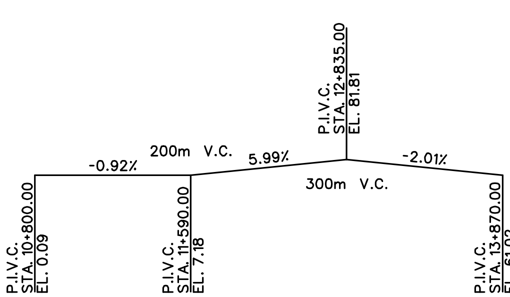


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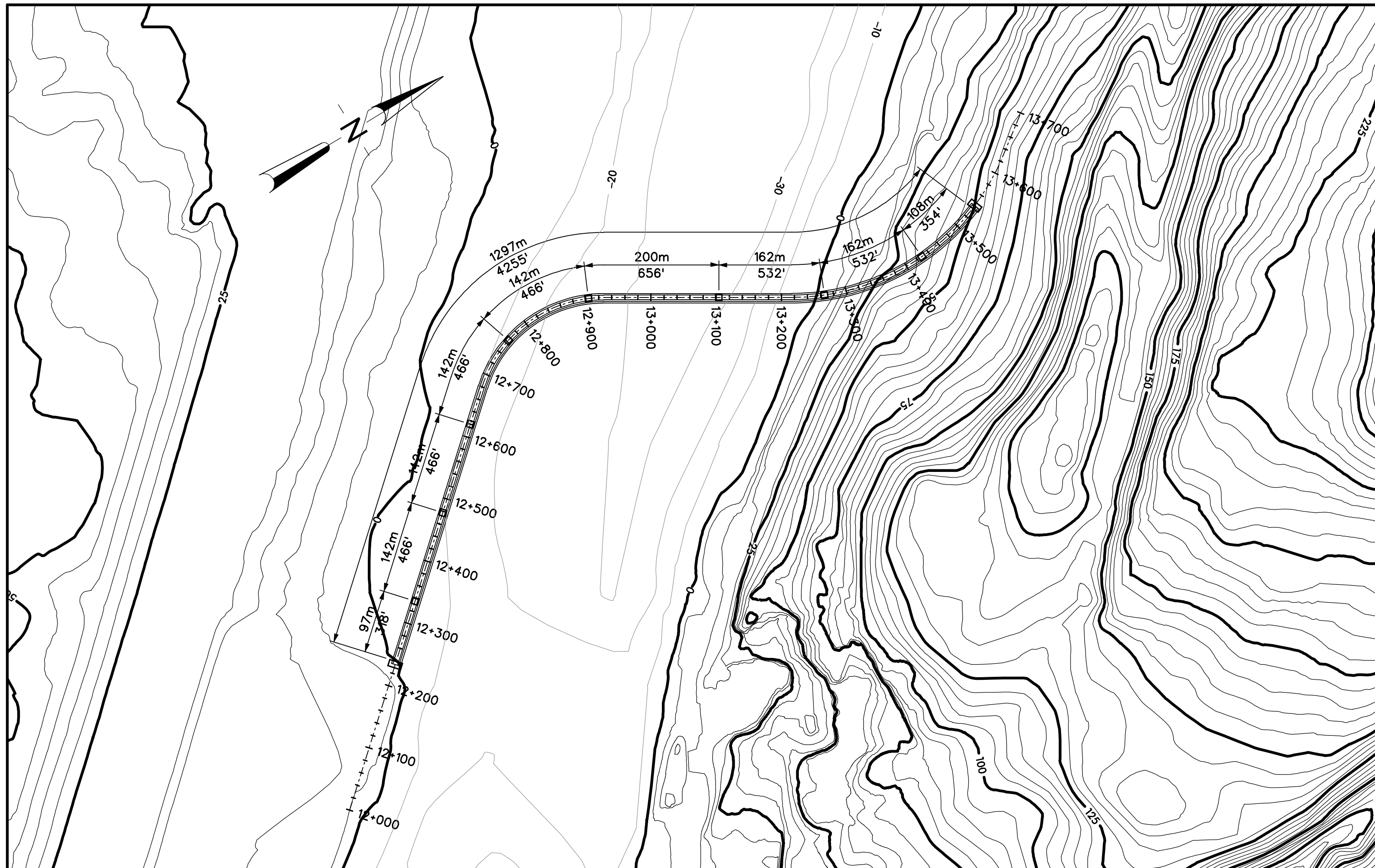


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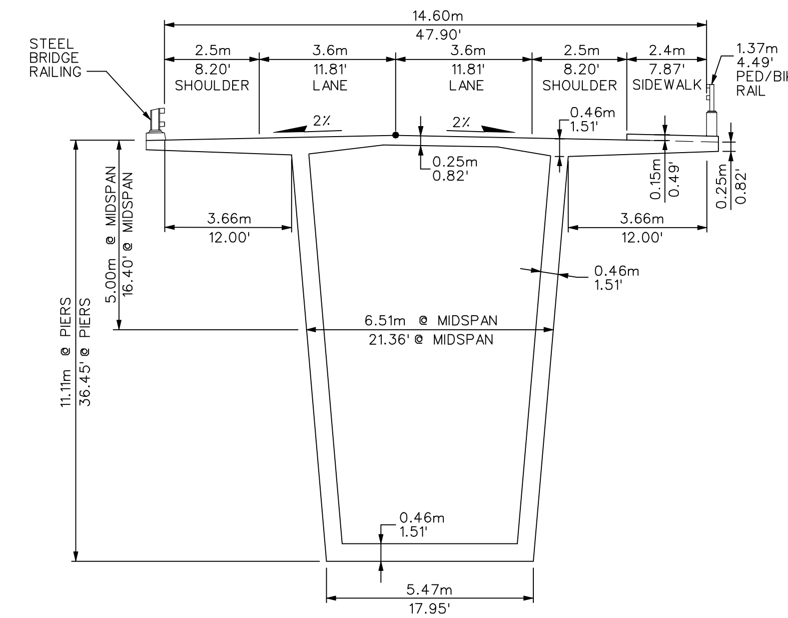


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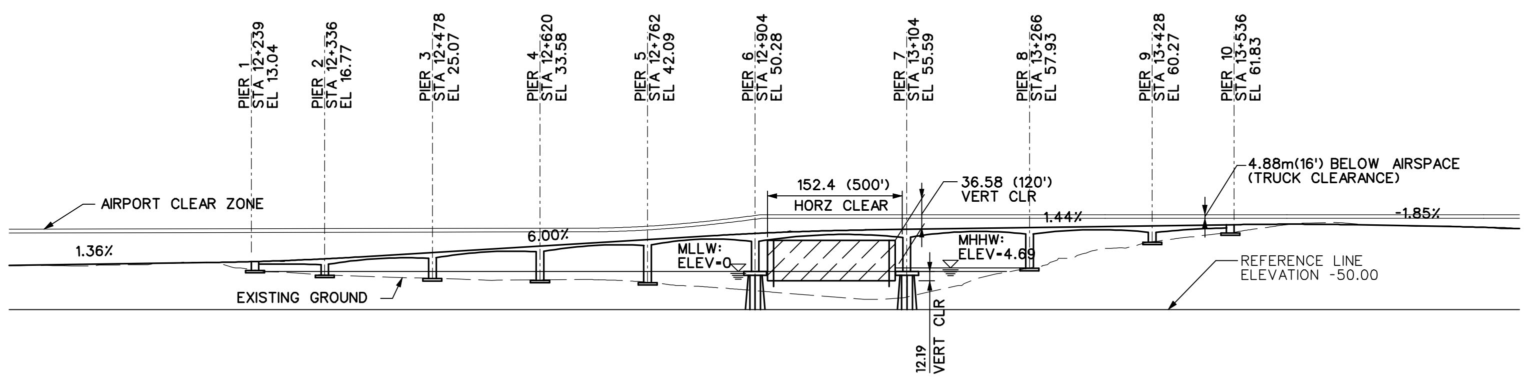


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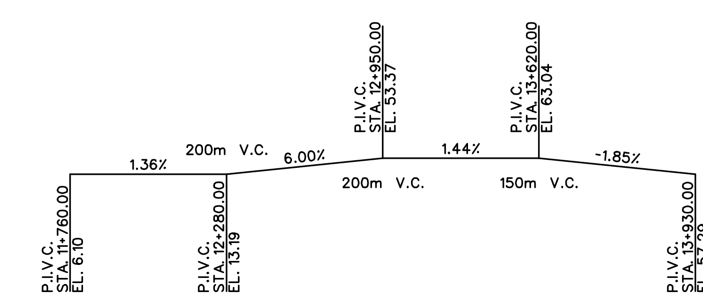


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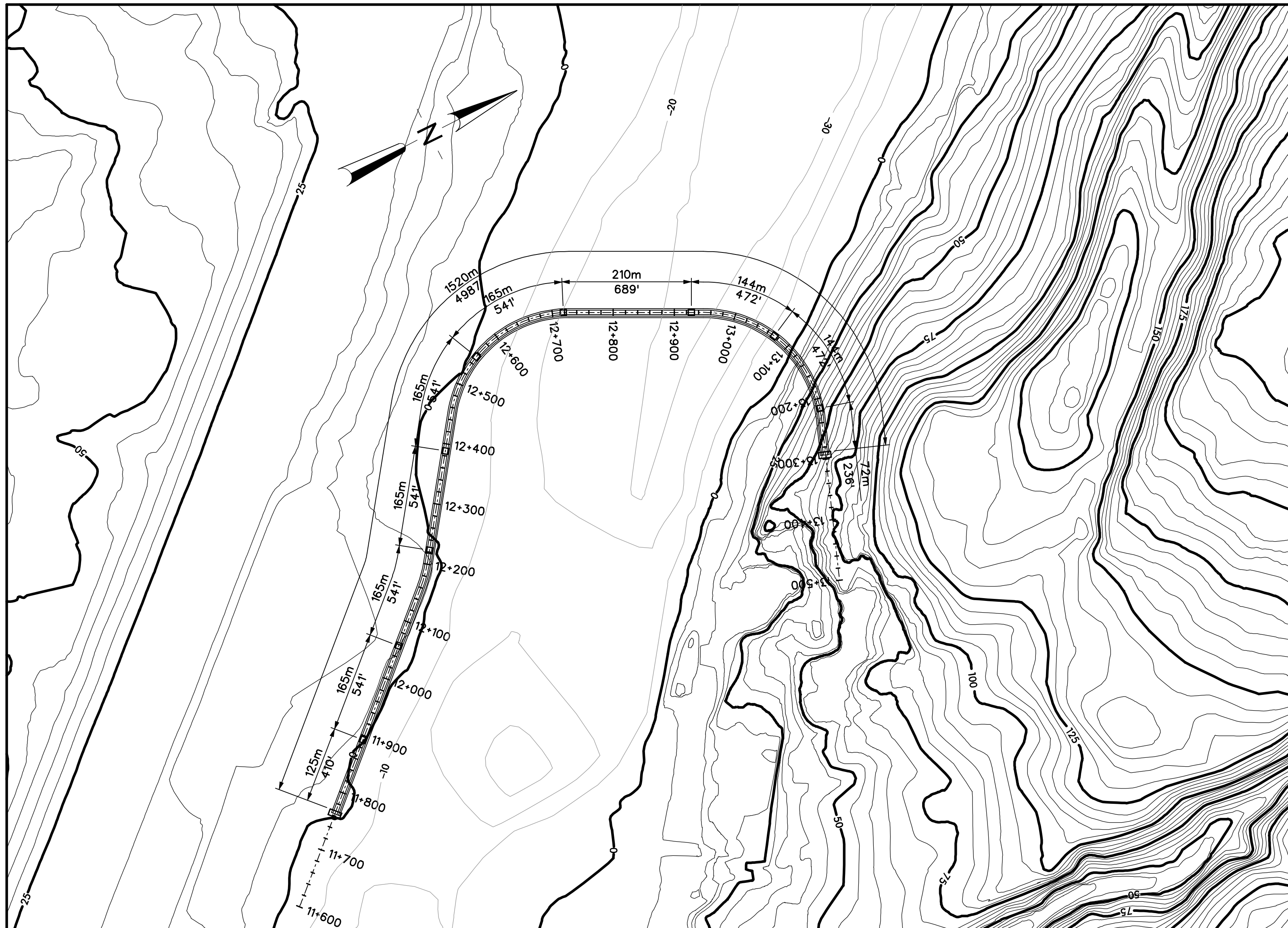


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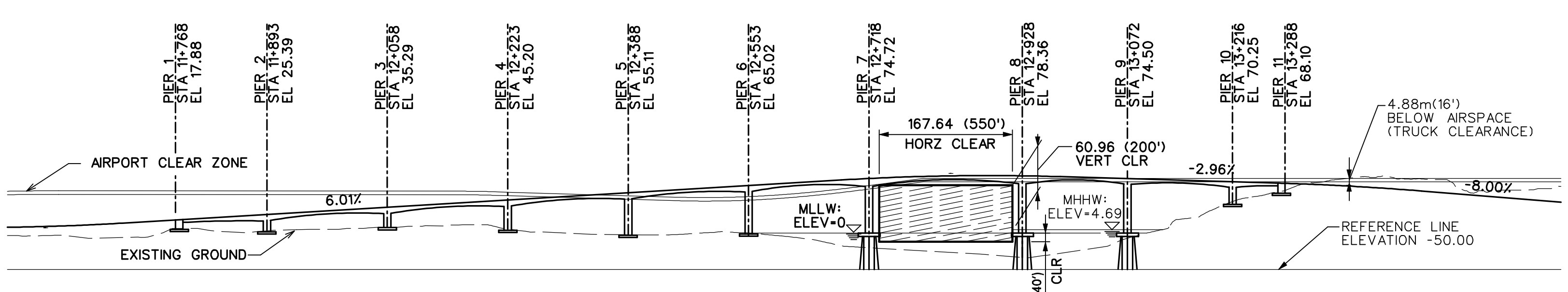


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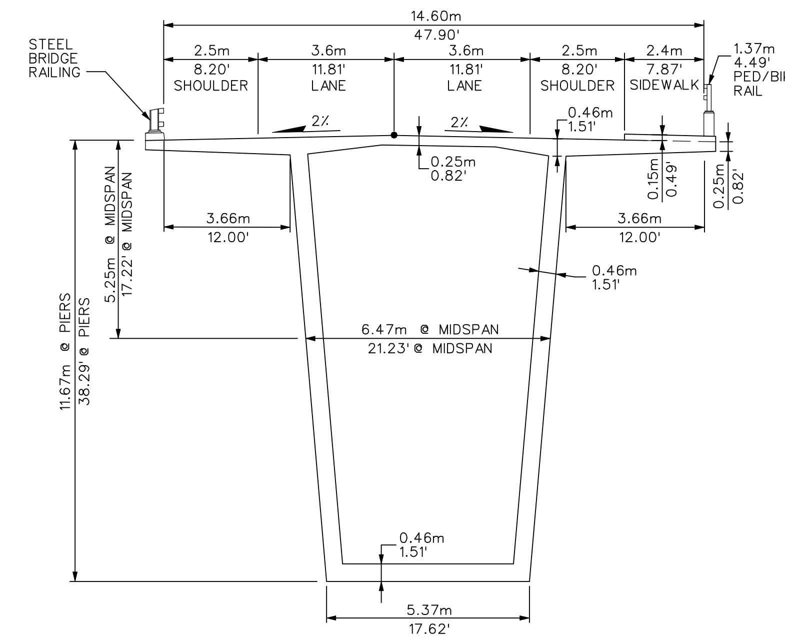


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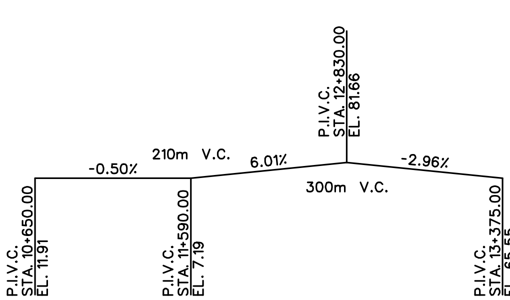


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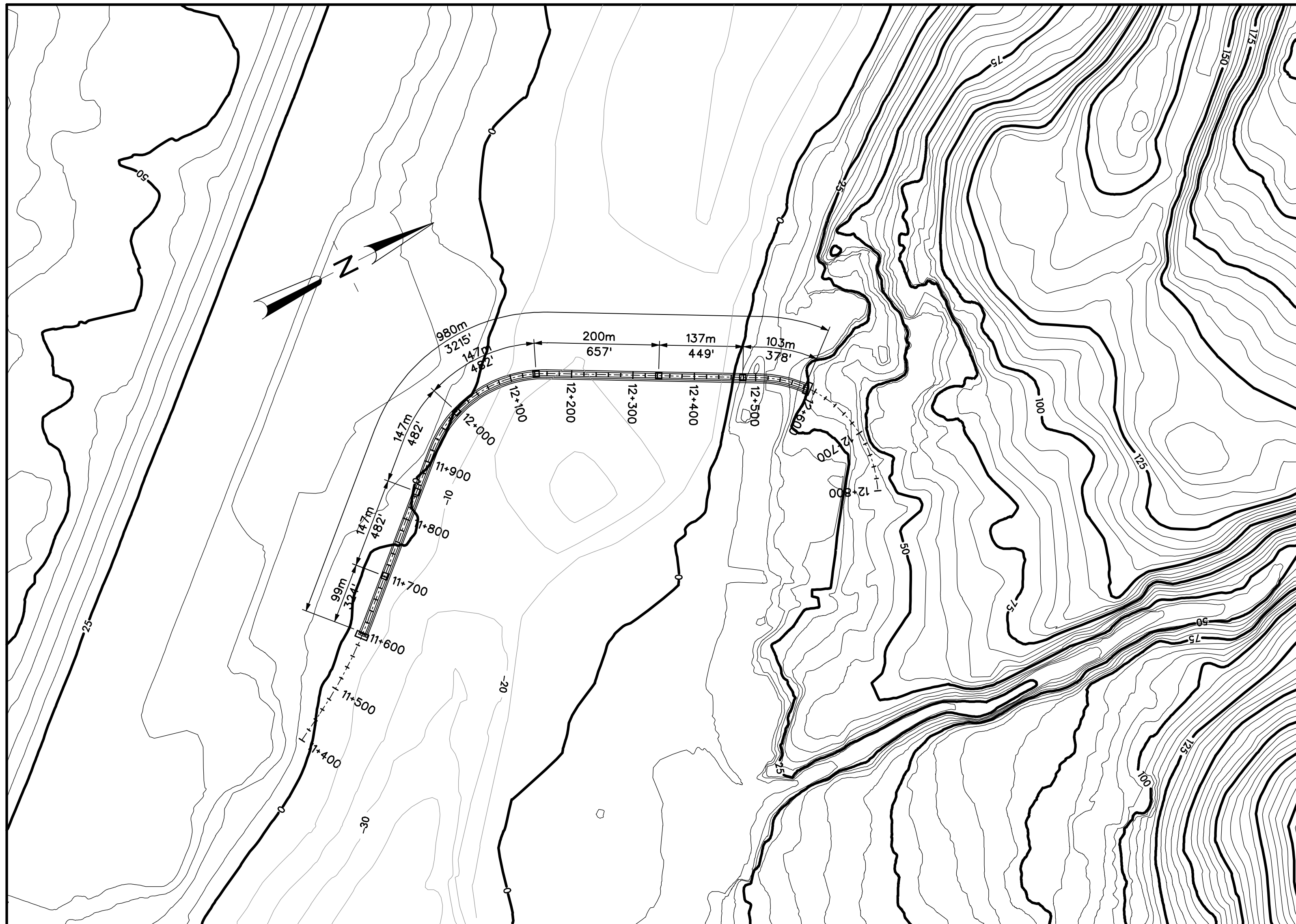


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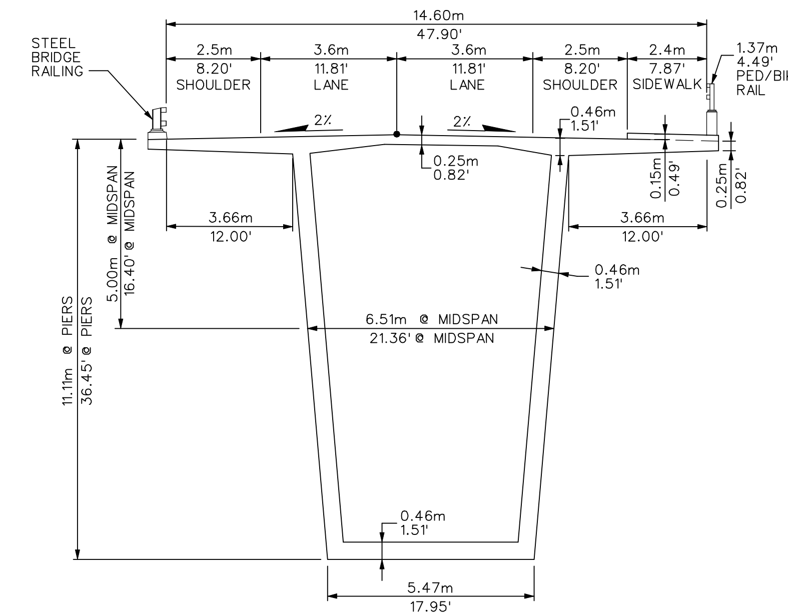


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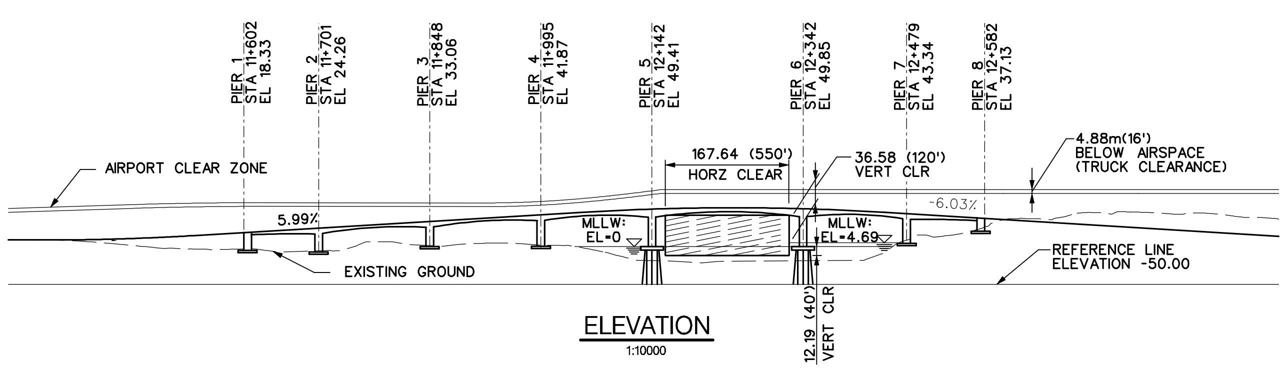


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VERTICAL DATUM: MLLW

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| 13+070.04 | 141° 43'03.61" | 195.00 | 561.82 | 482.32 | XXXX |
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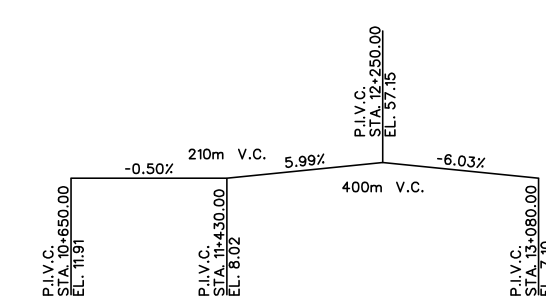



PROFILE GRADE

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

GRAVINA
ACCESS PROJECT
SHEET 4
ALTERNATIVE
D1-120

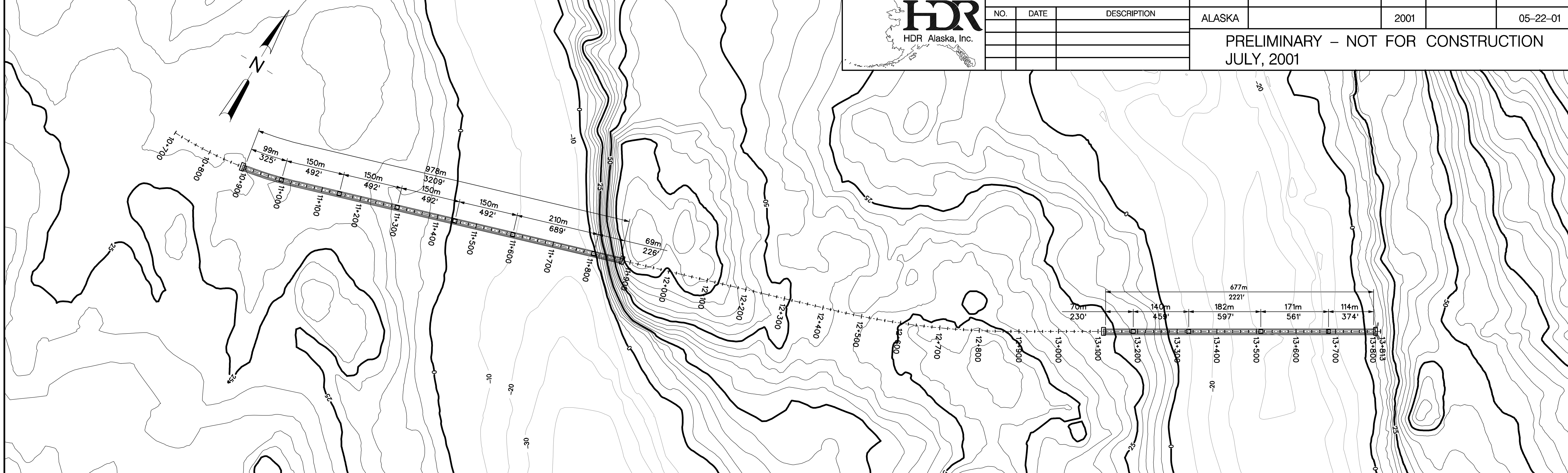
| FILENAME: XX | PLOT DATE: XX | DESIGNED BY: XX | DATE: XX |
|---|---|---|---|
| VPORT: XX | PLOT SCALE: XX | CHECKED BY: XX | DATE: XX |
| XREF: XX | SCALE: XX | DRAWN BY: XX | DATE: XX |

HDR Alaska, Inc.

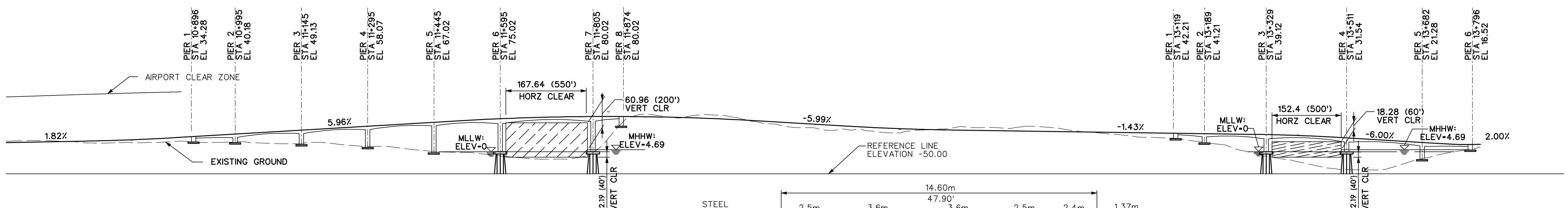
| REVISIONS | | |
|---|---|---|
| NO. | DATE | DESCRIPTION |
| | | |

| STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|
| ALASKA | | 2001 | | 05-22-01 |

PRELIMINARY – NOT FOR CONSTRUCTION
JULY, 2001

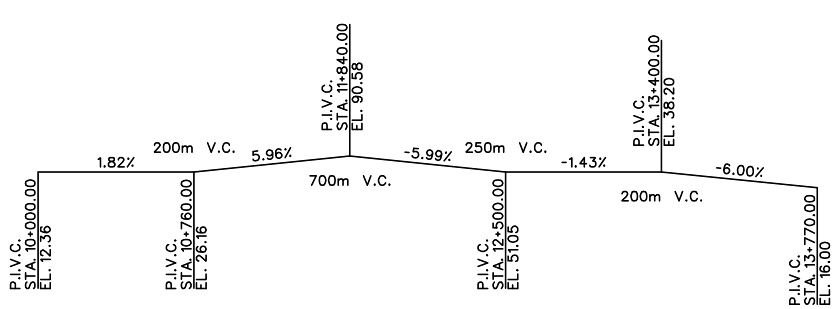



PLAN
1:10000

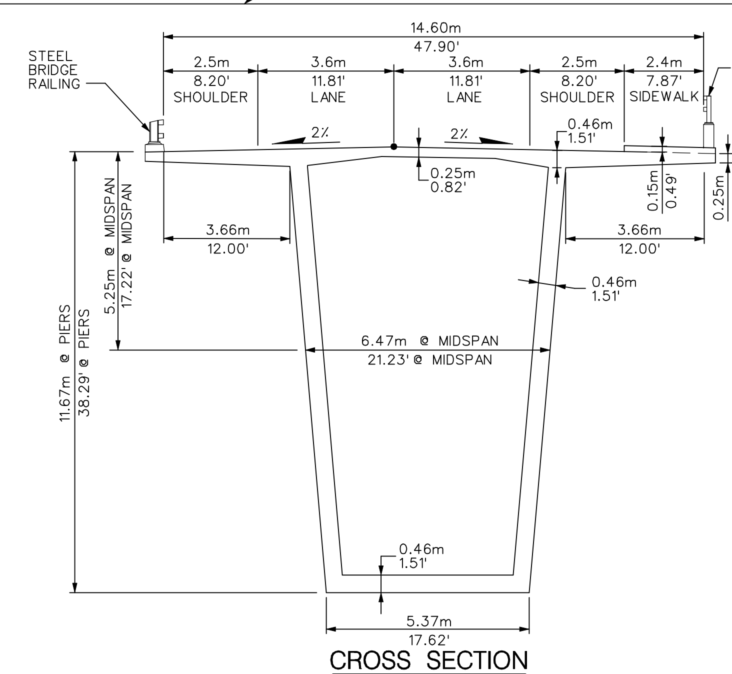



ELEVATION
1:10000

| CURVE DATA F3-200 | | | | | |
|---|---|---|---|---|---|
| P.I. STATION | DELTA | RADIUS | TANGENT | LENGTH | S |
| 12+661.34 | 13° 15'53.10" | 2000.00 | 232.55 | 463.03 | XXXX |
| | | | | | |
| | | | | | |
| | | | | | |




PROFILE GRADE




CROSS SECTION
1:200

| LINE | SURFACE | OFFSET |
|---|---|---|
| | Air_MLLW | 0.00 |
| | N_MLLW | 0.00 |
| | S_MLLW | 0.00 |

Scaled 1.0000 Times Ver.
Scaled 1.0000 Times Hor.

VERTICAL DATUM: MLLW

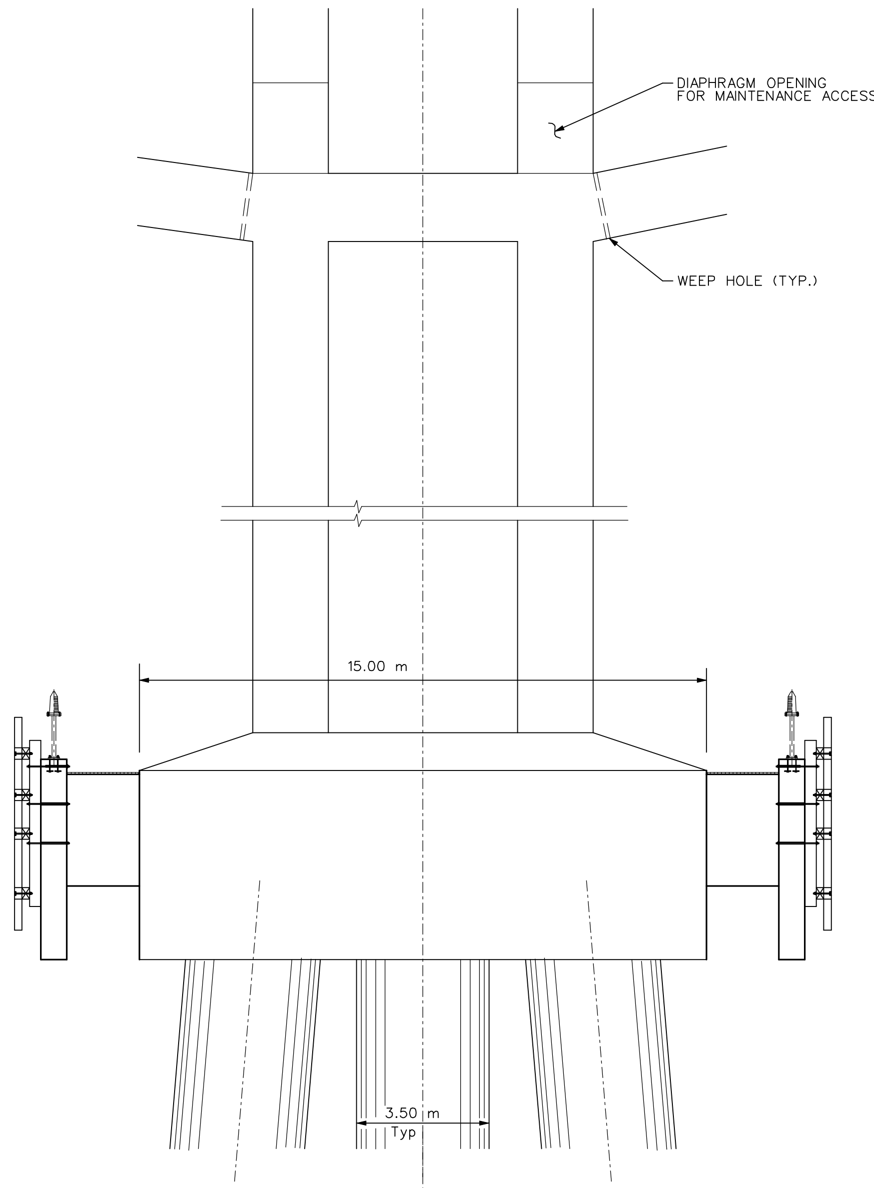
STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

GRAVINA
ACCESS PROJECT
SHEET 5
ALTERNATIVE
F3-200

| | | |
|---|---|---|
| FILENAME: XX | PLOT DATE: XX | DESIGNED BY: XX DATE: XX |
| VPORT: XX | PLOT SCALE: XX | CHECKED BY: XX DATE: XX |
| XREF: XX | SCALE: | DRAWN BY: XX DATE: XX |

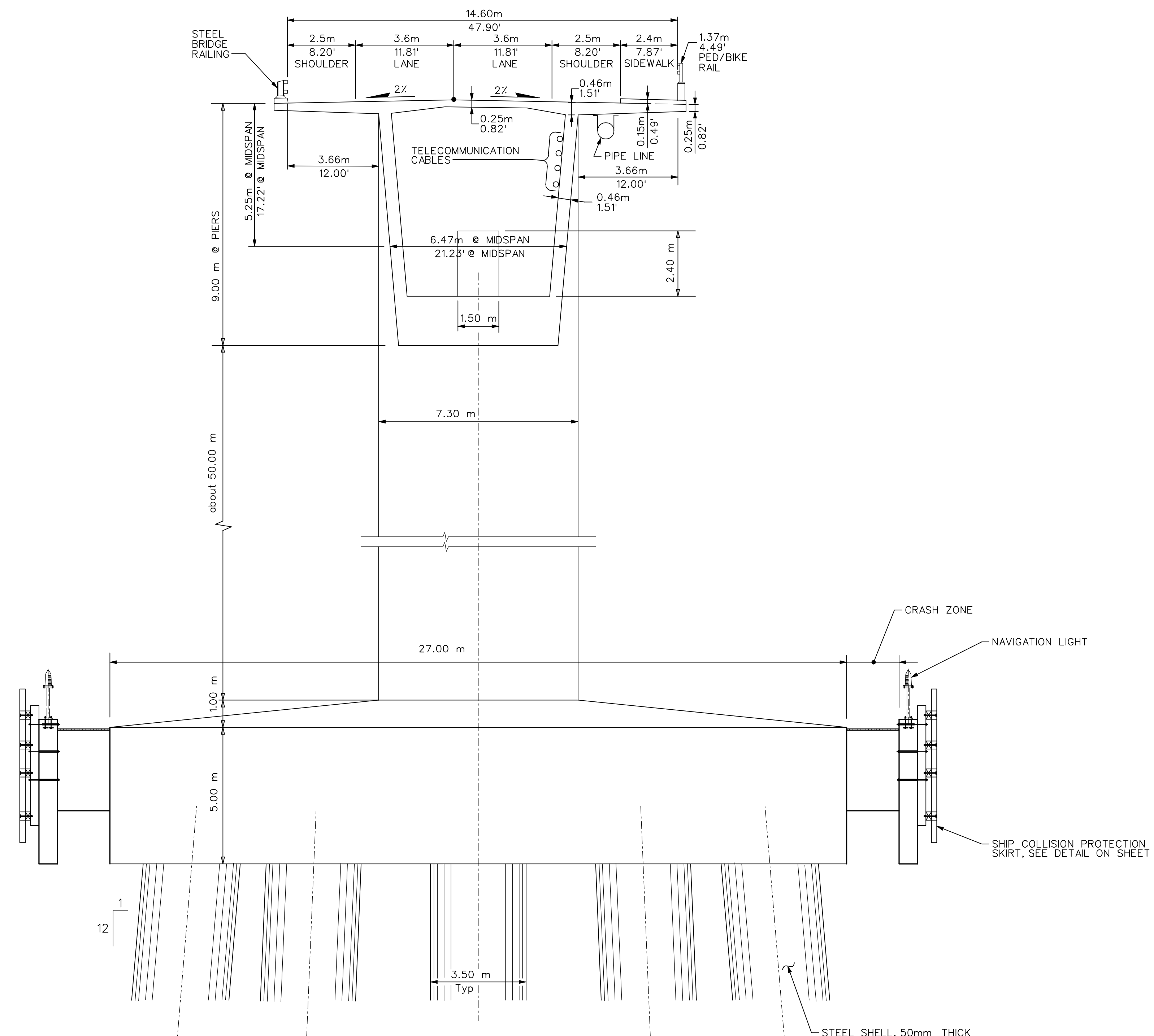
| HDR Alaska, Inc. | REVISIONS | | | STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|---|---|---|---|
| | NO. | DATE | DESCRIPTION | ALASKA | | 2001 | | 11-09-01 |

PRELIMINARY-NOT FOR CONSTRUCTION
NOVEMBER, 2001




LONGITUDINAL SECTION

1:200




TRANSVERSE SECTION

1:200

| FILENAME: XX | PLOT DATE: XX | DESIGNED BY: XX | DATE: XX |
|---|---|---|---|
| VPORT: XX | PLOT SCALE: XX | CHECKED BY: XX | DATE: XX |
| XREF: XX | SCALE: XX | DRAWN BY: XX | DATE: XX |

grav_sht06.dgn

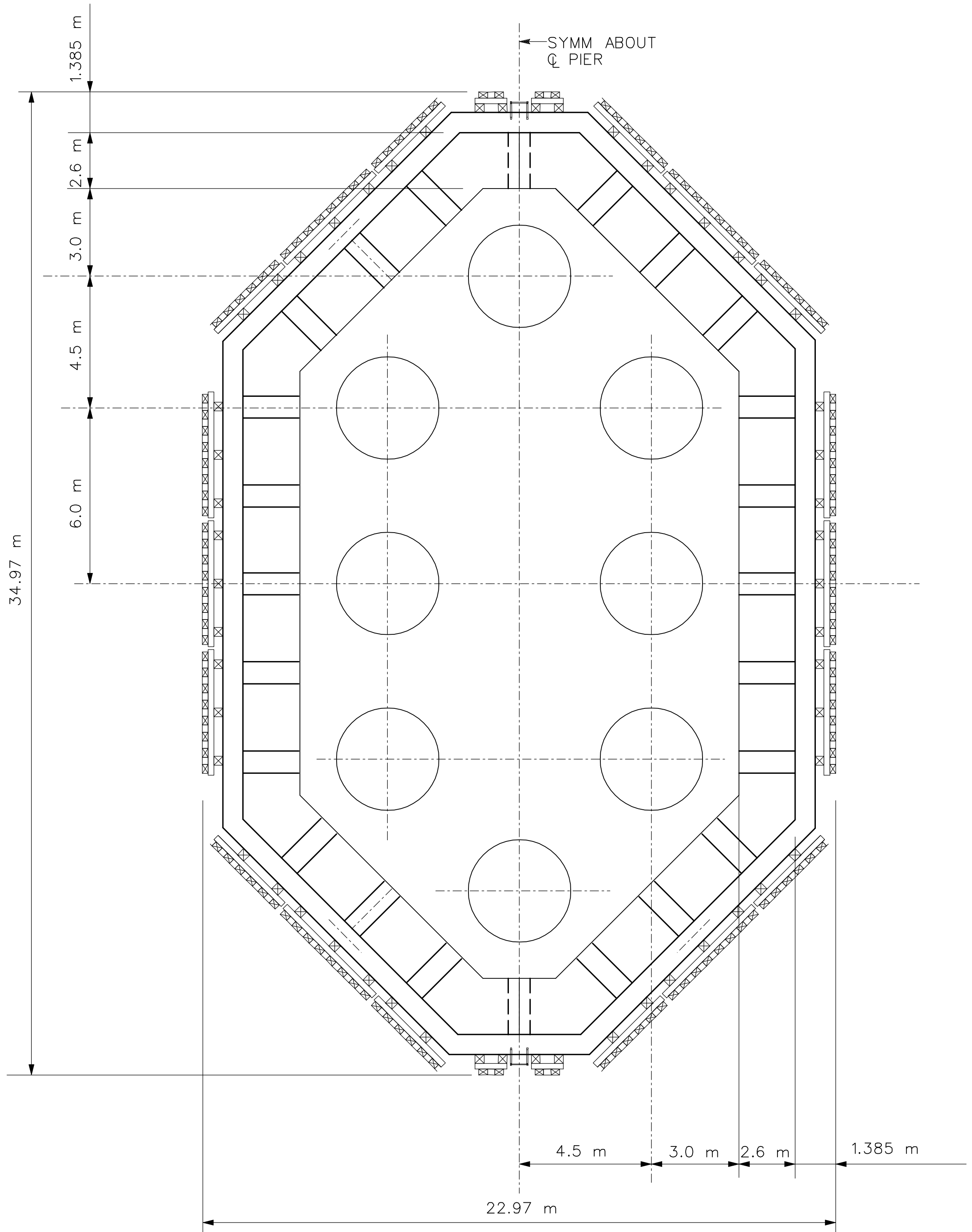
STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

GRAVINA
ACCESS PROJECT
SHEET 6
FIXED PIER IN
SHIPPING CHANNEL

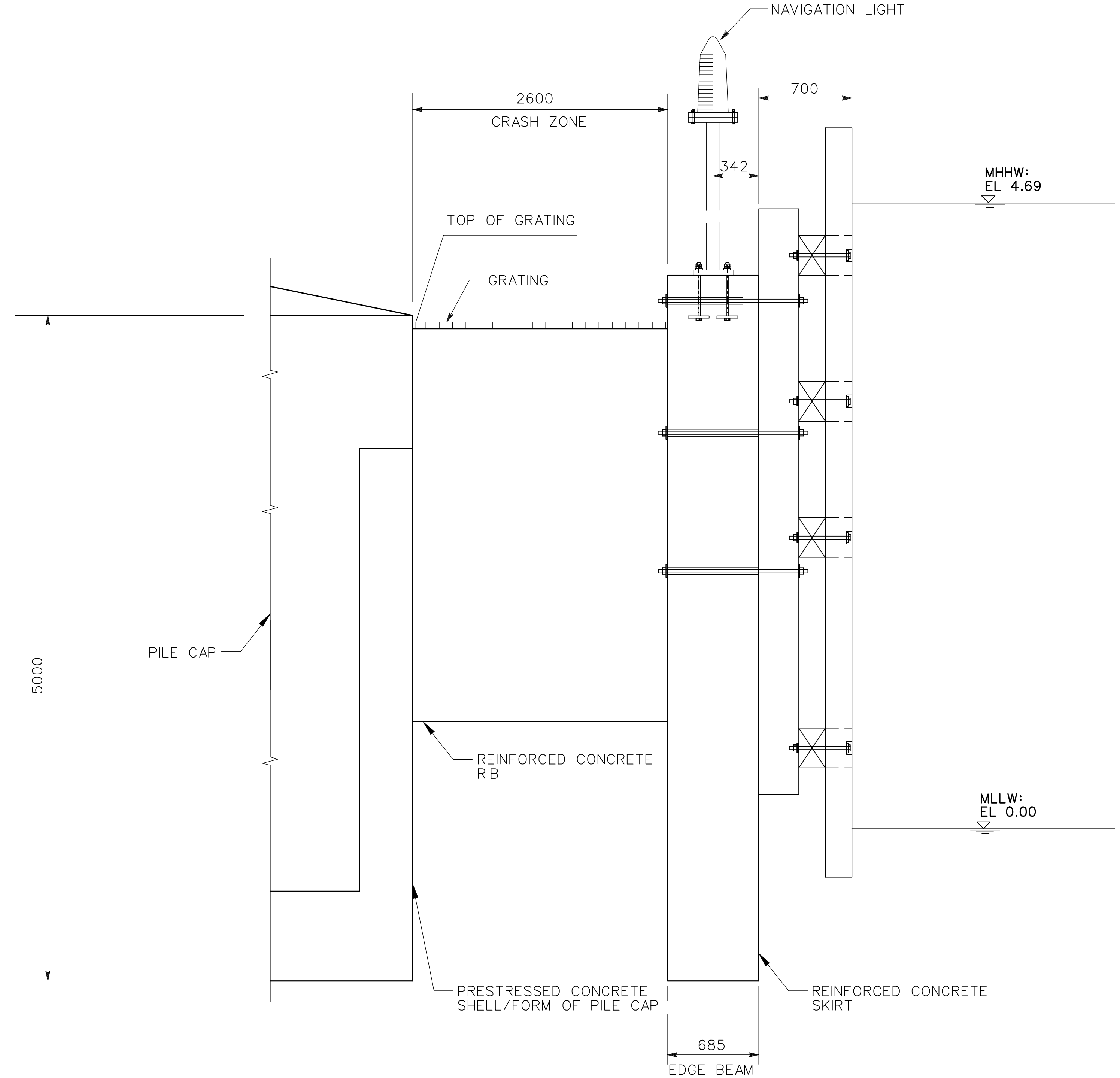
| REVISIONS | | | STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|---|---|---|
| NO. | DATE | DESCRIPTION | ALASKA | | 2001 | | 11-09-01 |

HDR Alaska, Inc.

PRELIMINARY-NOT FOR CONSTRUCTION
NOVEMBER, 2001




PLAN
1:100




SECTION
1:20

| | |
|---|---|
| FILENAME: XX | |
| VPORT: XX | |
| XREF: XX | |
| PLOT DATE: XX | |
| PLOT SCALE: XX | |
| SCALE: XX | |
| DESIGNED BY: XX | DATE: XX |
| CHECKED BY: XX | DATE: XX |
| DRAWN BY: XX | DATE: XX |

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

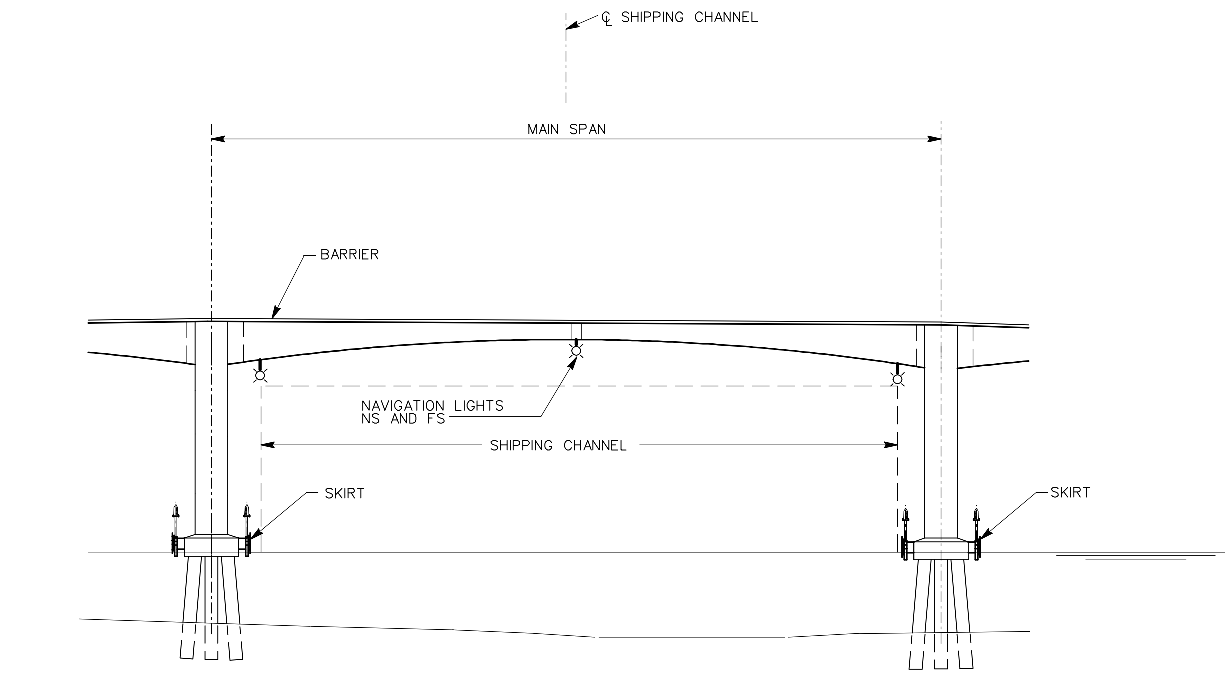
GRAVINA
ACCESS PROJECT
SHEET 7
FOUNDATION IN
SHIPPING CHANNEL

GRAV_SHT07.dgn

HDR Alaska, Inc.

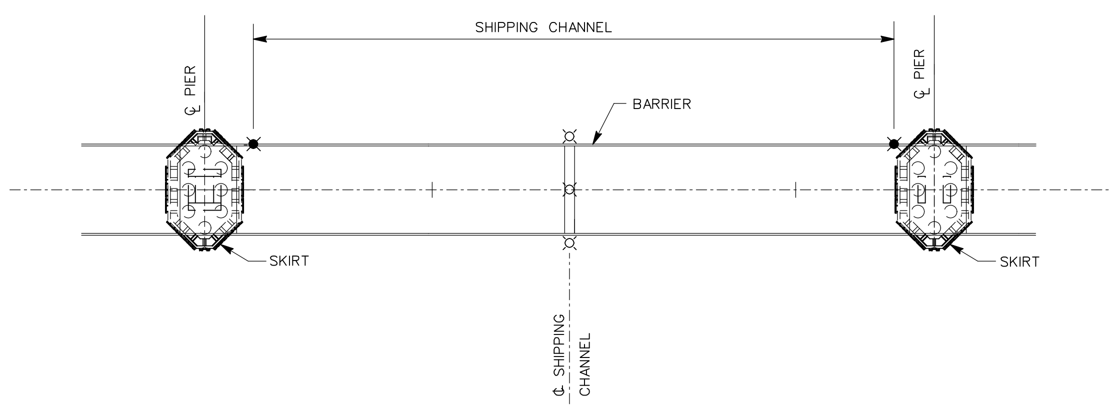
| REVISIONS | | | STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|---|---|---|
| NO. | DATE | DESCRIPTION | ALASKA | | 2001 | | 11-09-01 |

PRELIMINARY-NOT FOR CONSTRUCTION
NOVEMBER, 2001




DEVELOPED ELEVATION

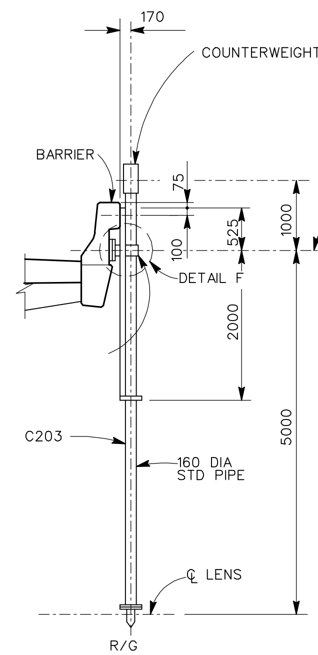
1:1000




PLAN

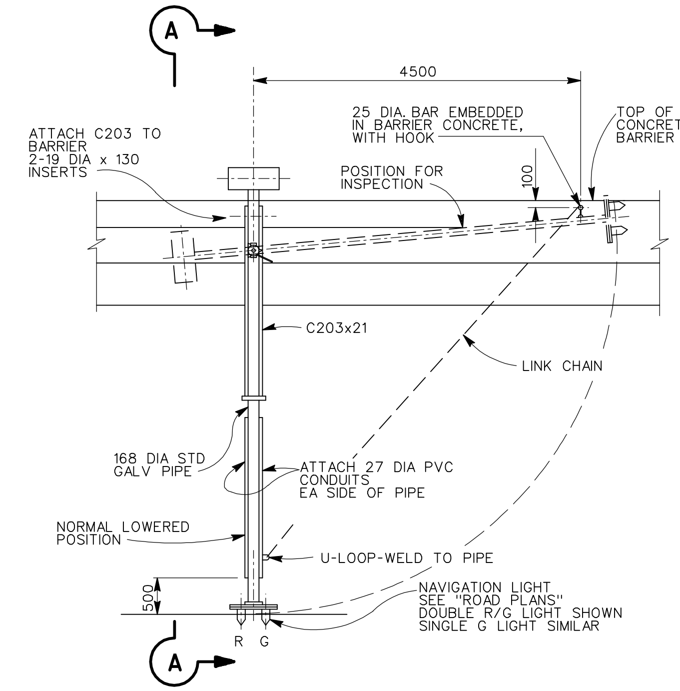
1:1000

Dimensions are in meters unless otherwise shown




SECTION A-A

1:50




DETAIL

1:50

NAVIGATION LIGHT FOR MAIN SHIPPING CHANNEL.
DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN.

FILENAME: XX
VPORT: XX
XREF: XX

PLOT DATE: XX
PLOT SCALE: XX
SCALE: XX

DESIGNED BY: XX DATE: XX
CHECKED BY: XX DATE: XX
DRAWN BY: XX DATE: XX

grav_sht08.dgn

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

GRAVINA
ACCESS PROJECT
SHEET 8
STRUCTURE PLAN &
SHIPPING CHANNEL DETAILS

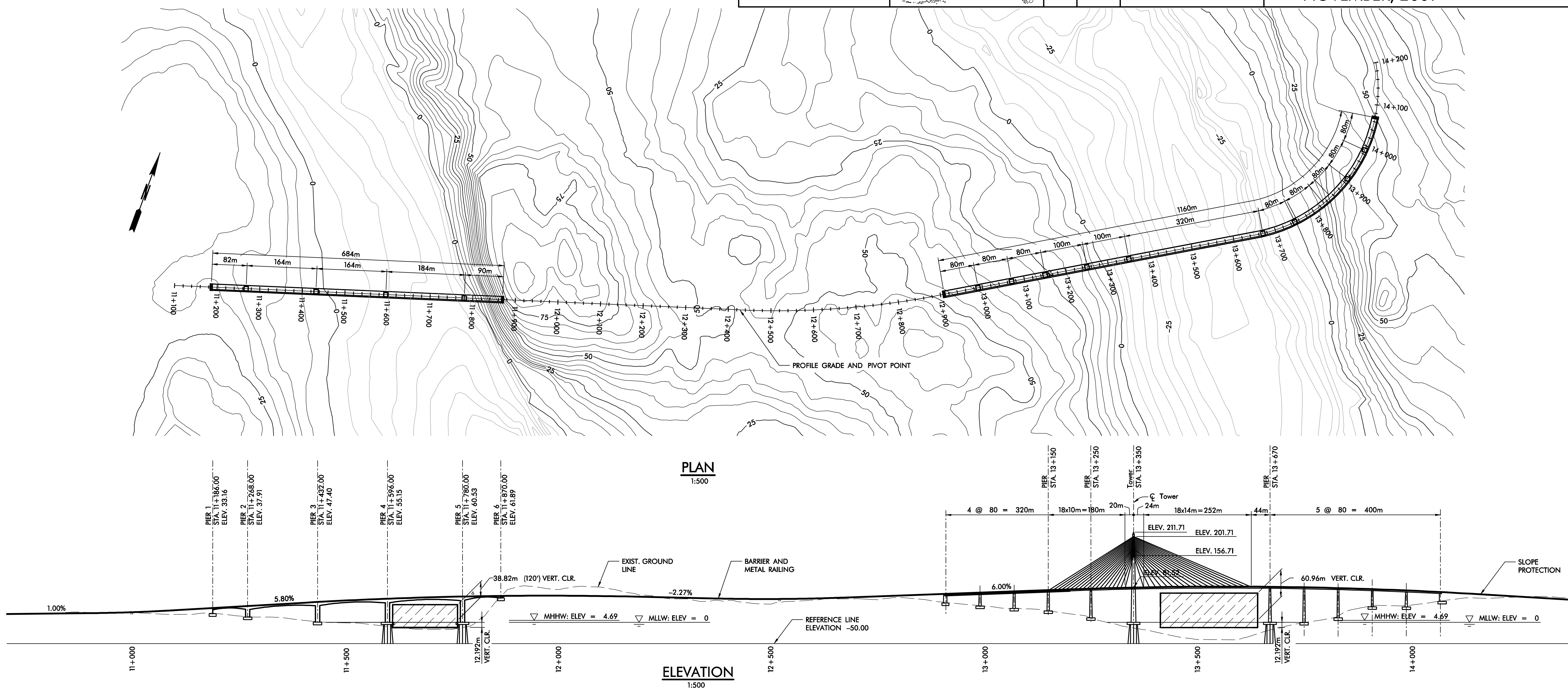
| T.Y. LIN INTERNATIONAL | HDR Alaska, Inc. | | | |
|---|---|---|---|---|

**REVISIONS**

| NO. | DATE | DESCRIPTION |
|---|---|---|
| | | |
| | | |
| | | |
| | | |

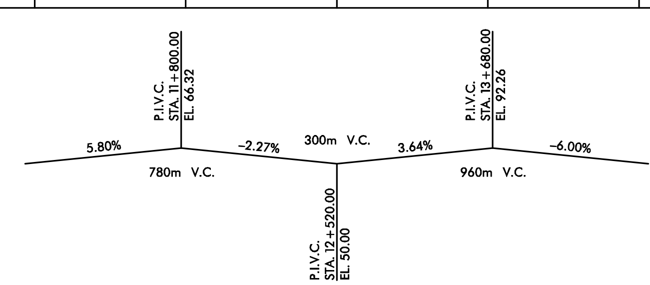
| STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|
| ALASKA | | 2000 | | 02–21–00 |

PRELIMINARY – NOT FOR CONSTRUCTION
NOVEMBER, 2001




## CURVE DATA F1

| P.I. STATION | DELTA | RADIUS | TANGENT | LENGTH | S |
|---|---|---|---|---|---|
| 12+661.34 | 13^15'53.10" | 2000.00 | 232.55 | 463.03 | XXXX |
| 14+119.36 | 105^55'14.19" | 350.00 | 463.80 | 647.03 | XXXX |
| | | | | | |
| | | | | | |




PROFILE GRADE

| LINE | SURFACE | OFFSET |
|---|---|---|
| ——— | Air_MLLW | 0.00 |
| — — — | N_MLLW | 0.00 |

Scaled 1.00 Times Ver.
Scaled 1.00 Times Hor.

VERTICAL DATUM: MLLW

FILENAME: XX 07072.144\DRAWINGS\BRIDGE-STRUCTURE\
VPORT: XX
XREF: XX
PLOT DATE: XX
PLOT SCALE: XX
SCALE: XX
DESIGNED BY: XX DATE: XX
CHECKED BY: XX DATE: XX
DRAWN BY: XX DATE: XX

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

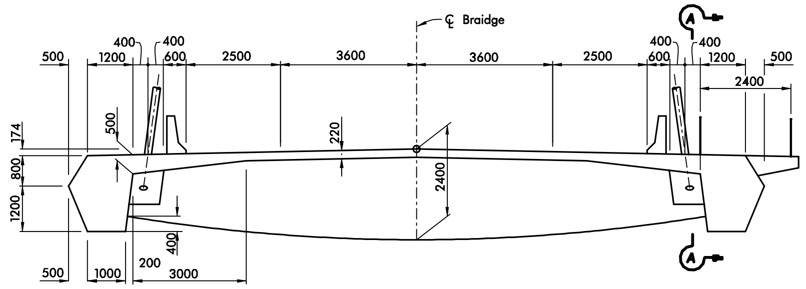
GRAVINA
ACCESS PROJECT
SHEET 9
CABLE–STAYED OPTION

File: grav_sht09.dgn Date: 12–11–2001

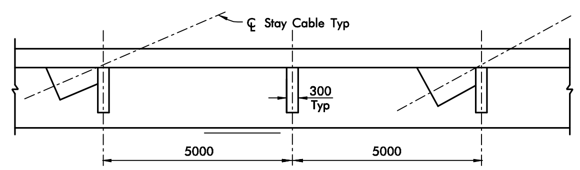
| REVISIONS | | | STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|---|---|---|
| NO. | DATE | DESCRIPTION | ALASKA | | 2001 | | 11–09–01 |

HDR Alaska, Inc.

PRELIMINARY–NOT FOR CONSTRUCTION
NOVEMBER, 2001

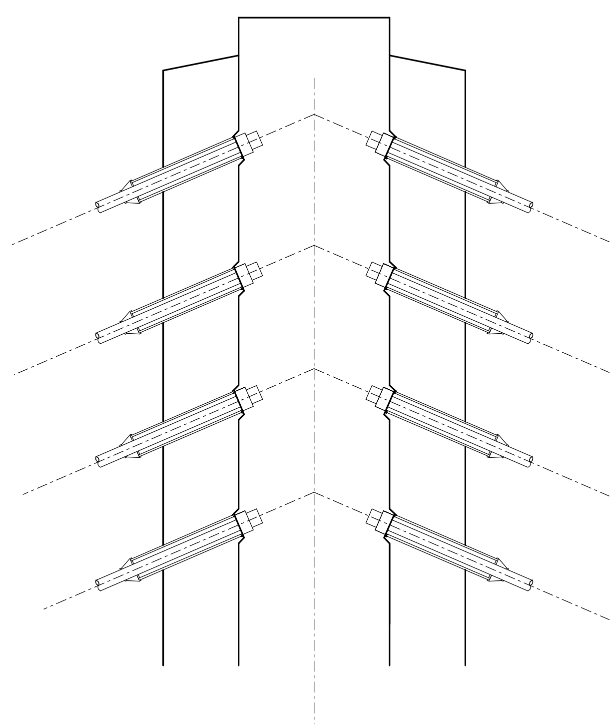



TYPICAL DECK CROSS SECTION
1:100

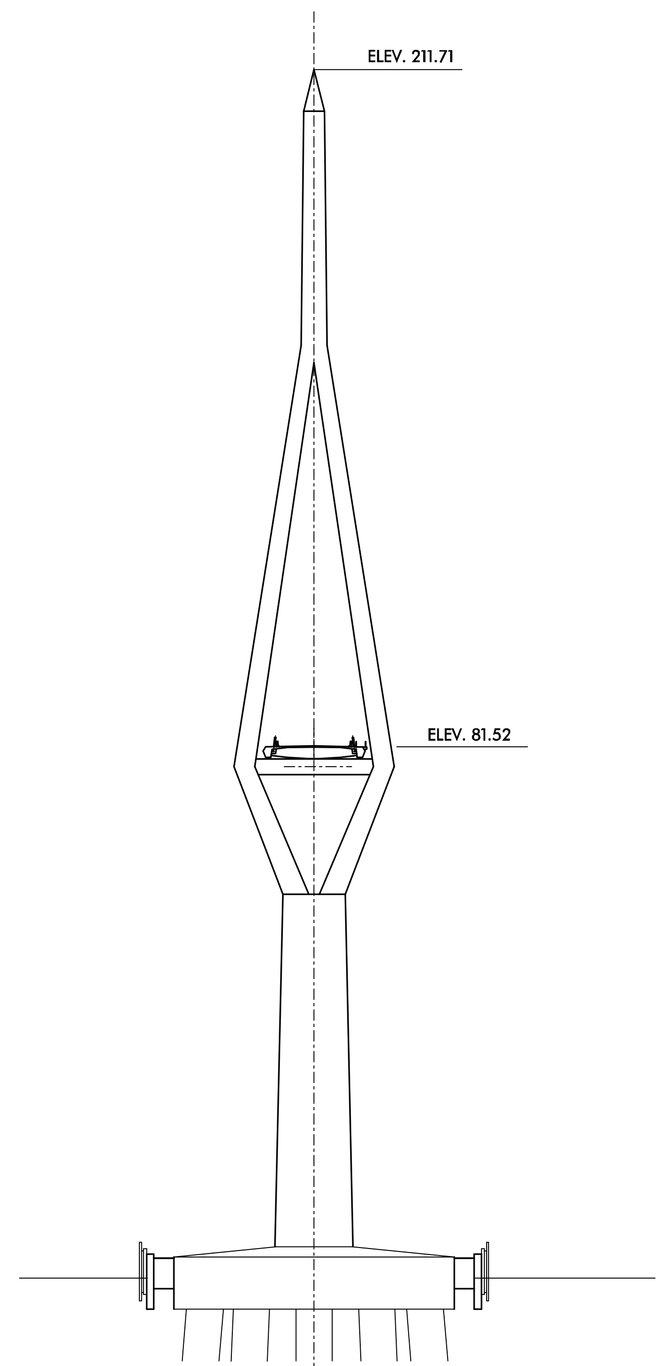



SECTION A–A
1:100

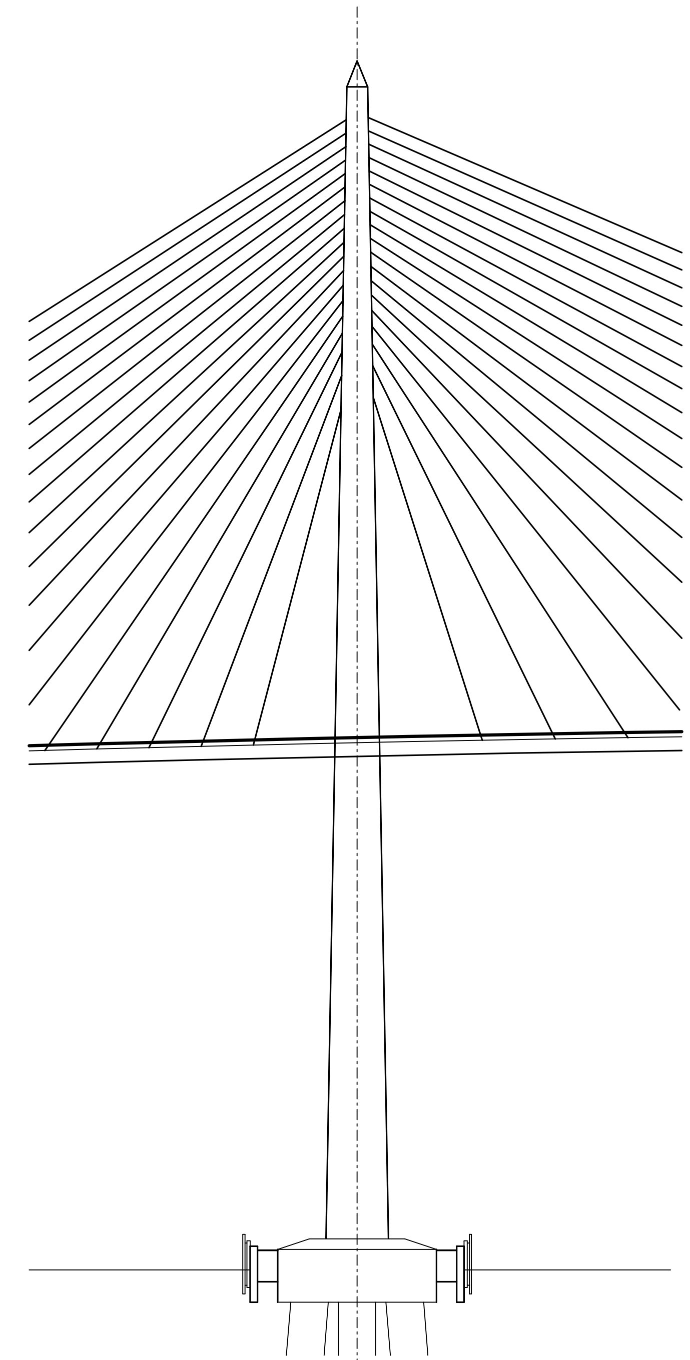
File: grav_sht13.dgn Date: 11–15–2001



STAYS TOWER LAYOUT




TOWER ELEVATION



TOWER ELEVATION

| | | |
|---|---|---|
| FILENAME: XX | PLOT DATE: XX | DESIGNED BY: XX | DATE: XX |
| VPORT: XX | PLOT SCALE: XX | CHECKED BY: XX | DATE: XX |
| XREF: XX | SCALE: | DRAWN BY: | DATE: XX |

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

GRAVINA
ACCESS PROJECT
SHEET 10
CABLE STAY
TYPE DETAILS

grav_sht10.dgn

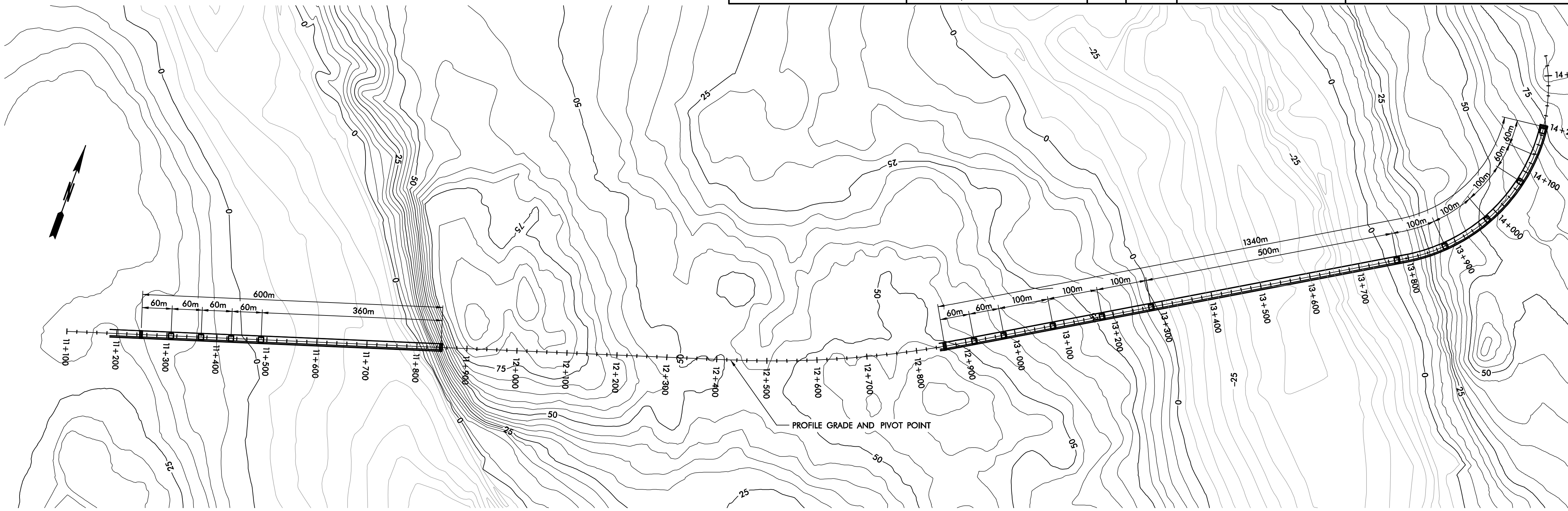
| REVISIONS | | |
|---|---|---|
| NO. | DATE | DESCRIPTION |
| | | |
| | | |
| | | |
| | | |

T.Y. LIN INTERNATIONAL

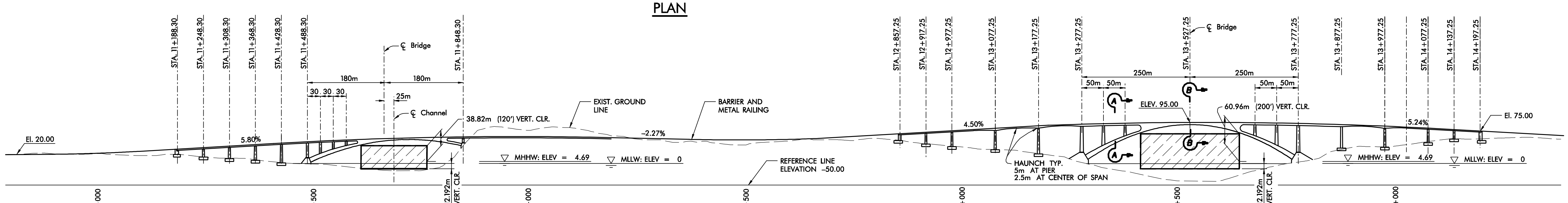
HDR Alaska, Inc.

| STATE | PROJECT DESIGNATION | YEAR | SHEET NO. | DATE |
|---|---|---|---|---|
| ALASKA | | 2000 | | 02-21-00 |

PRELIMINARY – NOT FOR CONSTRUCTION
NOVEMBER, 2001



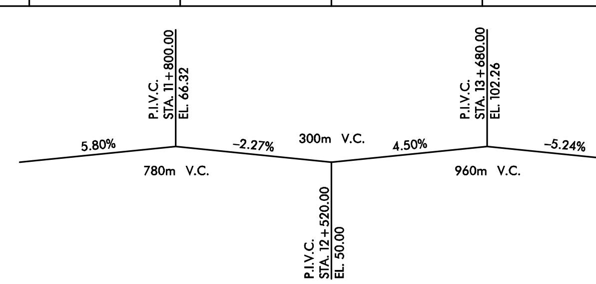

## PLAN



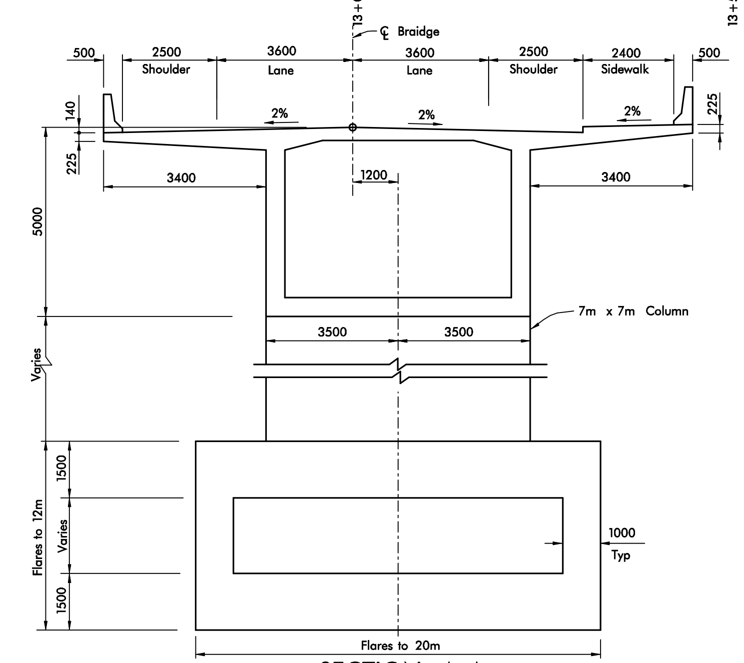

## ELEVATION

## CURVE DATA F1

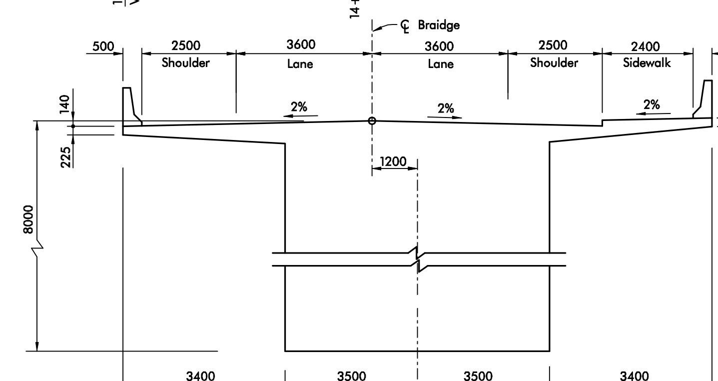
| P.I. STATION | DELTA | RADIUS | TANGENT | LENGTH | S |
|---|---|---|---|---|---|
| 12+661.34 | 13^15'53.10" | 2000.00 | 232.55 | 463.03 | XXXX |
| 14+119.36 | 105^55'14.19" | 350.00 | 463.80 | 647.03 | XXXX |
| | | | | | |
| | | | | | |




## PROFILE GRADE




## SECTION A–A
1:100




## SECTION B–B
1:100

| LINE | SURFACE | OFFSET |
|---|---|---|
| ———— | Air_MLLW | 0.00 |
| — — — | N_MLLW | 0.00 |

Scaled 1.00 Times Ver.
Scaled 1.00 Times Hor.

VERTICAL DATUM: MLLW

File: grav_sht11.dgn    Date: 12-11-2001

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

GRAVINA
ACCESS PROJECT
SHEET 11
ARCH OPTION

FILENAME: XX 07072.144\DRAWINGS\BRIDGE-STRUCTURE\TYLIN\
VPORT: XX
XREF: XX
PLOT DATE: XX
PLOT SCALE: XX
SCALE: XX
DESIGNED BY: XX DATE: XX
CHECKED BY: XX DATE: XX
DRAWN BY: XX DATE: XX