Appendix B

Technical and Engineering

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1. Technical/Engineering

This section of the report will detail the technical efforts and findings for the Alberta to Alaska Railway. It will present the methodologies, recommendations and findings with respect to the route selection and alignment, the earthworks, the hydrology, the tunnels, the geomorphology, and the bridges and structures.

1.1 Mapping

The starting point for the project was collecting as much data as possible from both public sources and through free data available. The scope of the data identification was searching for all available topographic information for the Alberta to Alaska Railway. The initial routes that were examined were proposed in the previous Al-Can study completed in 2005 and 2007, as well as a high level review of the existing topography by the Mapping and Track & Civil teams. The proposed rail route corridor that was studied begins in Fort McMurray, Alberta and continues through to Delta Junction, Alaska.

The data sets that were used are as follows:

- 10 Metre contours in DXF format
- 15 metre Landsat orthophoto Mosaics in Tiff and Jpeg Formats
- 3D black and white hillshades in Tiff format
- Vertical Elevation Color coded hillshades in Tiff format
- Road Network of entire corridor in DXF format
- Sheet Index in DXF and PDF formats
- Geographic Place Names in DXF format
- Al-Can route Alignments in DXF format
- Preliminary - Proposed route alignment from Fort Nelson to Fort McMurray – in DXF format.

All data was collected from existing base mapping, satellite imagery, and DEM. The entire corridor was broken into fifty seven (57) sheets or tiles in order to make the files sizes and data manipulation manageable. The accuracies of the various data sources vary between locations with the least accurate data being from Alaska. The following table shows the various data sources and approximate accuracies.

Table 1 Data Sources and Accuracy

Data Source	Accuracy	Resolution/Interval
Satellite Imagery	20 metre	15 Metre
Alignment/Profile	N/A	N/A
Contours-Alberta Contours-BC/Yukon Contours-Alaska	+/- 5 metre +/- 15 metre +/- 50 metre	10 metre interval 10 metre interval 10 metre interval
3D - Hillshade	N/A	Raster Data
DEM – Alberta	+/- 5 metre	100 metre interval
DEM – B.C.	+/- 15 - 20 metre	Raster Data
DEM – Yukon	+/- 15 - 20 metre	Raster Data
DEM - Alaska	+/- 50 metre	Raster Data

The data that was used was free information provided by various government agencies in both Canada and the United States. It was determined that these data sources provided enough accuracy for this level of study and allowed for the initial route selections used in this study. Data sets with greater accuracy and detail can be attained and used to further the route selection process through further study if required in the future.

In order to provide a corridor from which to determine the horizontal and vertical alignments of the proposed track, it was necessary to use a large corridor width. The data set accuracy did not allow for a great deal of interpolation of data between contours. This resulted in a corridor with a width between 50 and 250 km depending on the location along the route and the accuracy of the data set. The entire route is over 2,400 km in length. The route alignment from the Alaskan border to Fort Nelson, British Columbia is based on the 2005 and 2007 Al-Can studies and contains multiple alignment options. The route alignment from Fort Nelson to Peace River, Alberta, and east to Fort McMurray has been digitized using hillshade data, contour data, and existing highway information. This information should be considered very preliminary and is anticipated to change as more accurate data is made available and after further analysis. The elevations of the route selection corridor range from 650 meters to over 4,000 meters. The color hillshade mapping used is only able to break down each area based on 500 metre vertical intervals.

The alignment route is far from established at this early stage. The initial data should be a good starting point to determine a more specific alignment. This data allowed for the progression of the route selection and furthered the Track & Civil portion of the study.

1.2 Route Selection, Track & Civil (Horizontal, Vertical, & Modelling)

As mentioned in the previous section, the starting point for this project was a route selection process based on available public data sets and information contained in previous studies along parts of the proposed corridor. The route selection process was an iterative review of the information that was available including:

- Color coded topography mapping
- 10 m Contours
- Aerial photos
- Existing roads maps
- Provincial and National Park Boundaries.

The general procedure for selecting the route was as follows:

- Break down the corridor into five manageable segments, each segment was approximately 500 km in length. This breakdown by segment allowed for a more manageable data set as well as for multiple teams to work on the route selection process
- Review the color coded topography of the entire segment to determine the best terrain for the alignment corridor to follow

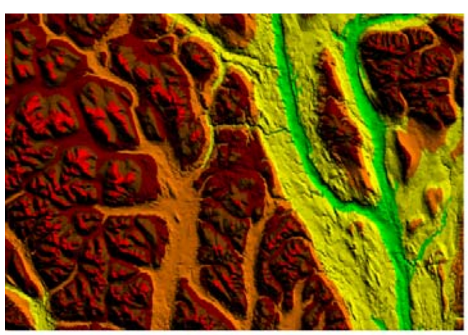


Figure 1 Colour Coded Topography

- Review the Al-Can Rail Link Alignment, where applicable, to be used as a base case to take advantage of and account for previous investigations conducted within the proposed corridor
- Compare the base case to the color coded topography

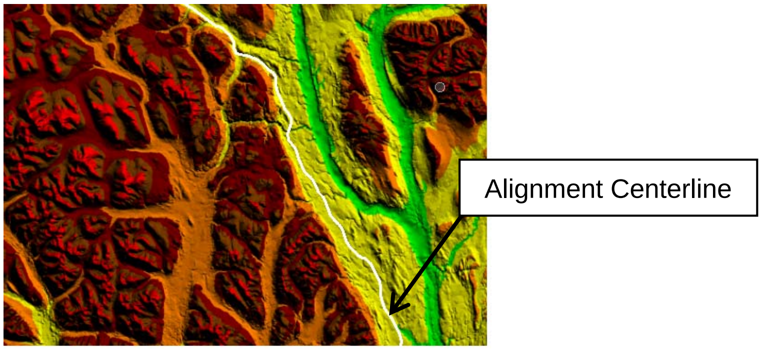


Figure 2 Colour Coded Topography with Route Selection

- Develop alignments and profiles based on the design criteria, topography, avoidance of national/provincial parks, river crossings, and road crossings
- Refine alignments and profiles based on inputs and constraints from other disciplines including:
  - Geomorphology: cut and fill slopes, depth of bedrock, and depth of top soil; and
  - Hydrology/Structures; difficult river crossings, attempt to minimize bridge height and length.

1.2.1 Segmentation of the Route

The Alberta to Alaska Railway alignment is, in general, similar to the previous alignments developed in the Al-Can studies for Segments 4 and 5 and includes variations in Segment 3. The alignment for Segments 1 and 2 were developed, without the use of the Al-Can alignment and were based solely on topographic constraints which included terrain, water bodies, identified environmentally sensitive areas, operational impacts, civil and earthwork impacts, constructability impacts, highways and communities.

The starting point of the route was determined by identifying an optimal location of the loadout yard based on a generalized footprint. The loadout yard was located approximately 50 km (31 miles) north of Fort McMurray to be in close proximity to the points of development. This general site area then became the starting point for Revision 2 of Segment 1.

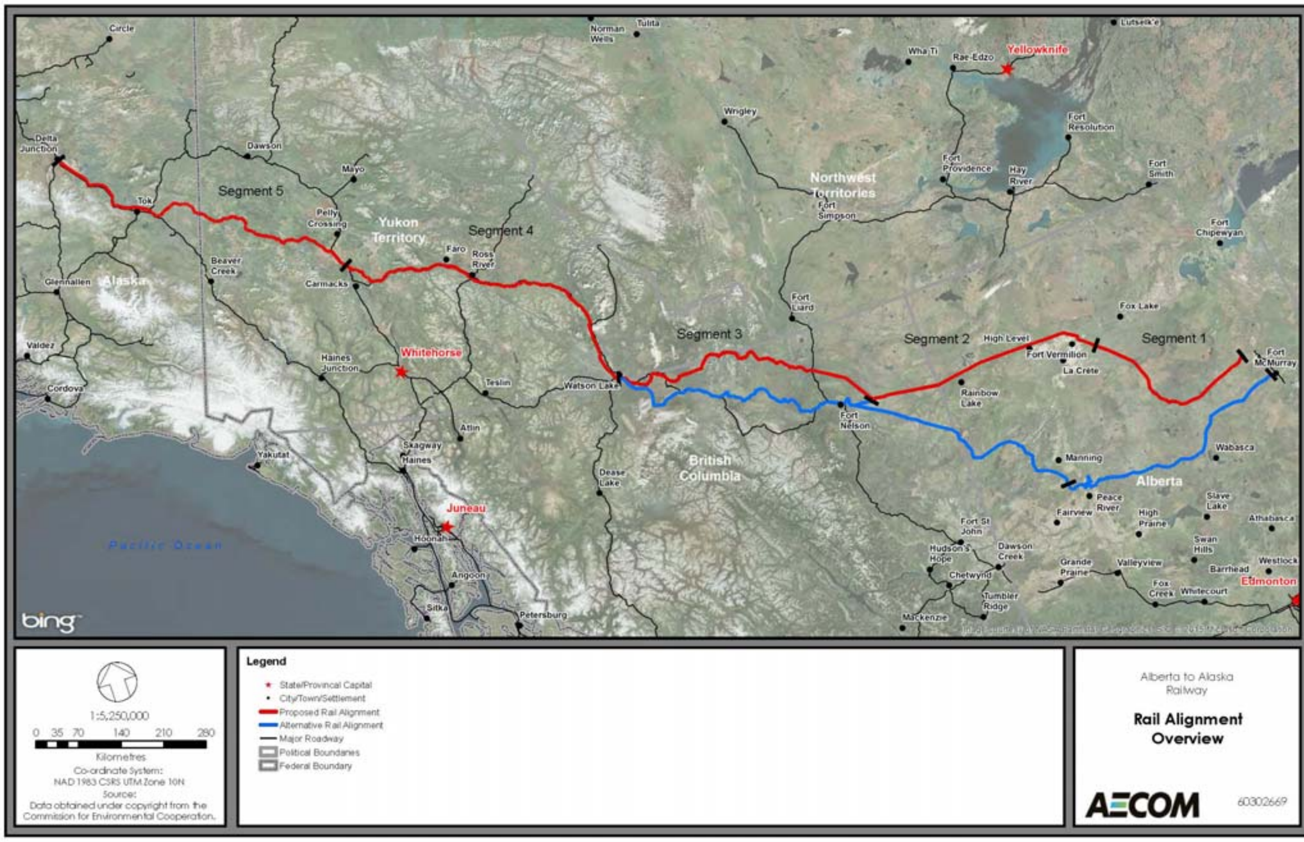


Figure 3 Rail Alignment Overview

1.2.1.1 Segment 1

Revision 1 - Fort McMurray, AB to Peace River, AB: PK 0 – PK 475 (Mile 0 – Mile 295)

Revision 1 of the Segment 1 alignment was 475 km (295 miles) long, starting in Fort McMurray, Alberta and ending in Peace River, Alberta. The profile started at an elevation of 365 m and gradually climbed to an elevation of 680 m at the 354 km (220 mile) mark. After reaching this high point, there was a descent to cross the Peace River 418 km (260 mile) mark at an elevation of approximately 380 m. The most direct route to cross the Peace River was not viable due to design criteria constraints, in particular the maximum allowable grades for the alignment. This constraint resulted in a 40 km (25 mile) northward diversion, impacting a number of farms. From the Peace River, the profile then climbed again for the remaining 56 km (35 miles), finishing at an elevation of 630 m.

Revision 2 – Fort McMurray, AB to Fort Vermillion, AB: PK 0 – PK 318 (Mile 0 – Mile 198)

Given the constraints in crossing the Peace River in Revision 1, it was deemed necessary to investigate an alternative alignment that would minimize the diversion length. Fort Vermillion, Alberta, approximately 225 km (140 miles) northeast of the initial Peace River crossing, was determined to be the optimal crossing location due to topography. This, in conjunction with a new starting point for the alignment, based on the optimum loadout yard location, resulted in the final alignment being 318 km (198 miles) long, starting at an elevation of 360 m. The profile climbs to a highpoint 630 m at the 161 km (100 mile) mark. The profile then descends gradually to 260 m at the end point. It is important to note that there were numerous constraints which impeded using the most direct route(s) from Fort McMurray to Fort Vermillion which included:

- Birch Mountains Wildland Park: There is a valley through this park, which would have allowed for the most direct route to Fort Vermillion. However, being a Provincial Park, it was considered necessary to place the railway corridor outside of this location.
- Topography of the Birch Mountains: These Mountains, surrounding the Provincial Park, were too high to cross given the maximum allowable grades.

Even with these constraints, this alignment proved to be more favourable than Revision 1. Revision 2 avoids the 40 km (25 mile) diversion as well as the sharp decent and ascent which were required to cross the Peace River in Revision 1. This alignment is also 156 km (97 miles) shorter than the Revision 1 alignment, with comparable earthwork quantities per kilometer.

1.2.1.2 Segment 2

Revision 1 – Peace River, AB to Fort Nelson, BC: PK 475 – PK 930 (Mile 295 – Mile 578)

This segment started approximately 15 km (9 miles) west of the difficult Peace River crossing noted in the Segment 1 – Revision 1 alignment. The first 257 km (160 miles) of this alignment were winding, to follow the topography, to avoid Chinchaga Wildland and Milligan Hills Provincial Parks and to reduce earthwork quantities. The next 177 km (110 miles) headed in a northwestern direction and were very straight as the topography is relatively flat and allows for a direct route. The final 21 km (13 miles) turned north to cross the river at Fort Nelson, British Columbia at the narrowest point.

The profile climbed from an elevation of 630 m to 870 m at the 700 km (435 mile) mark, and then descended to 460 m at the 797 km (495 mile) mark. The remainder of the profile was relatively flat towards the river crossing at Fort Nelson where there is a decent to 315 m. Similar to the Peace River crossing, this crossing, which is approximately 0.8 km (0.5 miles) wide, is not ideal and resulted in a 3.5 km (2.2 mile) long bridge with the application of the maximum allowable grades.

Revision 2 – Fort Vermillion, AB to Fort Nelson, BC: PK 318 – PK 711 (Mile 198 – Mile 442)

A second alignment for Segment 2 was considered in order to avoid the difficult river crossings at both Peace River and Fort Nelson. Revision 2 starts at Fort Vermillion and ends approximately 40 km (25 miles) northeast of Fort Nelson. This alignment is relatively straight, with only fourteen curves, and a gently rolling profile. The elevation ranges from 250 m to 470 m. This alignment eliminates the difficult river crossing at Fort Nelson and earthworks per kilometer are comparable to Revision 1.

1.2.1.3 Segment 3

Revision 1 – Fort Nelson, BC to Watson Lake, YK: PK 93 – PK 1,413 km (Mile 578 – Mile 878)

This segment followed the Al-Can alignment closely from Fort Nelson, British Columbia to Watson Lake, Yukon. The majority of the route that was determined is within a 1.6 km (1 mile) offset of the Al-Can alignment, with the greatest offset being less than 3.2 km (2 miles). Segment 3 traverses the most difficult terrain of the entire corridor which consists of undulating and mountainous terrain. Subsequently, the alignment was winding and the majority of the alignment followed the Liard River. The profile reached two peaks, one at the 1,091 km (678 mile) mark of 580 m and another at the 1,333 km (828 mile) mark of 720 m. There were two low points, 310 m at the start of the segment and 405 m at the 1,131 km (703 mile) mark. This alignment was not developed further because it passes through the Liard River Provincial Park and a number of tunnels and bridges would have been required for this alignment.

Revision 2 – Fort Nelson, BC to Watson Lake, YK: PK 711 – PK 1,244 km (Mile 442 – Mile 773)

It was determined that an alternative alignment would be developed to avoid the Liard Provincial Park. Due to the terrain in Segment 3, amending the Revision 1 alignment with a short detour around the Provincial Park was determined to be not feasible. The best option was to shift the entire alignment approximately 64 km (40 miles) north to follow a completely different valley. This shift resulted in an alignment that is relatively straight and flat for the first 145 km (90 miles), with only nine curves. The terrain for this portion of the alignment is rolling with a minimum elevation of 375 m and a maximum of 460 m. From the 856 km (532 mile) mark to the 1,154 km (717 mile) mark the alignment passes through some of the most challenging terrain in the corridor. This portion of the alignment is winding to avoid excess earthworks leading to a high percentage of curves. The profile climbs to a peak of 920 m at the 1,033 km (642 mile) mark where a 7 km (4.4 mile) tunnel is proposed. The descent starts at the tunnel portal and continues to the 1,154 km (717 mile) mark, ending at an elevation of approximately 580 m. Within this section of Segment 3 there are eight proposed bridges with heights over 30 m. Particularly challenging features of the Segment 3 alignment were the Liard River crossing at km 880 (mile 547) and two ridges at km 820 (mile 572) and km 952 (mile 592). The final 89 km (55 miles) of the alignment wind along a major river, but are relatively flat. The alignment ends at an elevation of 650 m.

1.2.1.4 Segment 4

Watson Lake, YK to Carmacks, YK: PK 1,244 – PK 1,843 (Mile 773 – Mile 1,145)

The Al-Can alignment analyzed two routes from Watson Lake to the Alaska border. The first was from Watson Lake, British Columbia to Carmacks, Yukon, along the Ladue River to the Alaska border. The second was from Watson Lake, to Whitehorse, Yukon, through Beaver Creek, and ending at the Alaska border. Review of these two route options based on topography and design constraints determined that the Carmacks route was the better of the two options due to flatter topography. A sub-alternative in the Al-Can corridor from Carmacks to Beaver Creek was also discounted due to its difficult terrain and the requirement for a 14 km (8.4 mile) long tunnel.

Segment 4 closely follows the Al-Can alignment from Watson Lake to Carmacks. In the first 56 km (35 miles), the largest offset from Al-Can alignment is 6 km (4 miles), and in the last 48 km (30 miles) the largest offset is 16 km (10 miles). From the 1,300 km (808 mile) to the 1,794 km (1,115 mile) mark the greatest offset is less than 3 km (2 miles). Alignment alternatives in Segment 4 were very limited due to the mountainous terrain and it was confirmed, through the option analysis, that the Al-Can alignment generally followed the most reasonable path from Watson Lake to Carmacks. The alignment closely follows Highway # 4 from the 1,244 km (773 mile) to the 1,791 km (1,113 mile) mark, and generally follows a number of different rivers. The profile climbs from 650 m to a single peak of 1,010 m at the 1,502 km (933 mile) mark. The remainder of the alignment descends to an elevation of 550 m at the end of the Segment. The Segment 4 alignment crosses the highway nine times over its 599 km (372 mile) length mainly due to the mountainous terrain. During the next engineering phase of the project the number of crossings could be minimized, with the introduction of tunnels, which will be evaluated based on a cost-benefit analysis. Also, within Segment 4 there are seven proposed bridges with heights greater than 30 m.

1.2.1.5 Segment 5

Carmacks, YK to Delta Junction, AK: PK 1,843 – PK 2,440 (Mile 1,145 – Mile 1,516)

The Al-Can alignment proposed two routes from Carmacks to the Alaska border which are discussed in the Segment 4 section above. The preferred route for Segment 5 was determined to be following the Ladue River. The Segment 5 alignment is within a 2.5 km (1.5 mile) offset of the Al-Can alignment. From the Alaskan border to Delta Junction, the alignment was developed based on topography, and other previously mentioned constraints and inputs. This alignment closely parallels multiple major rivers for its entirety which include the Yukon River from km 1,843 (mile

1,145) to km 2,068 (mile 1,285), the White River from km 2,068 (mile 1,285) to km 2,108 (mile 1,310), the Ladue River from km 2,108 (mile 1,310) to km 2,229 (mile 1,385), and the Tok River and Robertson River to the end of the alignment. The Segment 5 profile is generally rolling, with one major peak at the 2,229 km (1,385 mile) mark, having an elevation of 645 m, where a 3 km (2 mile) long tunnel is proposed. Following this peak, the profile is rolling to a final low point at the end of the alignment with an elevation of 350 m. The greatest challenges in this segment included its steep grades near the major peak, as well as steep existing ground slopes along rivers. In many cases the alignment is cut into the side of a steep slope, above a river bank.

Details of important alignment data is summarized in the following section.

1.2.2 Design Criteria

The design criteria are based on AREMA and common North American standards for freight railways. The criterion has been selected to apply to high-standard, heavy-haul railway operations in the northwestern region of North America. The principal design criteria are as follows:

Table 2 Track Design Criteria

Design Criteria	
General	
Loading	315,000 lbs. per car.
Operating Speeds	Maximum train speeds: Mainline Loaded: 80 km/h (50 mph) Mainline Empty: 100 km/h (60 mph) Sidings: 65 km/h (40 mph) Trains within uncontrolled yards: 25 km/h (15 mph)
Train Consist	Total Consist Length: 3,530 m (11,575 ft.) Locomotives: 6 Tank Cars: 192
Track Structure	Gauge: 1,435 mm (56 ½ in.) (Standard Gauge) Rail: 136-lb. Head Hardened Track Structure: Ballast: 300 mm (12 in.) Subballast: 300 mm (12 in.) Shoulder Width: 300 mm (12 in.) Concrete Ties: Length: 2.74 m (9.0 ft.) Depth: 250 mm (10 in.) Spacing: 600 mm (24 in.)
Mainline Horizontal Alignment	
Circular Curve Radius	The radius of the curve is initially defined by the speed/ superelevation. Minimum Radius: Preferred Minimum Radius: 2°-0’. Absolute Minimum Radius: 3°-30’. Maximum Radius: Preferred Maximum Radius: 1°-0’. Absolute Maximum Radius: 0°-30’.
Tangents	Reverse Curves (Opposite Hand): Preferred Minimum Tangent: 30 m (100 ft.)

Design Criteria	
	Absolute Minimum Tangent: 25 m (85 ft.) Like Curves (Same Hand): Preferred Minimum Tangent: 30 m (100 ft.) Minimum tangent lengths are measured between spirals.
Points of Intersection (PI)	A PI without a curve is only permitted when the results of the superelevation, curve, and spiral formulae are all less than the minimums (e.g. if the only calculated solution requires a curve with a radius of >0°30’ with a resulting curve length of <30 m (100 ft.), then a simple PI is acceptable).
Mainline Vertical Alignment	
Gradients	For Westward (Loaded) trips: Maximum compensated grades shall be 1.0%. For Eastward (Unloaded) trips: Preferred maximum compensated grade shall be 1.0% and absolute maximum compensated grade shall be 1.5%. Compensated grades shall be used for all designs.
Curve Compensation	Compensated grade is the sum of the actual gradient plus the computed compensation value. G_c = [G – (0.04 * Dc)] (AREMA) Where: G_c = Compensated Gradient (%) G = Actual Gradient (%) D_c = Degree of Curve in decimals of degree
Vertical Curves	Vertical curves are not used for this study.
Combined Horizontal & Vertical Curves	The compensated grade criteria are sufficient.

1.2.3 Alignment

The Alberta to Alaska Railway alignment is approximately 2,440 km (1,516 miles) long and consists of 5 segments, connecting Fort McMurray to Delta Junction. The 5 Segments consist of the following:

Table 3 Segment Information

Segment	From	To	Length
Segment 1	Fort McMurray, AB	Peace River, AB/ (*Fort Vermillion, AB)	318 km (198 miles)
Segment 2	Peace River, AB/ (*Fort Vermillion, AB)	Fort Nelson, BC	393 km (244 miles)
Segment 3	Fort Nelson, BC	Watson Lake, YT	533 km (331 miles)
Segment 4	Watson Lake, YT	Carmacks, YT	599 km (372 miles)
Segment 5	Carmacks, YT	Delta Junction, AK	597 km (371 miles)

* *The match point between Segments 1 & 2 was originally Peace River, AB. It was later revised to Fort Vermillion, AB.*

Important alignment data is summarized in the following table:

Table 4 Segment Alignment Data

Segment	Percent of Alignment in Curves	Percent of Alignment in Absolute Minimum Radius Curve	Average Grade*	Minimum Elevation (m)	Maximum Elevation (m)
Segment 1	4%	0%	+0.29%; -0.34%	260.0	629.1
Segment 2	2%	0%	+0.28%; -0.22%	251.3	472.1
Segment 3	24%	0%	+0.47%; -0.46%	288.5	920.4
Segment 4	27%	0%	+0.47%; -0.53%	560.9	1,010.2
Segment 5	21%	3%	+0.38%; -0.40%	349.3	644.8

*+ Ascending in the Westward direction and – Ascending in the Eastward direction

1.2.4 Track Materials

The track structure will consist of concrete ties spaced at 600 mm (24 inches) intervals. Concrete ties have been tried and tested for heavy haul applications and provide a much stronger track structure with minimal maintenance requirements compared to other tie types. The 600 mm (24 inches) tie spacing allows for a factor of safety that can cope with unknown environmental factors. This will be particularly advantageous given the remote locations which are difficult to access along the rail corridor. 136 lb. head hardened rail as per American Railway Engineering and Maintenance-of-Way Association (AREMA) Chapter 4 which is continuously welded together will be used. The rail will be fastened to the concrete ties using a typical elastic fastener and insulator system. A rail pad will be introduced between the base of the rail and the concrete tie rail seat to avoid excess vibration and abrasion. The ballast depth will be 300 mm (12 inches) and consist of granite because of large temperature variations and high freeze/thaw cycles. A typical track cross section can be seen in Figure 4 below. Based on the track structure described above; future tonnage increases can be incorporated without further adjustment to the track structure.

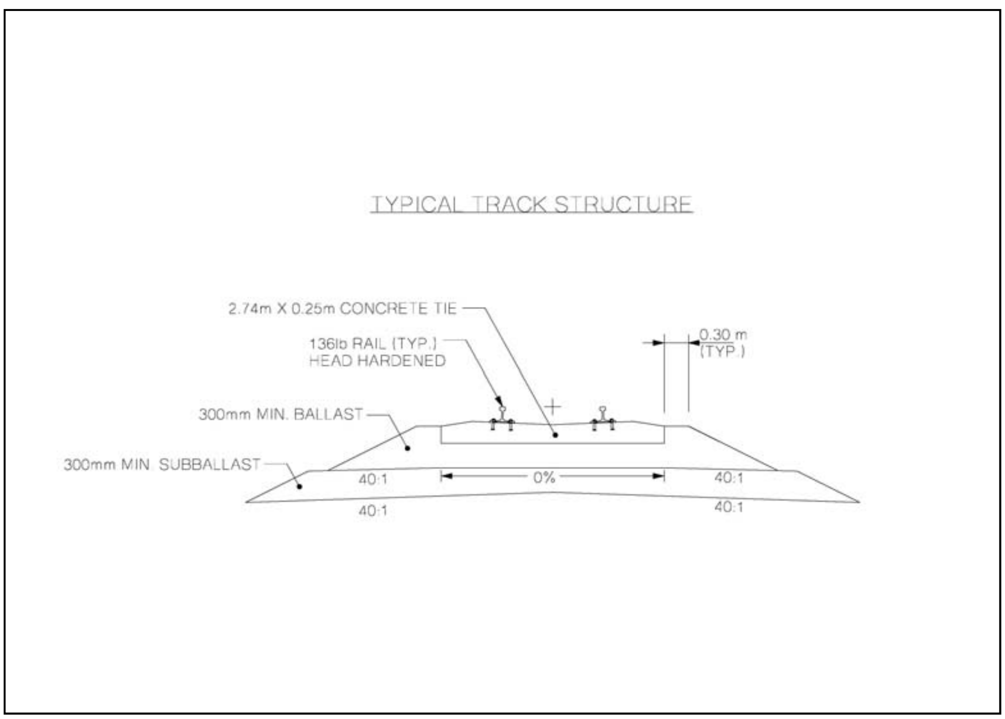


Figure 4 Typical Track Structure

All turnouts will be of AREMA geometry standards, and be consistent in size where possible. The use of standard AREMA type turnouts will minimize lead time, reduce capital and operating expenditures due to the availability of these materials within the market, and contain a relatively straightforward geometry compared to other types of turnouts. Maintaining a consistent turnout size will provide the opportunity to capitalize on the economies of scale during procurement and increase the availability of spare parts during maintenance. The size and quantity of turnouts associated with each siding and yard, relative to the 1.0 mbpd and the 1.5 mbpd production situations, are summarized in the table below:

Table 5 Number and Type of Turnouts

Location	1.0 M Barrels per Day			1.5 M Barrels per Day		
	No. 10 Turnouts	No. 12 Turnouts	No. 20 Turnouts	No. 10 Turnouts	No. 12 Turnouts	No. 20 Turnouts
Loadout Yard	60	0	6	83	0	6
Unload Yard	45	0	6	68	0	6
Sidings	0	2	2	0	2	2
Crew Changes	0	1	7	0	1	7
Fueling	0	5	5	0	5	5

1.2.5 Sidings

There are three types of sidings, a typical siding used for train meets, a siding to facilitate train meets and crew change points and a siding to facilitate train meets, crew change points and fueling.

A typical siding consists of a clear length of 3,600 m (11,811 feet) spaced at approximately 60 km (37 mile) intervals. Siding locations were chosen to facilitate train operations, minimize major earthworks impacts and avoid heavy grades (greater than 0.5% on average), tunnels, bridges and curves where turnouts are located.

Back Tracks are included within sidings to have a clear length of 150 m (492 ft.), be double ended (connect to the siding at both ends) and placed at one end of siding to avoid blocking access in the event a train is in the siding. Sidings that facilitate crew change points have a grade of 0% to allow the train to be left unattended safely. The number and the earthwork quantities associated with the three types of sidings relative to the 1.0 mbpd and the 1.5 mbpd production situations are summarized in the table below:

Table 6 Detailed Siding Information

	1.0 M Barrels per Day			1.5 M Barrels per Day		
	Sidings	Crew Changes	Fueling	Sidings	Crew Changes	Fueling
Number	33	4	1	54	4	1
Earth Cut (m^3)	2,861,000	684,000	329,000	4,737,000	684,000	329,000
Rock Cut (m^3)	1,558,000	415,000	414,000	2,647,000	415,000	414,000
Fill (m^3)	4,248,000	1,054,000	704,000	7,101,000	1,054,000	704,000
Top Soil (m^3)	1,107,000	265,000	119,000	1,816,000	265,000	119,000
Total Earthworks (m^3)	9,774,000	2,418,000	1,566,000	16,301,000	2,418,000	1,566,000

During the next engineering phase of the project, simulation results will validate and dictate the theoretical siding locations. Detail analysis will be done to ensure the sidings maintain the grade requirements and the turnouts are not located within horizontal and vertical curves.

1.2.6 Future Work

During the next engineering phase of the project, alignment optimization will be conducted to:

- Balance cut and fill quantities, whilst reducing major cut, fills and material haul distances
- Minimize intrusion into sensitive areas
- Reduce the number of sharp horizontal curves
- Reduce the degree of curvature
- Reduce the gradient for the loaded trains.

Analysis will also be done during the next phase to account for:

- A more accurate base plan
- At grade crossings (including snowmobiles)
- Grade separations (for busy crossings and wildlife)
- Access roads to the railway corridor
- Drainage along the right-of-way including a delineation of drainage network and basin catchments, flood plain and high water analysis and the identification of minor culverts
- Vertical curve introduction
- Simulation results to account for operational requirements
- Verification of siding locations.

1.2.7 Earth Works

To account for the various geotechnical conditions along the railway corridor three different templates were used based on inputs from the Geomorphology team. They consisted of:

Table 7 Cut and Fill Parameters

Fill Height (m)	Fill Slope	Cut Depth (m)	Cut Slope	Rock Slope
0 - 4	2:1	0 - 4	2:1	0.5:1
4 – 8	2.5:1	4 – 8	2.5:1	0.5:1
8 +	3:1	8 +	3:1	0.5:1

The bedrock and top soil depth was determined from geomorphology input with the lengths simplified for workability. Typical earthwork cross-sections can be found in Figures 5 and 6 below.

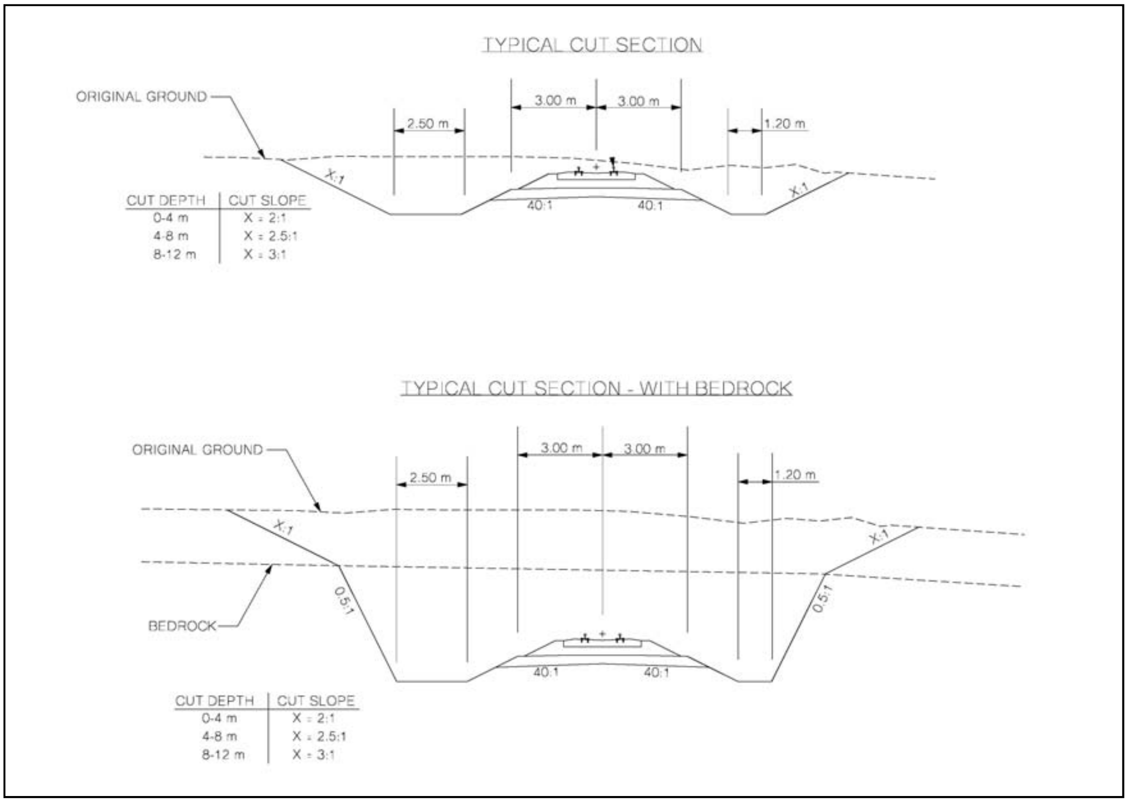


Figure 5 Typical Fill Sections

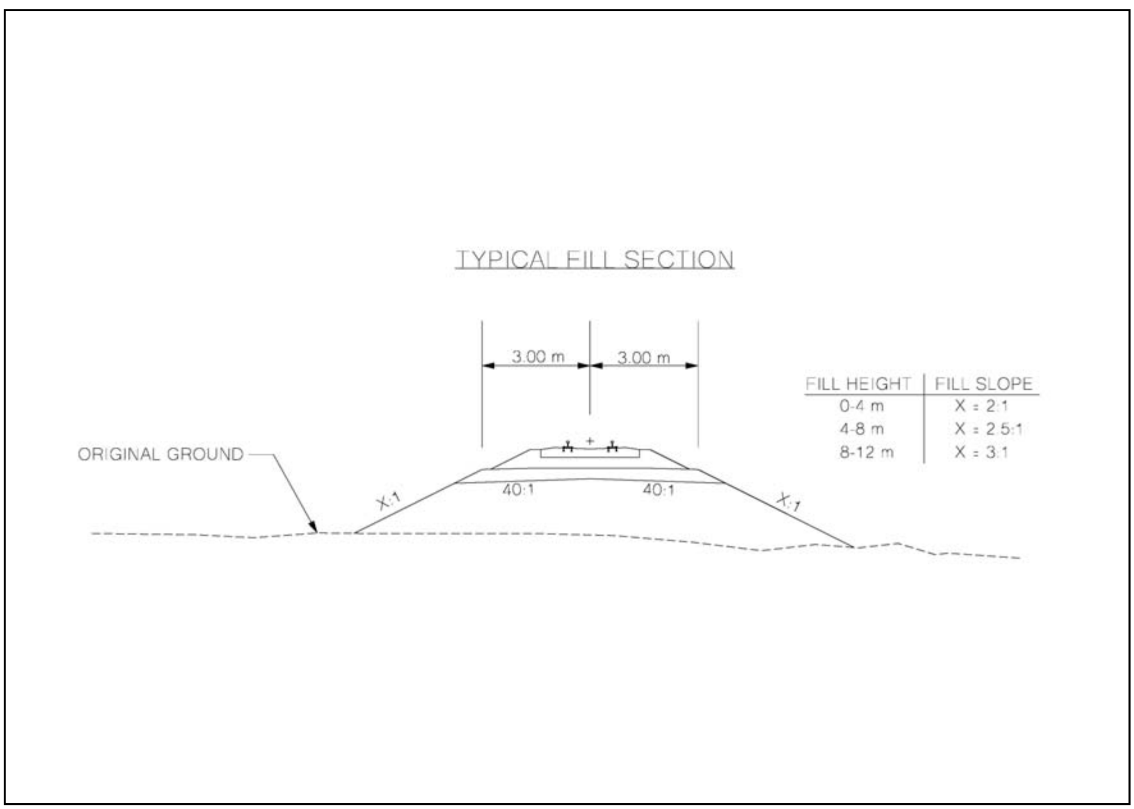


Figure 6 Typical Cut Sections

Ditches were included on either side of the railway formation, one being 1.2 m and the other being 2.5 m. During the next engineering phase, ditch widths and depths will be verified based on hydraulic capacity, constructability and maintenance requirements. 2.5 m wide ditches are in place to capture water flowing towards the railway trackbed, facilitate ease of construction and cleanout by front end loader and provide access along the corridor in remote areas. Where the 2.5 m wide ditches have to change sides due to drainage flow paths, this change will occur at level crossings where a vehicle can also change sides.

Table 8 Cut and Fill Quantities

Segment	Earth Cut (m^3)	Rock Cut (m^3)	Fill (m^3)	Top Soil (m^3)	Total Earthworks (m^3)
Segment 1	4,900,000	0	4,800,000	2,600,000	12,300,000
Segment 2	3,600,000	0	3,100,000	3,000,000	9,700,000
Segment 3	36,300,000	45,700,000	77,700,000	13,100,000	172,800,000
Segment 4	48,000,000	26,300,000	69,400,000	24,200,000	167,900,000
Segment 5	76,100,000	25,600,000	100,900,000	22,500,000	225,100,000
Total	168,900,000	97,600,000	255,900,000	65,400,000	587,800,000

1.3 Tunnels

Tunnel locations were determined based on deep cuts which are not practically feasible via conventional earthwork excavating methods. From a preliminary cost-benefit analysis, it was determined that cuts with an average depth greater than 40 m should be considered to be tunnels.

In the Alberta to Alaska Railway alignment, there are a total of two rail tunnels determined at this level of study; one located in Segment 3 with a length of 7 km (4 miles), starting at the 1,022 km (635 mile) mark and the other located in Segment 5 with a length of 3 km (2 miles), starting at the 2,229 km (1,385 mile) mark. Both of the tunnels have a constant grade of -1.0% (downward in the westward direction). It was determined that these tunnels cannot be eliminated with further refinement of the alignment within the current corridor based on the available inputs and constraints.

In addition to the two major tunnels identified, there were an additional five locations where shorter tunnels could be implemented in place of average cut depths of 40 m or more. It was determined, however, that further alignment refinement could possibly eliminate these deep cuts, therefore they were not considered as tunnels at this time.

It is assumed that tunnels shorter than 1 km will not have Mechanical and Electrical (M&E) works. Tunnels greater than 1 km are assumed to include M&E due to the necessity to maintain ventilation and various suppression and detection systems.

During the next engineering phase of the project, tunnel optimization and an evaluation of construction types will conclude the tunnelling construction methodology best suited to the project conditions and needs. With this, the design criteria relevant to the M&E works for the tunnel will also be developed and will include suitability of the tunnels, ventilation system, fire suppression systems, fire detection systems, fire separation, electrical systems, controls system, communications system, lighting systems, HVAC systems, security and drainage sump systems.

1.3.1 Mined Tunnel Geometry

The shape and size of the mined tunnel cross-sections would be determined based on further future study based on the potential geology, length of tunnel and locomotive characteristics.

1.3.1.1 Tunnel Grades

A minimum and maximum grade where a separate drainage system is not available will be determined by the rail grade but should not be less than 0.5%. The design should also incorporate provisions for drainage water sumps at the alignment low points.

1.3.1.2 Ground Cover

The minimum ground cover for a tunnel should be one times the tunnel width and height. The minimum ground cover at the mined tunnel portals should be 0.5 times tunnel width or height, whichever is larger.

1.3.1.3 Loads and Load Factors

The conditions influencing the load on the tunnel support system that should be considered in the design are as follows:

- In-situ stresses (horizontal and vertical as applicable)
- Presence and quality of groundwater
- Joint geometry, spacing of discontinuities
- Rock strength and deformation properties
- Construction means and methods
- Excavation shape and size
- Reinforcement by rock bolts, shotcrete and grouting.

1.3.1.4 Applied Loads

The rock support system should be designed to withstand all in-situ loading conditions. In general, it should be designed to resist the following loadings:

- Seismic loading
- Appropriate ground and variable hydrostatic loading
- Long and short term ground yield or squeeze
- Long and short term loads induced by the construction sequencing and means and methods
- Loads arising from adjacent tunnelling, or excavation.

1.3.1.5 Water-Tightness

The criteria for water-tightness of the completed excavation will be established in future stages of study as follows:

- No visible ingress of water or damp patches above springline.
- Damp patches only below springline. A damp patch is discoloration of part of a surface, moist to touch. There will be no visible movement of a film of water across a surface.

1.3.2 Standard Tunnel Lining Systems

Based on ground conditions along the alignment it assumed that mined tunnels in rock will be designed and constructed using rock support system without a cast-in-place concrete lining if possible. In this case, shotcrete is used both as initial and final lining in applied. In soft ground tunnels, a final lining would be required.

1.3.2.1 Rock Support System

Mined tunnels and caverns will be supported by rock bolts, rock dowels, mine strapping devices, shotcrete or combination thereof. Rock reinforcement elements will be designed in accordance with AREMA requirements and will be capable of achieving the required Design Life as permanent works.

1.3.2.2 Ground Water Ingress during Construction

Water ingress during construction should be limited to levels that ensure the specified limits on drawdown in the groundwater table are not exceeded.

1.3.2.3 Rock Load

The mined excavation is a compound structure consisting of the rock formation surrounding the excavation and the ground support system. The ground pressures will vary at different construction stages and with the type of ground support system used.

Rock loads will be established by empirical, ground stress-strain, or force equilibrium methods. Calculating the rock loads for the tunnel design will be based on geotechnical design parameters as presented in the geotechnical reports.

1.3.2.4 Seismic Analysis

It is anticipated that any mined tunnels will be located entirely within rock strata. The tunnel alignment shall avoid crossing active faults if possible. If crossing active faults cannot be avoided, the tunnel will be designed to tolerate the expected fault displacements with no or minor damage.

1.3.2.5 Ground Movements

Ground movement predictions will be based on stress-strain analyses.

1.3.2.6 Stress-Strain Methods

The stress-strain analysis should consider three-dimensional rock stresses, loadings, and displacements around the underground opening. The analysis should account for factors that influence the loads on the excavation. The analysis shall include relevant safety factors and the allowable ground movements.

Table 9 presents the amount of stress release as a function of the distance between the face and the support.

Table 9 Stress Release Criteria for Two-Dimensional Stress-Strain Analysis

Distance of excavation face from support as ratio of excavation equivalent dimension (D_e)	Stress release as % of in situ stress
(0 – 0.1) D_e	30%
(0.1 – 0.3) D_e	50%
(0.3 – 1) D_e	70%
(1 – 3) D_e	80%
(>3) D_e	100%

1.3.2.7 Design of Rock Support System

The methods for rock support design will be empirical, utilizing stress-strain relationships, and force equilibrium.

The following information would need to be derived from extensive geotechnical investigation during design:

- Rock mass quality using the Norwegian Geotechnical Institute's Q rating system
- Presence and condition of faults, shear zones, discontinuities, joint patterns, hydrothermal alteration, etc.
- Review of any measurements of jointing patterns (strike, dip, frequency, joint sets)
- Estimate of Deformation parameters of intact rock and rock mass including joints
- Estimate of Joint condition in quantitative terms of friction angle, dilation angle, and cohesion
- Estimate of rock mass strength in terms of the Hoek-Brown failure criteria with the associated parameters.

1.3.2.8 Empirical Methods

The empirical design of ground support measures should be considered using the Norwegian Geotechnical Institute's Rock Tunneling Quality Index, or NGI-Q (Barton, Lien, and Lunde 1974; Barton 2002; and Grimstad, et al. 2003).

The value for the rock quality index Q is determined from:

$Q = (RQD/J_n)(J_r/J_a)(J_w/SRF)$ (1)

Where:

RQD= Rock quality designation

J_n= rating for the number of joint sets (9 for 3 sets, 4 for 2 sets, etc.) in the same domain

J_r= rating for the roughness of the least favorable joint set or filled discontinuities

J_a= rating for the degree of alteration or clay filling of the least favorable joint set or filled discontinuity

J_w= rating for the water inflow and pressure effects, which may cause outwash of discontinuity infillings

SRF= rating for faulting, for strength/stress ratios in hard massive rocks, for squeezing or for swelling

The above ratings should be determined from tables developed by Barton (2002).

A representative Q-index should be determined for the design based on the available Geotechnical Data Reports. The representative Q-index should be obtained by considering a zone that extends 1/4 times excavation width or height, whichever is larger, above and below the tunnel crown.

To relate the Q-value to the ground support requirements, the excavation equivalent dimension (D_e) should be defined as the larger of excavation width and height, divided by the excavation support ratio (ESR). The ESR value for the mined tunnels should be taken as 1.0. In calculating the excavation equivalent dimension, the increase in the theoretical width or height due to excavation overbreak.

The shotcrete thickness and bolt spacing should be determined from the Q-Chart shown in the figure below.

1.3.2.9 Rock Bolt Design

The bolt diameter should be determined based on the bolt tributary area and the bolt allowable load. Bolts will be designed such that the axial stress in the bolt does not exceed 60% of the tensile yield stress of the bolt. The permanent roof support pressure will be calculated from the following expressions:

Proof	=	2 Q-1/3 / J_r	with 3 or more joint sets	(2a)
Proof	=	2 Jn1/2Q-1/3 / ($3J_r$)	with less than 3 joint sets	(2b)

In view of the more favorable position of excavation walls as opposed to roof, it should be allowed to consider a hypothetically increased wall quality Q_{wall}, in accordance with the following equations:

Q_{wall}	=	5Q	(Q>10)	
Q_{wall}	=	2.5Q	(0.1<Q<10)	(3)
Q_{wall}	=	Q	(Q<0.1)	

The length of rock bolt or dowel should be determined from the following expression:

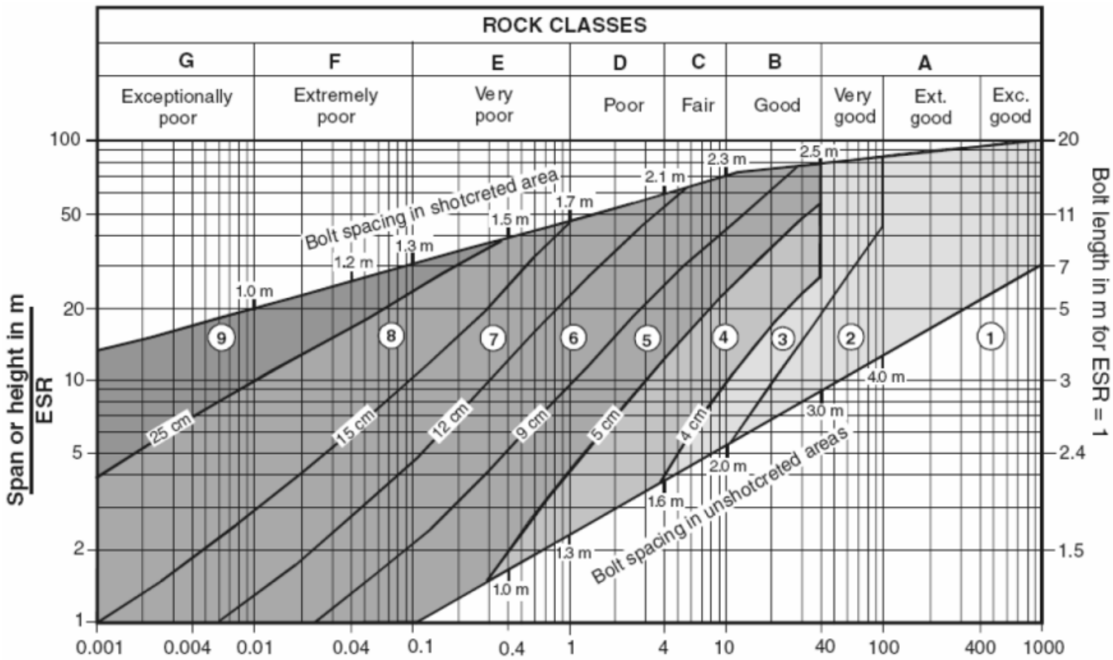
L	=	2 + 0.15 D_e	(4)
L	=	bolt or dowel length in metres	
D_e	=	excavation equivalent dimension in metres	

Rock bolt lengths shall not be less than:

- The length required for support of rock blocks as identified in the force equilibrium method;
- The length required to anchor outside of the failure zone around the excavation as determined by a stress-strain analysis with a Hoek-Brown or Mohr-Coulomb failure criterion; and the length required to stabilize wedges in the crown, as determined by stress-strain discontinuum analysis.

Where:

L= 2 + 0.15 D_e (4)
L= bolt or dowel length in meters
D_e= excavation equivalent dimension in meters



Note: Shaded areas 1 to 9 show different recommended reinforcement categories

Figure 7 Q-Chart for Selecting Initial Excavation Support

1.3.2.10 Excavation Sequence

The design of tunnel sizes and shapes will be governed by excavation rate, opening stability, and ground settlement and vibration concerns. Different excavation sequences will be considered and their advantages and disadvantages will be studied through numerical modelling. The optimum excavation sequence should be determined based on the results of these analyses.

1.4 Hydrology

1.4.1 Methodology

The design of structures, facilities, rolling stock and other aspects of the new proposed railway are governed by the hydro climatic conditions encountered along its route. This section gives a summary and appreciation for climatic conditions from different regions along the alignment, in terms of monthly precipitation averages, mean monthly temperatures and extreme wind velocities. This is key information required to create a project tailored design criteria for many aspects of the new railway alignment. For hydraulic structures, more information was gathered from various national databases for a first attempt at dimensioning all bridges and culverts required along the entire proposed route. However for the level of detail of this study, only rivers were simply modeled individually to obtain their dimensional requirements. This was carried out first for large rivers, by a frequency analysis of the flood discharge in relation to reference sites and by regional analysis to develop and extrapolate data from the reference sites in order to obtain the discharges at the crossing locations of the railway. The required bridge openings were then determined by estimation of the high water level from a relationship between the maximum flow, flow velocity and cross sectional area of the stream. Small rivers were approached differently, using the rational method to model the 100 year return period flow, this from the analysis of a number of Intensity-Density-Frequency curves from sources along the proposed alignment. For smaller streams, expertise in similar regions and projects allowed a comprehensive estimate to be created on the density of watercourses in relation to different physical characteristics of zones encountered along the proposed alignment. Further to this, a distribution of different culvert dimensions was created from the analysis and compilation of data from other similar projects and applied to the overall number of estimated streams along the route.

This first attempt at quantifying hydraulic structures accounted for 70 bridges (required due to hydraulic conditions, not topographical constraints), 77 large culverts and 4,087 small culverts. The analysis of selected meteorological stations identified the east section of the alignment to receive the most precipitations during the summer months and to have warmer conditions all year long. However this report was based on the analysis of a fraction of available data along the proposed alignment, in terms of hydrometric and meteorological databases. Additionally the available topographic base maps were not of the accuracy to determine all minor stream crossings and accurately delineate the sections of larger streams. At a further stage of the project, more data will be analysed to refine the modelling methods used in this study and better Digital Terrain Models (DTM's) will be created to identify stream crossings and catchment areas with greater accuracy.

1.4.2 Introduction for the Study Area

This report gives a summary of the hydrological analysis carried in the pre-feasibility study for the construction of the Alaska to Alberta Railway connection. It contains the following:

- General introduction
- Summary description of the project
- Description of the studied zone
- Objectives of the hydro climatological analysis
- Climate and precipitation data
- Statistical analysis of stream flows
- Design basis for hydraulic structures.

The summary hydro climatological details introduced in this report follow or excess the level of precision as set for a scoping study. The modelling efforts were prioritised for named streams along the alignment, as the structures required for these crossings have a larger individual impact on the Capex of the project. However, assumptions as mentioned in this section were taken in order to quantify all water way crossings along the railway alignment.

1.4.3 Studied Zone and its Physical Attributes

The projected route for the railway will connect Fort McMurray to Delta Junction. The zone of interest lies within the latitudes 57^{0} 08' 55'' N and 64^{0} 02' 10'' N, and between the longitudes 111^{0} 55' 17'' W and 145^{0} 41' 02'' W.

From a physical point of view, the railway crosses many different zones, from the plains of Alberta, through the mountainous Rockies and finishing its route in the valley plateaus of Alaska. Due to the length of the preliminary selected route, further geomorphological studies as well as vegetation sampling should be carried out as these factors play an important role in the retention capabilities of catchment basins and therefore on the flow rates of streams. However when the soil is frozen the infiltration rate is at a minimum affecting the flow rates.

The current climatic conditions for the preliminary railway route are characterised by long winters and short summers, with significant adverse winter conditions in some locations. This has a significant impact on constructability considerations, maintenance plans, railway crew equipment, rolling stock and facilities. These conditions should be studied further in relation to their impact on various aspects of constructing and operating the proposed railway. Other hazardous conditions, mainly found in the mountainous regions of the project are avalanches and rock fall, which should be identified along the route and impact mitigation plans proposed.

Under the present climatic conditions, snow, ice and permafrost play important roles in the hydrology of the area spanned by the project. These aspects will require further study in order to fully assess their impact on hydraulic structures. Nordic conditions can have major design implications and require special design consideration, dependant on the severity of the conditions observed.

The remoteness of sections of the selected route should also affect design considerations for hydraulic structures, due to the increased impact to the operations in the case of a washout or other disruption. The distance of structures from available construction materials and crews can considerably affect downtimes and operation costs. These factors should be evaluated in more depth and considered in the design phase of the project.

1.4.4 Objectives

The hydrological and climatological analyses covered in this report are part of a scoping study. They aim at characterising the different zones found across the project, from a physical, climatological and hydrological point of view, in order to respond to a number of needs related to the design, construction, development and other related environmental and socio-economic studies. Specifically, it gives the required information for the predeterminations of key variables required for the dimensioning of hydraulic structures along the railway alignment (bridges, culverts, embankments, etc.).

In relation to the climatology, it gives the required information in relation to temperature extremes (maximum and minimum), precipitations (monthly rainfall and snowfall) and wind velocities. Secondary information not directly related to the dimensioning but more for the actual design of the structures crossing the identified streams along the railway alignment.

1.4.5 Climate and Precipitations

For this study, the information from many meteorological stations was gathered. This was carried out in order to obtain information about, precipitation, daily temperatures, wind velocities and climatic extremes. The data found and shown in this section is not directly related to the modelling nor dimensioning of hydraulic structures, however is crucial for countless other design features for the proposed railway. The information shown in this section is a summary of complete data sheets found in Appendix 1. Further to this, the data sheets contain various other information not summarised in this section, such as visibility hours, wind chill and humidex which should be further analysed as these have direct impacts on constructability, as far as scheduling, selection of construction materials and equipment.

The following stations were selected for their proximity to the selected railway route:

Table 10 Station Locations

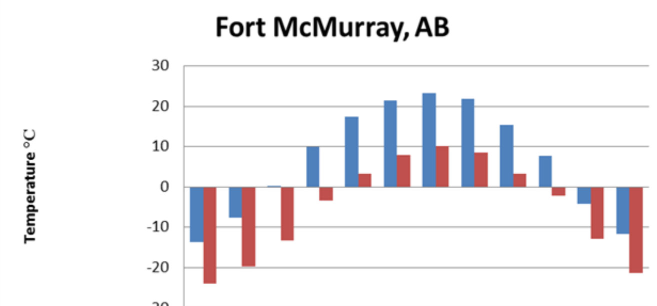
Station	Province/Territory/State	Latitude	Longitude	Elevation (m)
Fort McMurray	Alberta	56°39'00.000" N	111°13'00.000" W	369.1
Peace River	Alberta	56°13'37.000" N	117°26'50.000" W	570.9
High Level	Alberta	58°37'17.000" N	117°09'53.000" W	338.0
Fort Nelson	British Columbia	58°50'11.000" N	122°35'50.000" W	381.9
Watson Lake	Yukon	60°06'59.400" N	128°49'20.400" W	687.4
Dawson	Yukon	64°02'35.000" N	139°07'40.000" W	370.3
Gulkana	Alaska	62°14'25.000" N	145°25'22.000" W	1,480.0
Tok	Alaska	63°30'00.000" N	143°00'00.000" W	503
Big Delta	Alaska	64°08'50.000" N	145°48'06.000" W	386.5
Delta Junction	Alaska	64°00'00.000" N	145°40'00.000" W	335

The quality and quantity of meteorological data found throughout the project zone is far greater than shown in this report. However at this stage of the project the data gathered will be sufficient at demonstrating the various different climatic conditions encountered throughout the railway alignment. At a further stage of the project, more data will be gathered and analysed for a better delineation of various climatic regions along the proposed railway alignment.

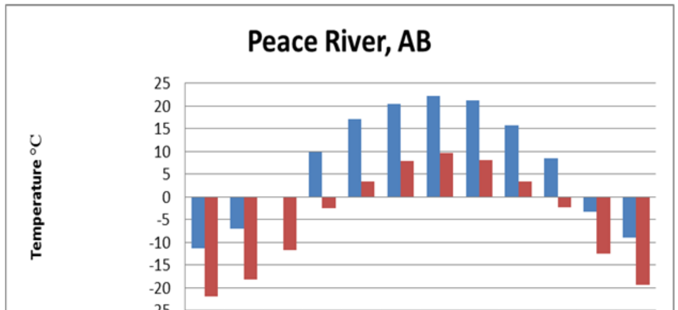
1.4.6 Temperature

This section of the report shows the average daily minimum and maximum temperature graphs for each meteorological station listed in the previous section. These graphs give a good overview of the difference in temperatures between the different regions of the project. For the complete set of data from the meteorological stations, refer to Appendix B-1.

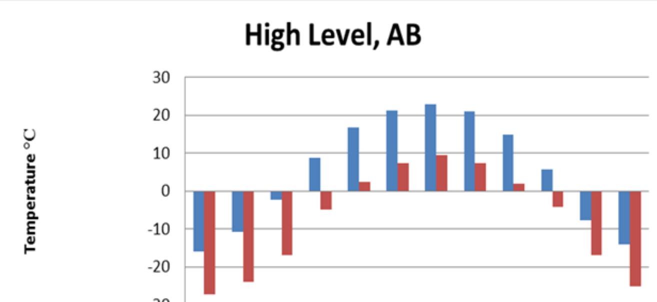
Figure 8 Temperature Graphs



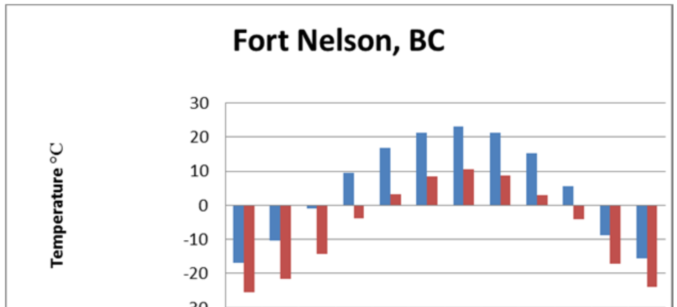
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-13.6	-7.6	0.3	10	17.4	21.4	23.2	21.9	15.4	7.8	-4.2	-11.6
Daily Minimum (°C)	-24	-19.8	-13.2	-3.3	3.3	7.9	10.2	8.6	3.3	-2.2	-12.8	-21.4



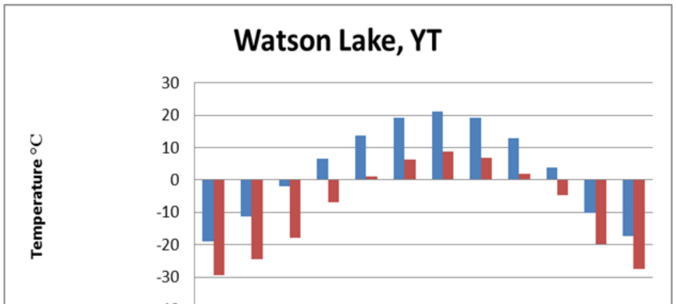
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-11.4	-7.1	0	9.9	17	20.4	22.2	21.2	15.7	8.4	-3.4	-9
Daily Minimum (°C)	-21.9	-18.3	-11.8	-2.6	3.4	7.9	9.7	8.1	3.3	-2.4	-12.5	-19.3



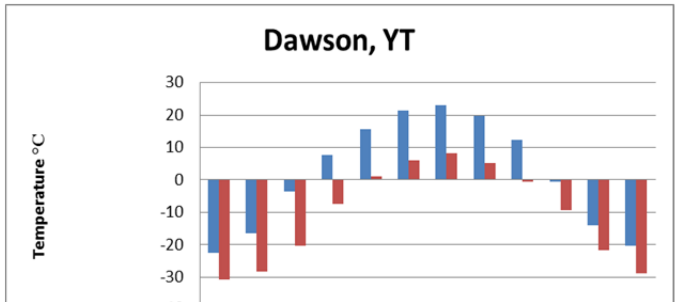
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-16	-10.8	-2.4	8.8	16.8	21.1	22.8	21	14.9	5.7	-7.7	-14.2
Daily Minimum (°C)	-27.2	-24	-16.8	-4.9	2.3	7.4	9.5	7.4	1.8	-4.3	-16.9	-25.2



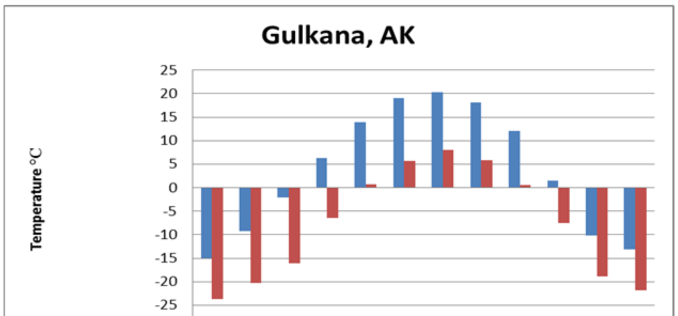
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-17	-10	-1	9.5	17	21	23	21	15	5.5	-9	-16
Daily Minimum (°C)	-26	-22	-14	-4	3.2	8.4	11	8.6	2.9	-4	-17	-24



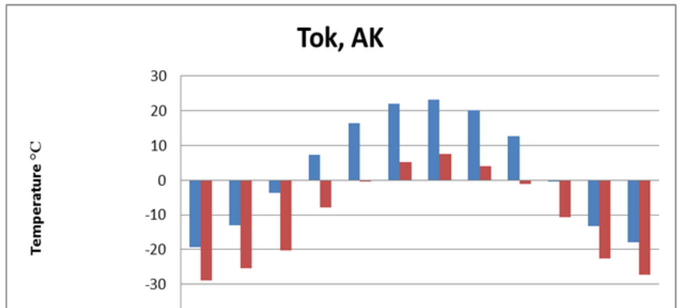
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-18.9	-11.3	-2	6.7	13.7	19.1	21.2	19.1	12.9	3.8	-10.3	-17.4
Daily Minimum (°C)	-29.4	-24.5	-17.9	-6.8	1	6.3	8.9	6.8	2	-4.8	-19.7	-27.4



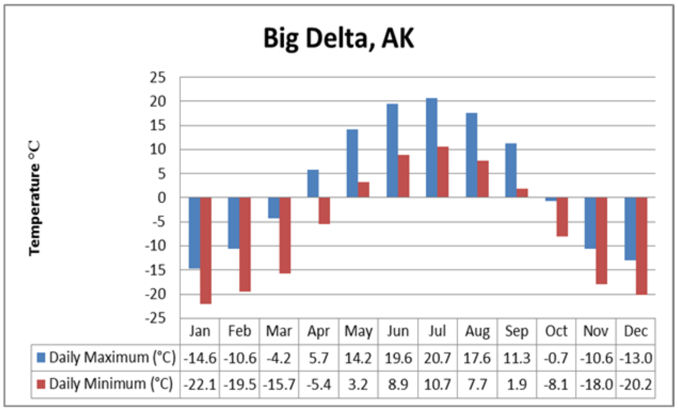
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-22.5	-16.5	-3.6	7.6	15.5	21.3	23.1	19.8	12.2	-0.5	-13.9	-20.4
Daily Minimum (°C)	-30.9	-28.2	-20.2	-7.5	1	6	8.1	5.1	-0.6	-9.4	-21.8	-28.9

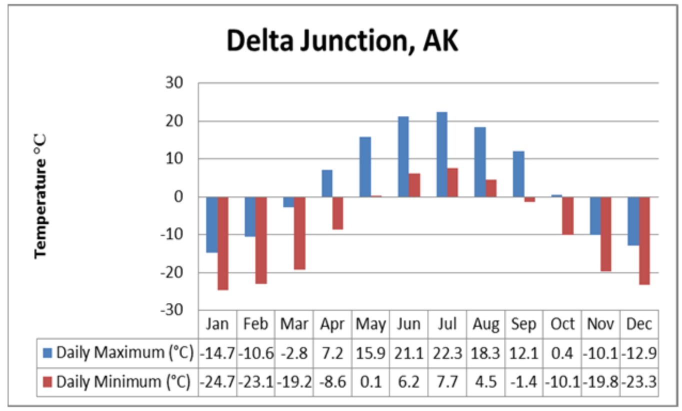


	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-15.0	-9.2	-2.1	6.3	13.9	19.1	20.3	18.2	12.0	1.5	-10.2	-13.2
Daily Minimum (°C)	-23.7	-20.3	-16.1	-6.4	0.8	5.7	8.1	5.8	0.6	-7.6	-18.9	-21.9



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Maximum (°C)	-19.3	-13.0	-3.8	7.3	16.3	21.9	23.1	20.1	12.6	-0.4	-13.2	-17.9
Daily Minimum (°C)	-28.9	-25.5	-20.3	-7.9	-0.5	5.1	7.6	4.1	-1.1	-10.6	-22.7	-27.3



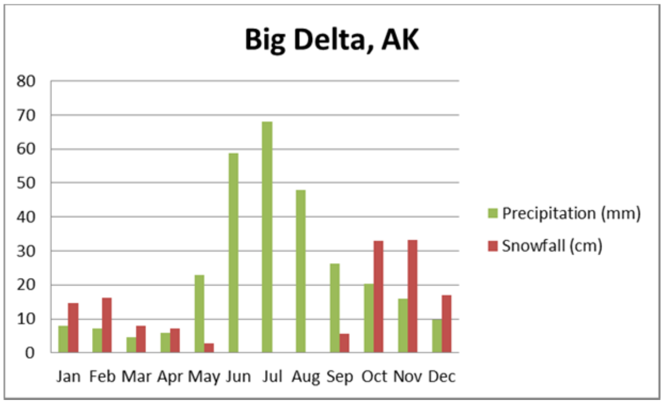


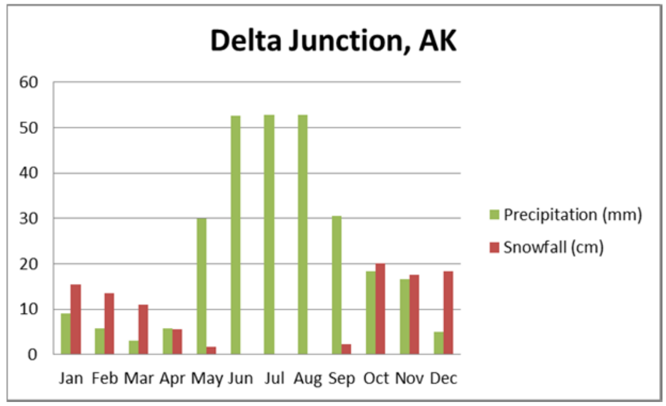
The extreme maximum recorded temperature throughout the selected meteorological stations was of 36.7 °C at Peace River and Fort Nelson, and the extreme minimum recorded temperature was of -58.9 °C at Watson Lake.

1.4.7 Precipitation

In this section, the average monthly precipitation recorded at each selected meteorological station is shown in terms of rainfall and snowfall. However for stations in the State of Alaska, information about rainfall was not found, and therefore interpretation of the monthly precipitation is required, for complete set of data refer to Appendix B-1 and B-2. These graphs give a good representation of conditions found along the proposed railway alignment, however further study of “Intensity-Duration-Frequency” curves should be carried out in a later stage of the project for the modelling of smaller streams along the alignment. The statistical analysis for the hydrological flow is presented in Appendix B-3.

Figure 9 Precipitation Graphs





As can be observed in the preceding graphs, conditions would appear to be dryer on the western part of the railway alignment, receiving fewer precipitations during the summer months than the eastern side, the observable summer month's precipitation peaking at its highest in British Columbia. With further studies and analyses of precipitation data from meteorological stations along the proposed railway alignment, areas of similar hydrological characteristics will be determined allowing models to be created for the dimensioning of culverts within these regions.

1.4.8 Wind Velocity

The wind velocity is an important factor to take in consideration, not as much for the hydraulic modelling of rivers and streams, but more for the design of structures crossing these points, as well as other railway facilities required for the project. The table below gives the maximum recorded wind velocities, for the mean, maximum hourly and maximum gust, for complete set of data for all meteorological stations selected in this section, refer to Appendix B-1.

Table 11 Wind Velocities

Parameter	Velocity (km/h)	Wind Direction	Meteorological Station
Mean Velocity	14.4	W	Peace River, AB
Maximum Hourly	80	W	Watson Lake, YT
Maximum Gust	150	SW	Watson Lake, YT

The values observed above are the ultimate values found through review of the meteorological stations listed above, excluding all stations in Alaska for which this data was not available. However there are large variations in the velocities in relation to the location of the station and this is also valid for locations along the proposed railway alignment. There are places which will be more prone to windy conditions and higher velocity winds. This will be studied in greater depth in a further stage of the project to insure the values selected in the design criteria for railway structures corresponds to the findings of these stations.

1.4.9 River Modelling

At this level of the design for this project, only major and minor rivers have been modeled. Other streams along the alignment have been estimated and sized differently, as detailed in section 1.4.9.3. This section of the report explains the approach and methodology applied for the dimensioning of major hydraulic structures by describing all assumptions and data used in the process.

1.4.9.1 Selection of Return Period

The dimensioning of a hydraulic bridge or very large culvert requires the determination of many parameters in relation to the site in question, such as the characteristics of the stream and the desired level of protection for a given structure.

A structure which is over dimensioned will insure security in terms of the conveyance flow, however the capital costs of investment for its construction will be high. On the other hand an under dimensioned structure will have a lower construction cost, however the interruptions of service and damages to this structure will be frequent and could even be catastrophic. For these reasons a balance between costs, damages and risks has to be carried out for the selection of an appropriate average recurrence interval. In this case a return period of 100 years has been selected as this follows the standard design criteria throughout the regions of the project route for such structures.

Note that time restrictions prevented frequency analysis of every possible hydrometric station. The maximum recorded discharges were used, and 100 year modelled discharges were validated at selected hydrometric stations.

1.4.9.2 Approach and Methodology for Flow Calculation

All rivers crossing the proposed railway alignment were identified and traced using ArcGIS software (ArcGIS Explorer). Other parameters such as the delineation of the catchment basin, measurement of the longest flow path, mean slope and river width were carried out using Google Earth. These programs offered reasonable accuracy for the level of design for this scoping study. More sophisticated GIS models should however be used in a later stage of the project.

1.4.9.3 Major Rivers

In order to cover the needs for the pre-feasibility study of the Alberta to Alaska Railway study, many bibliographic and national hydrometric databases were consulted. The available modelling methods were reviewed and the data obtained was gathered to come up with two following steps:

1. Frequency analysis of the flood discharge in relation to reference sites
2. Regional analysis to develop and extrapolate data from the reference sites in order to obtain the discharges at the crossing location of the railway in relation to the modelled stream.

Through the statistical analysis of data obtained from hydrometric stations, two zones were identified with similar parameters. These zones are as follows:

Table 12 Zone 1: Delineated Within Segments 1 and 2

Station	Data Available for the Following Years	Latitude	Longitude	Reference Catchment Area (km^2)
Chinchaga River	1969-2011	58°35’49.000’’	118°20’02.000’’	10369.6
Wabasca River	1970-2011	57°52’28.000’’	115°23’20.000’’	35800
Hay River	1963-2012	60°44’34.000’’	115°51’34.000’’	51700

Refer to Appendix 3 for the statistical analysis graph and table.

Table 13 Zone 2: Delineated Within Segments 3, 4 and 5

Station	Data Available for the Following Years	Latitude	Longitude	Reference Catchment Area (km^2)
Snake River	1963-1967	65°14’45.000’’	133°24’10.000’’	2770
Frances River	1962-2011	60°28’26.000’’	129°07’08.000’’	12800
Yukon River	1902-2011	60°42’50.000’’	135°02’35.000’’	19600
Yukon River	1951-1995	62°05’45.000’’	136°16’18.000’’	81800

Refer to Appendix 3 for the statistical analysis graph and table.

Limited data points were used in this study due to time constraints; however the interpretation of this data provides a fair account of the relationship found in each identified zone along the alignment. At a further stage of the project more data will be plotted in order to validate the finding of this scoping study.

In relation to these zones, two formulas were created to determine the mean discharge from the following regional relationships:

- Zone 1
  - $Q_m = 0.002 * A + 9.5675$
- Zone 2
  - Qm = 4x10-8 * A2 + 0.0122 * A + 14.614

Q_m and A being respectively the mean discharge (m^3/s) and catchment area (km^2)

Therefore for a discharge of frequency T, its flow can be determined by the following relationship:

$Q_T = F_p * Q_m$

With: $F_p = 1x10^{-9} * A^2 + 0.00007 * A + 2.2336$

The peak factor (F_p) being the relationship between the average rate and 100 year return period rate. This value is obtained by the regional method applied throughout the railway alignment.

In addition to the use of this modelling method and preceding equations to obtain the required river flows. Three hydrometric stations were found located on three different rivers crossing the proposed railway alignment. These were located at Peace River, Liard River and Yukon River, which gave all required information for the 100 year return period. The Gringorten empirical plotting point relationship was used to estimate the 100 year return period peak discharge at these sites.

The maximum flows in relation to the identified major river crossings along the proposed alignment are shown in Appendix B-4.

1.4.9.4 Minor Rivers

For minor rivers a different approach was used by analysing some Intensity-Duration-Frequency (IDF) curves at locations along the alignment. Through this analyses three regions of similar precipitation characteristics were determines as follows:

- Region 1: Segments 1 & 2
  - IDF curve from Peace River
- Region 2: Segments 3 & 4
  - IDF curve from Tetsa River
- Region 3: Segment 5
  - IDF curve from Watson Lake.

For the IDF curves used for these three regions, refer to Appendix 2.

The flow rate for these rivers was calculated using the ten year return period flow method, by calculating the 10 year peak flow rate. The following formula was obtained by a statistical analysis of 630 watersheds with an area inferior to 2,000 km^2.

$Q_{i10} = A^{0.8} (P_{j10} / 80)^2 R$

Where:

Q_{i10} is the peak flow at a return period of 10 years (m^3/s)
A is the catchment area in km^2
P_{j10} is the maximum daily rainfall at a 10 year return period in mm
R is the regional coefficient which is taken to be 1 for this situation (all regions)

The peak 100 year return period flow is then calculated using the following equation:

$Q_{100} = Q_{i10} * F_p$

Where the peak factor is obtained by the IDF curves for the individual regions mentioned above, these are as follows:

Table 14 Peak Factors

Frequency	Fp Region 1	Fp Region 2	Fp Region 3
Q_{100}	1.47	1.5	1.44
Q_{50}	1.37	1.33	1.25
Q_{10}	1	1	1

The calculate of flow rates using the rational method also requires the evaluation of concentration times for each watershed (T_c), which is the longest time it takes for a drop of water in the a particular catchment basin to reach the inlet of the hydraulic structure.

The concentration time can be estimated using several different formulas such as the Ventura, Passini, Giandotti and Kirpich, each with their own level of validity. However for this exercise the Giandotti method was used, and represented below:

$$T_c = 4 \frac{\left(S_{BV} + 1{,}5 \frac{L}{1000} \right)^{1/2}}{0.8\, H_{BV}^{1/2}} \quad \text{Si } 25\text{ km}^2 < S_{BV} < 400\text{ km}^2$$

Where,

T_c: Concentration time in hours
S_{BV}: Catchment area (km^2)
L: The length of the hydraulic path in metres
H_{BV}: The average height of the watershed in metres

For the complete list of identified minor rivers and their flows, refer to Appendix B-5.

1.4.9.5 Approach and Methodology for Water Level Calculation

The high water level (HWL) was estimated using a relation between the flow rate, flow velocity and cross section area of the stream. The following relationship was used:

Q = V * A

Knowing the flow rate from the regional method, a standard velocity for the stream was estimated, which allowed the high water level to be identified and tabulated.

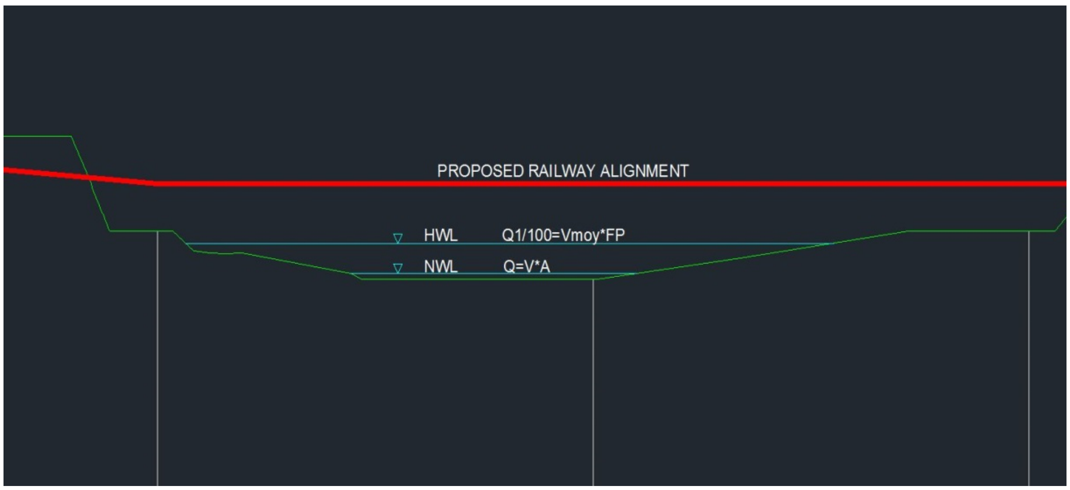


Figure 10 Estimated High and Normal Water Levels

The results of this relationship give a good approximation for the water levels in streams crossed by the railway alignment. However the baseline data used such as the stream sections, flow velocities and flow rates are preliminary. The knowledge of the water level at a return period of 100 years allows the cross verification of the current bridge elevation with the minimum required elevation, including the 1 m minimum freeboard to respect under the bridge and therefore is crucial for the design of hydraulic structures. More accurate models will be developed in a later stage of the project as the accuracy of the baseline data improves.

1.4.10 Design Basis for Culverts

1.4.10.1 Identification

In this phase of the project, minor streams could not be identified along the entire alignment due to the sheer number of streams this represented. For this reason the overall number of minor streams was estimated using the following approach.

1.4.10.2 Zone 1 – Eastern Plains

This zone is constituted of grassland plains with a medium density of streams per kilometer. For this reason and for the purpose of this exercise, one culvert per kilometer was estimated along this strip of the alignment ranging from kilometer point 1000+000 to 2390+000.

Streams in this zone are expected to have low velocities with a high percentage of sediment in the water as these flow along fields and open grasslands. The control of sediment build up at the inlet of the culvert will need to be assessed a later stage of the design. It is to be expected that in some locations the minimum cover heights for selected culvert diameters will be hard to obtain and alternative designs will be required.

1.4.10.3 Zone 2 – Mountainous (Rockies)

This zone is constituted of steep rocky mountain faces with a very high density of streams per kilometer. For this reason and for the purpose of this exercise, two culverts per kilometer were estimated along this strip of the alignment ranging from kilometer point 3000+000 to 4594+037.

Streams in this zone are expected to have high velocities, flowing down the mountain ranges with a low catchment basin concentration time and low soil absorption rate. It is expected that these streams will also be intersecting the railway alignment at high embankment locations due to the presence of high and steep valleys. The culvert designs in this zone will require more rip rap to mitigate erosion of the railway embankment and require a risk analysis to be carried out to assess the risk of blockages due to debris (boulders, tree trunk, etc.) accumulating at the inlet of the culvert. Further to these, the location of these structures will be very remote, and any maintenance or replacement will be more complex and supply of materials much more expensive and hard to obtain.

1.4.10.4 Zone 3 – Western Plateaus

This zone is constituted of plateaus between the Rocky Mountains with a mixture of valleys and plains with a high density of streams per kilometer. For this reason and for the purpose of this exercise, a culvert every 0.75 km has been estimated along this strip of the alignment ranging from kilometer point 5000+000 to 5598+380.

Streams in this zone will have a mix of constraints and design challenges expressed in the earlier two zones. Further design challenges may arise with a comprehensive geomorphologic study.

1.4.10.5 Results

Following the methodology described through the previous sections, the subsequent results were obtained:

Table 15 Estimated Culverts Per Zone

Description	Number of Culverts
Zone 1	710
Zone 2	2208
Zone 3	797
Contingency (10%)	372
Total	4087

A ten percent contingency was added to the estimate in order to account for all small drainage paths, construction needs, etc.

Note that sample counts of culvert crossings were performed to develop average culvert frequency.

1.4.10.6 Dimensioning

In this pre-feasibility study, the number of culverts and their dimensioning has been entirely estimated using similar railway alignment scenarios, as the design effort required in the identification, delineation and design of all hydraulic structures required for this category of streams is particularly labour intensive and requires very accurate GIS terrain models to be created. However this present approach at quantifying hydraulic structures for minor streams gives the best overview of the design challenges ahead for this aspect of the design. The following distribution of culvert dimensions was gathered from other similar scenarios and used in this project.

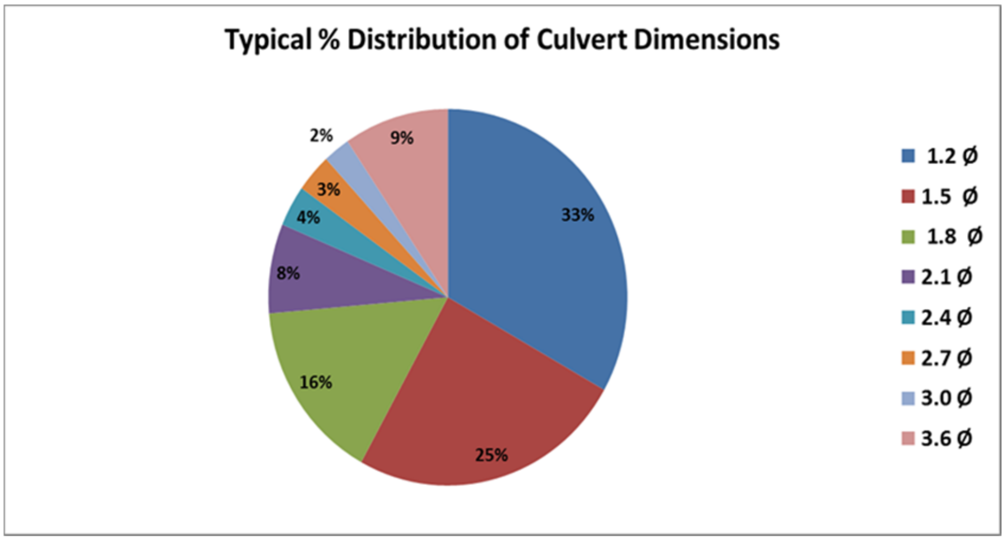


Figure 11 Typical Culvert Distribution

As the project evolves and heads towards the feasibility study, better terrain models will be created allowing a refined identification of minor streams along the alignment and ultimately allow simple modelling methods such as the rational method to be used in the calculation of stream flows, velocities and water levels to increase the accuracy of the dimensioning of hydraulic structures and their required designs in terms of culvert type and erosion protection of the embankment. Additionally as the project evolves, geomorphological mapping in concordance with vegetation identification along the railway alignment will allow the modelling methods presently used to be refined with better information on the soils and the vegetation's ability to retain water and therefore come up with better predictions for the runoff coefficients and concentration times.

1.4.11 Operational Aspects

The maintenance program for culverts during the operation of the proposed railway will generally be particularly complex due to the vast number and spread of these structures along the alignment. The following stage of design should assess inherent risks involved with this type of structure to incorporate certain design features in relation to their physical location. Aspects such as remoteness, ease of access, visibility from a high rail vehicle, sediments in streams, velocity of water for debris transportation and ice formation should all be analysed and structures designed accordingly.

From vast experience with railways in similar regions as the proposed railway alignment for this project; beaver dams upstream and directly within hydraulic structures have always been a high concern for embankment failures. This occurs when a beaver dam breaks upstream, creating a flow surge which exceeds the design capacity of the hydraulic structure or when the dam itself is in the structure, greatly reducing the efficiency of the structure and creating higher hydraulic pressures on the embankment. However there are multiple design features which can be incorporated to reduce this failure risk, solutions which will be determined in a later stage of the project. High risk locations will also be identified along the alignment in order to allow comprehensive maintenance and observation programs to be created as well as additional design features to be incorporated to some hydraulic structures in order to mitigate this occurrence.

1.4.12 Hydrology Conclusions and Recommendations

1.4.12.1 Conclusions

This hydrology study has been carried out in order to obtain the required information for the dimensioning of hydraulic structures along the railway alignment. The principal objective was to come up with a method to estimate flows for large streams and develop a method for estimating the needs in terms of smaller streams. This study was based on a revue of available hydrological and meteorological literature at a national and regional scale.

In conclusion, this study gives a first glance overview of the scale and quantity of hydraulic structure for the entire proposed railway alignment. The information obtained is preliminary however due to vast experience with other projects of this scale and in similar regions, this estimate and glance at the overall requirement in terms of hydraulic structures gives a good base for an initial cost estimate and understanding of the constructability challenges ahead.

1.4.12.2 Recommendations

For the next step of the project, which will be the feasibility study, it is recommended to use the rational method for catchment basins less or equal to 25 km^2. This modelling method should use IDF data found closer to the source, as the flood level and flows for smaller streams are largely affected by rainwater, whereas snow and ice melt will be the dominant flooding cause for larger streams in the region. Increasing the accuracy for smaller streams will require the following data to be obtained or analysed:

- IDF curves in order to identify clear hydrological zone with similar parameters
- Geomorphological and vegetation investigation for soil data in order to process water absorption rates and runoff coefficient
- Accurate GIS terrain models to be developed in order to identify and measure catchment areas for smaller streams, as well as stream slopes and catchment slopes.

For medium streams ranging between 25 km^2 and 100 km^2; the TR55 modelling method should be used as it offers an accurate however efficient way of estimated the flow rates for this category of crossings. However to proceed with this estimating method, more accurate information will be required on the terrain and as for smaller streams, better geomorphological data in order to predict soil absorption rates and runoff coefficients.

For large streams in this context, all crossings with a catchment area larger than 100 km^2; the regional modelling method should be carried out. This has already been applied to large stream in this scoping study, but in the following step of the project, models will be refined and more data will be considered. The following data will be required to extend the level of accuracy of streams models:

- Accurate topographical and bathymetric surveys as well as a geotechnical investigation, in order to get an accurate idea of the correct localisation of the hydraulic structure, the extent and layout of the future bridges.
- Site investigation; essential exercises to visualise and appreciate the selection of the ideal crossing locations for the proposed hydraulic structures. Additionally this allows information to be gathered on high water levels from local research and visual investigation of the stream and surrounding vegetation.

The aim of the scoping study was to give an appreciation of the required bridge openings at identified river crossings, however in the later stage of the design, positioning of piers and abutments in relation to local terrain and stream conditions will be essential. These include aspects such as erosion mitigation measures and prevent river flow constraints. Nevertheless in depth work is required between the bridge designers, environmental team and hydrology team in order to come up with sound designs for each river crossings.

For future studies, river hydraulics should be modeled with HEC-RAS (USACE) or equivalent. More sophisticated rainfall-runoff models may also be considered, as the level of topographic information advances.

Overall hydraulic structures represent a large initial capital investment in the construction of a new railway alignment, however the quality and accuracy of the work must be achieved through following the appropriate design steps in order to minimise operational maintenance costs and operation downtimes.

1.5 Geomorphology

The section of the report will discuss and illustrate the geomorphological aspects of the project based on a review of the existing information available along the proposed route gathered at the start of the study. This was a desktop exercise which utilised the mapping and aerial photography information along with the experience of the team to determine, classify and describe the features of each type of terrain. This information was iteratively fed back and forth to the Route Selection, Track & Civil, Hydrology and Bridges teams to help refine the route selection, the types of bridges and culverts as well as the constructability of the railroad.

1.5.1 Methodology

1.5.1.1 Terrain Classification

Nine different terrain units were used to classify the ground along the selected routes. These terrain units included: organic deposits, fluvial, eolian, colluvium, glaciolacustrine, glaciofluvial, moraine, moraine veneer, stagnant ice moraine, fluted moraine and bedrock.

Each route was analyzed using 1:50,000 NTS mapping, satellite imagery and available geological mapping information. The figures below show the identical route alignment plotted over NTS and surficial geology maps, respectively.

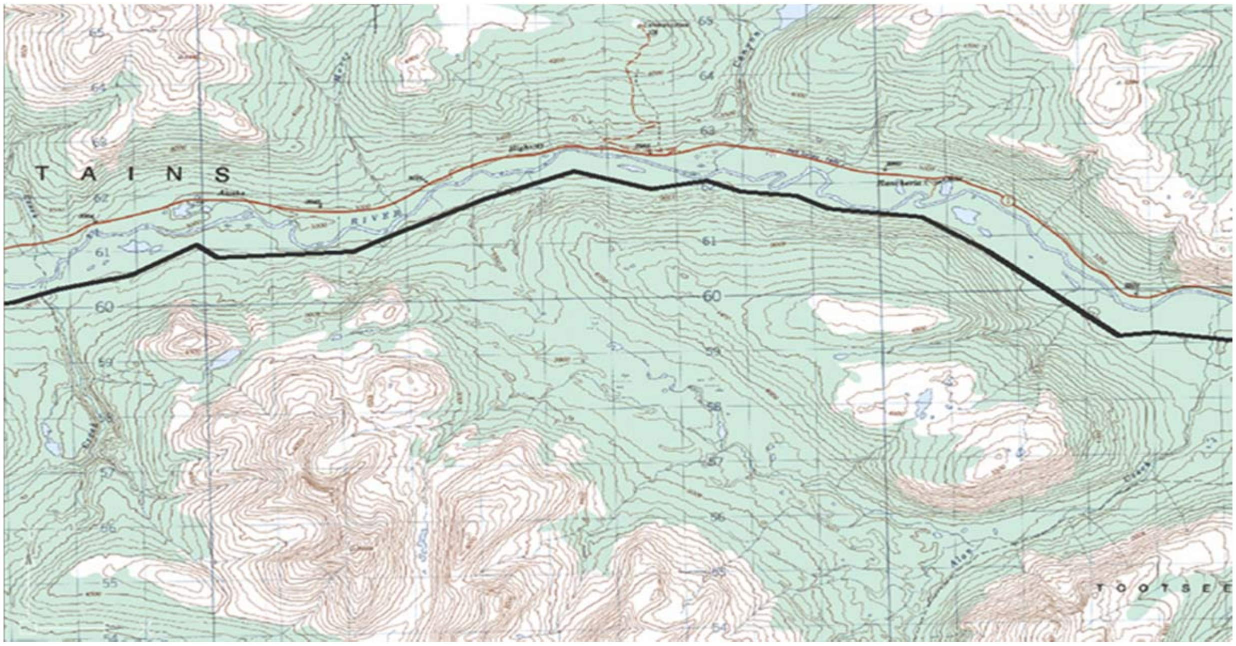


Figure 12 NTS Mapping

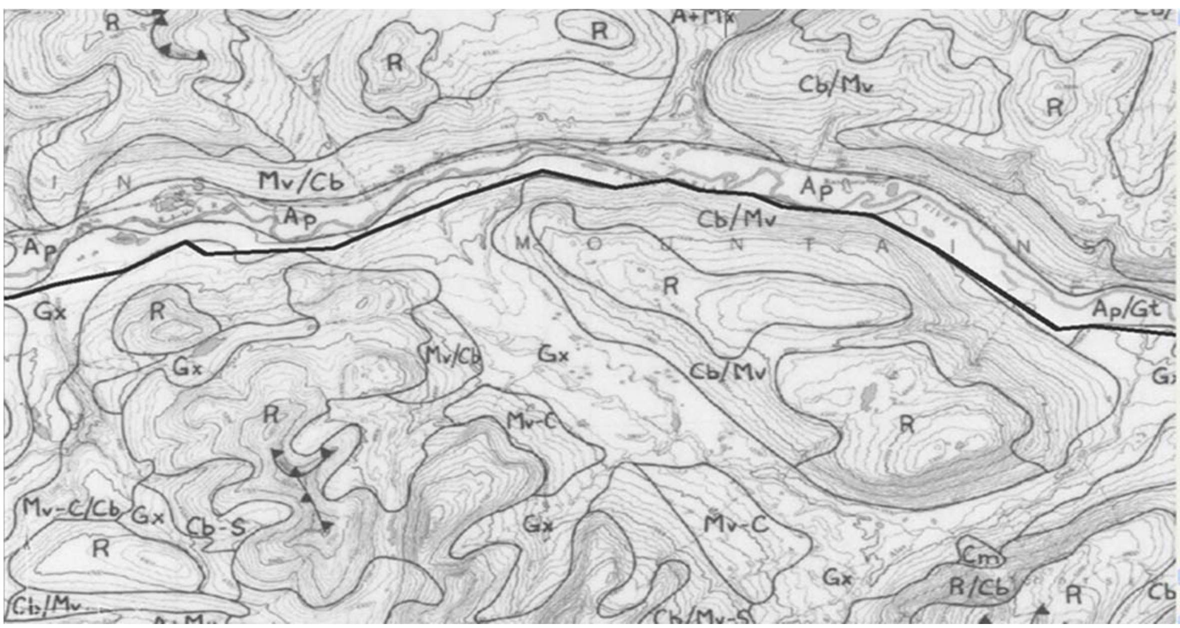


Figure 13 Surficial Geology Mapping Alignment

The terrain types were identified from surficial geology maps obtained from the Alberta Geological Survey, Geological Survey of Canada and the Alaska Geological Survey. When insufficient geological mapping sources were available, satellite imagery was used to estimate the terrain units. The following sections are a summary of each identified terrain unit.

1.5.1.2 Organic Deposits (Holocene)

Organic Deposits consist of peat occurring in wetlands commonly underlain by fine-grained, poorly drained glaciolacustrine or lacustrine deposits, occasionally underlain by glacial moraine (till). This category includes marshes, swamps, bogs and fens.

Figure 14 Organic Deposit

1.5.1.3 Fluvial Deposits (Holocene)

Fluvial deposits are formed when sediment is transported and deposited by streams and rivers. Generally, these deposits consist of gravel and/or sand and/or silt (and rarely clay). Gravel is typically rounded and contains interstitial sand. Fluvial sediment is commonly moderately to well-sorted and displays stratification, although massive, non-sorted fluvial deposits do occur. Fluvial deposits in the large valley bottoms typically have a sandy texture because of the abundance of reworked glaciolacustrine sediment. The term is synonymous with alluvial, however, alluvial deposits are generally referred to when there is a large change in hydrologic flow causing deposition of sediment in fan-like forms. The figures below show typical fluvial and alluvial deposits.

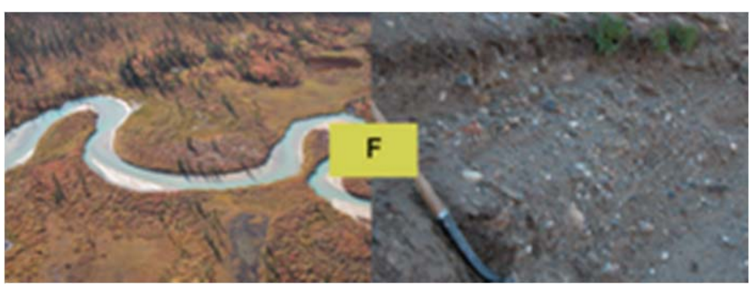


Figure 15 Fluvial Deposit

Figure 16 Alluvial Deposit

1.5.1.4 Eolian Deposits (Holocene)

Eolian deposits form when sediment is transported and deposited by wind action. It generally consists of medium to fine sand and coarse silt that is well-sorted, non-compacted, and may contain internal structures such as cross-bedding or ripple laminae, or may be massive. Individual grains may be rounded and exhibit frosting. Eolian landforms may be active or vegetated and inactive. Figure 17 shows a typical eolian deposit.

Figure 17 Eolian Deposit

1.5.1.5 Colluvium Deposits (Holocene and Pleistocene)

Colluvium deposits are products of mass wastage that have reached their present position by gravity induced movements without the action of wind or water. They generally consist of massive to moderately well stratified, non-sorted to poorly sorted sediments with any range of particle size from clay to boulders and blocks. The character of any particular colluvium deposit depends upon the nature of the material from which it was derived and the specific process by which it was deposited. Figure 18 depicts typical colluvium deposits.

Figure 18 Colluvium Deposit

Talus cones form as a result of rock falls and are also included under this classification. Talus tends to accumulate at the base of a slope and form conical piles along natural ravines in the faces of cliffs as shown in Figure 19.

Figure 19 Colluvium Deposit, Talus Cone

This category also includes the unconsolidated material in the unglaciated portion of Yukon and Alaska that has not been placed by water as shown in Figure 20.

Figure 20 Colluvium Deposit, Unglaciated Terrain

1.5.1.6 Glaciolacustrine (Pleistocene)

Lacustrine deposits form when sediment is deposited in or along the margins of glacial lakes including sediments that were released by melting or floating ice. Generally glaciolacustrine sediments include: lake bed sediments consisting of stratified fine sand, silt and/or clay. They commonly contain ice-rafted stones and lenses of till and/or glaciofluvial material, and moderately sorted to well sorted, stratified sand and coarser beach sediment transported and deposited by wave action along the margins of glacial lakes. The figure below shows a typical glaciolacustrine deposit.

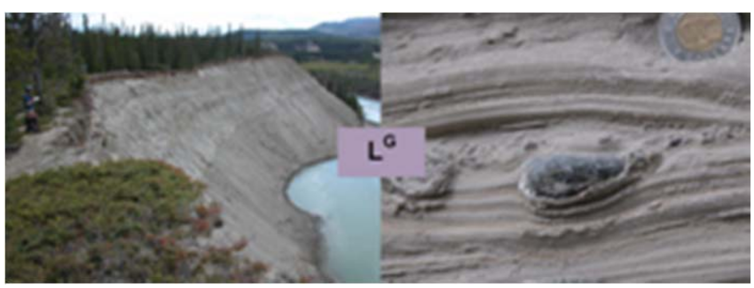


Figure 21 Glaciolacustrine Deposit

1.5.1.7 Glaciofluvial Deposits - (Pleistocene)

Sediments deposited by glacial meltwater in subaerial, subaqueous and subglacial environments. The sediment consists of massive to stratified coarse to fine grained gravel, sand and silt. These deposits tend to occur in the base of meltwater channels and mountain valleys. This category also includes eskers. Figure 22 presents the sediment types in a typical glaciofluvial deposit.

Figure 22 Glaciofluvial Deposit

1.5.1.8 Glacial Deposits - Moraine (Pleistocene)

Glacial till deposited directly by glacier ice. In general, moraines consists of well compacted to non-compacted material that is non-stratified and contains a heterogeneous mixture of particle sizes, commonly in a matrix of sand, silt and clay as well as minor pebbles, cobbles and boulders. This unit is characterized by a lack of distinctive topography and is at least 5 m thick. In Alberta and northeastern British Columbia the moraines are clay dominated. In the mountainous areas of British Columbia and the Yukon the moraines are silt dominated. Figure 23 shows a typical moraine deposit.

Figure 23 Moraine Deposit

1.5.1.9 Glacial Deposits - Moraine Veneer (Pleistocene)

This category is analogous to the Moraine category but the thickness of the moraine is 2 m or less and directly overlies bedrock. Isolated bedrock outcrops are common with this unit. Figure 24 shows a moraine veneer.

Figure 24 Moraine Veneer Deposit

1.5.1.10 Glacial Deposits - Stagnant Ice Moraine

Sediments resulting from the collapse and slumping of endglacial and supraglacial debris due the melting of buried stagnant ice at the glacier margin. Sediment is mainly till but includes local glaciolacustrine and glaciofluvial sediments, characterized by low to high relief hummocky topography. The figure below presents a view of typical topography.

Figure 25 Stagnant Ice Moraine Deposit

1.5.1.11 Glacial Deposits - Fluted Moraine

Glacially streamlined sediments that are mainly consisting of till. Terrain consists of alternating furrows and ridges or elongated ridges that run parallel to the local ice-flow direction. Category includes flutes and drumlins. Figure 26 presents a view of typical topography.

Figure 26 Fluted Moraine Deposit

1.5.1.12 Bedrock (Pre-Quaternary)

Bedrock was defined as any consolidated material unable to be removed using conventional mechanical construction methods. Bedrock was identified as outcrops or areas of rock covered by a thin mantle of unconsolidated or organic materials. Figure 27 shows a typical bedrock deposit.

Figure 27 Bedrock Deposit

1.5.2 Bedrock Classification and Depth to Bedrock

The type of bedrock and depth to bedrock underneath the proposed alignment was compiled. The bedrock type was determined from geological maps produced by the Alberta Geological Survey, Geological Survey of Canada and the Alaska Geological Survey. Published depth to bedrock was of a very regional nature, hence these values were estimated from the regional maps and modified by examining the topography and the distance to rock outcroppings.

1.5.3 Features Requiring Civil Structures

Geomorphic or anthropogenic features that would require civil structures were identified along the route along with the potential structure. These features include rivers, creeks, roads and pipelines. The evaluation for tunnels was completed in a different section. The type of civil structure required (bridge, culvert etc.) was based on a subjective evaluation of the feature.

1.5.4 Constructability Assessment

An assessment of the constructability of the route was made. This assessment is based on the estimated cut or fill depths, the depth to bedrock, the type of bedrock and the type of terrain. The product of the assessment is the estimated depth of topsoil, the cut or fill side slopes and the ease of excavation.

The depth of topsoil was estimated from the type of terrain. In general, moraines would have a topsoil and root zone ranging from 0.5 to 1 m thick. Glaciolacustrine deposits would have a thinner zone (0.3 to 0.6 m) due to the high clay content. Organic deposits would have no topsoil or a thin layer (0.3 m) depending whether they are water saturated. Glaciofluvial deposits would have a moderate topsoil thickness (0.6 m) due to their coarse nature. Due to active processes, topsoil on fluvial deposits tends to be thin (0.3 m). Colluvial deposits can have a range of topsoil thickness (0.3 to 1 m) depending on how active the process generating the colluvial deposit is. Eolian deposits will have a thin topsoil zone (0.3 m) due to the dry, coarse nature of the deposit.

Cut and fill slopes are based on the depth or height of the cut/fill. Cut/fills in unconsolidated material that are less than 4 m would have a 2:1 slope (horizontal to vertical). Cut/fills greater than 4 m but less than 8 m would have 2.5:1 slope or would have 2:1 slopes with an intermediate bench. Cut/fills greater than 8 m would have a 3:1 slope or 2:1 slopes with a number of intermediate benches based on the depth/height. In bedrock the cut would have a slope of 0.5:1 while the fill would be at 1.5:1. The one exception is on organic deposits. As these deposits can be wet to surface, a floating fill method was selected. This requires the placement of a geogrid on the non-stripped surface and maintaining sideslopes of 4:1 to distribute the load over the surface of the organic deposit.

Unconsolidated material can be typically moved with standard earth moving equipment. This is designated “standard”. Bedrock has been classified into three types: rippable, blasting wide pattern and blasting close pattern. “Rippable” bedrock is bedrock that can be ripped by attachments to earth moving equipment. As such it will have a slightly increased cost as compared to “standard”. Typically shale, some sandstones and some metamorphic rocks such as schists fall into this category. “Blasting wide pattern” is for bedrock that is too hard to be ripped but will fracture relatively easily with blasting. Limestone and some other rock types fall into this category. The cost for this method of excavation will be significantly higher than “standard”. “Blasting close pattern” is for very hard bedrock such as volcanic rock and granodiorite. Due to its hardness, the number of blast holes is increased compared to a wide pattern blast and the length of the holes is shorter leading to increased frequency of blasting. Costs for this type of blasting are typically 50% higher than a wide pattern blast.

1.5.5 Classification

Each segment of the route was analyzed and the terrain, bedrock and depth to bedrock classified along the proposed alignment. The features requiring civil structures, along with the proposed structures, and the constructability was also identified. The tables in Appendix B-6 present the detailed classification. Summaries for each segment are presented below.

1.5.5.1 Segment 1

Segment 1 runs from the start of the route northwest of Syncrude’s operations (55 km north of Fort McMurray, Alberta) to 30 km southeast of Fort Vermillion, Alberta. It generally travels through moraine and organic deposits with some glaciolacustrine deposits near Fort McMurray and Fort Vermillion. In general, the route has low relief except for a section in the middle of the segment where more rolling relief is encountered. The following is a summary of the terrain traversed by the route in Segment 1:

Table 16 Segment 1: Classification

Terrain Unit	Percentage of Route	Total Distance (Kilometres)
Organic	37%	118
Moraine	32%	102
Glaciolacustrine	18%	57
Stagnant Ice Moraine	11%	35
Fluted Moraine	1%	3
Fluvial	1%	3

The type of bedrock along the route consists of mudstone, sandstone and siltstone at the following proportions:

- Mudstone at 75%
- Siltstone at 19%
- Sandstone at 6%.

The depth to bedrock ranges from 5 to 70 m.

The features that will require civil structures along the route include 23 unnamed creeks, two named creeks and the following rivers:

- Dover River
- Dunkirk River
- Liege River
- Panny River
- Mikkwa River (two separate crossings), and
- Wabasca River.

1.5.5.2 Segment 2

Segment 2 runs from the conclusion of Segment 1 (30 km southeast of Fort Vermillion, Alberta) to 49 km east-southeast of Fort Nelson, British Columbia. It generally travels through moraine, organic deposits and glaciolacustrine deposits. In general the route has low relief. The following is a summary of the terrain traversed by the route in Segment 2:

Table 17 Segment 2: Classification

Terrain Unit	Percentage of Route	Total Distance (Kilometres)
Glaciolacustrine	56%	220
Organic	22%	86
Moraine	21%	83
Fluvial	1%	4

The type of bedrock along the route consists of mudstone and shale at the following proportions:

- Mudstone at 91%
- Shale at 9%.

The depth to bedrock ranges from 10 to 50 m.

The features that will require civil structures along the route include 42 unnamed creeks, 6 named creeks and the following rivers:

- Bear River
- Peace River
- Caribou River
- Ponton River
- Busche River
- Chinchaga River
- Little Hay River, and
- Hay River.

Other features include 10 road crossings, one highway crossing, a CN Rail line crossing and a buried high pressure pipeline.

1.5.5.3 Segment 3

Segment 3 runs from the conclusion of Segment 2 (49 km east-southeast of Fort Nelson, British Columbia) to Watson Lake, Yukon. It travels through the northern plains, and into the mountains along the southern portion of the La Biche Range. It then moves into the Liard Plain to Watson Lake. It generally travels through moraine and organic deposits until the mountains then it goes through moraine, moraine veneer and colluvium in the uplands and glaciofluvial and fluvial deposits in the base of the valleys. The route has low relief until west of the Liard River crossing where it moves into mountainous terrain. The following is a summary of the terrain traversed by the route in Segment 3:

Table 18 Segment 3: Classification

Terrain Unit	Percentage of Route	Total Distance (Kilometres)
Moraine	29%	155
Glaciofluvial	19%	101
Colluvium	12%	64
Organic	10%	53
Fluvial	9%	48
Moraine Veneer	8%	42
Glaciolacustrine	7%	37
Bedrock	2%	11

The type of bedrock along the route consists of mudstone and shale at the following proportions:

The type of bedrock along the route consists of shale, conglomerate, siltstone, sandstone, chert, dolomite, limestone and slate at the following proportions:

- Shale at 45%
- Dolomite at 16%
- Limestone at 13%
- Slate at 12%
- Sandstone at 6%
- Siltstone at 4%
- Conglomerate at 3%
- Chert at 1%.

The depth to bedrock ranges from at surface to 20 m.

The features that will require civil structures along the route include 58 unnamed creeks, 12 named creeks and the following rivers:

- Sahtaneh River
- Kiwigana River
- Liard River (three separate crossings)
- Beaver River
- Crow River (two separate crossings)
- Smith River
- Coal River, and
- Hyland River.

Other features include one road crossing, four highway crossings, and a buried high pressure pipeline.

1.5.5.4 Segment 4

Segment 4 runs from the conclusion of Segment 3 (Watson Lake, Yukon) to 28 km north of Carmacks, Yukon. It travels along a glacial valley on the eastern side of the Pelly Mountains for approximately 200 km before entering the Yukon Plateau. The route runs through the plateau for approximately 100 km where it enters the Tintina Trench; an ancient major fault structure now buried by sediment. The route traverses the trench for approximately 100 km whereupon it enters the Pelly Mountains through a glacial valley. After approximately 100 km, the route re-enters the Yukon Plateau to the end of the segment.

The route generally travels through moraine in the mountains and glaciofluvial deposits in the base of the valleys. The following is a summary of the terrain traversed by the route in Segment 4:

Table 19 Segment 4: Classification

Terrain Unit	Percentage of Route	Total Distance (Kilometres)
Moraine	48%	288
Glaciofluvial	22%	132
Moraine Veneer	13%	78
Stagnant Ice Moraine	6%	36
Fluvial	3%	18
Colluvium	2%	12
Fluted Moraine	1%	6
Organic	1%	6
Glaciolacustrine	1%	6
Bedrock	1%	6
Eolian Sand	1%	6

The type of bedrock along the route consists of shale, conglomerate, siltstone, sandstone, chert, quartzite, dolomite, limestone, volcanic rocks, phyllite, gneiss, schist, basalt, amphibolites, quartz porphyry, diorite, granodiorite, dacite tuff and slate at the following proportions:

- Limestone at 16%
- Shale at 13%
- Conglomerate at 12%
- Phyllite at 8%
- Quartz Porphyry at 6%
- Dacite Tuff at 6%
- Siltstone at 5%
- Volcanic rocks at 5%
- Granodiorite at 5%
- Chert at 4%
- Gneiss at 4%
- Schist at 4%
- Amphibolite at 3%
- Slate at 2%
- Basalt at 2%
- Diorite at 2%
- Dolomite at 1%
- Sandstone at 1%
- Quartzite at 1%.

The depth to bedrock ranges from at surface to 25 m.

The features that will require civil structures along the route include 115 unnamed creeks, 15 named creeks and the following rivers:

- Francis River
- Tachina River
- Hoole River
- Ketza River
- Lapie River
- Magundy River (two separate crossings) , and
- Tatchun River.

Other features include 16 road crossings and 13 highway crossings.

1.5.5.5 Segment 5

Segment 5 runs from the conclusion of Segment 4 (28 km north of Carmacks, Yukon) to Delta Junction, Alaska. It travels along the Yukon River valley in the Yukon Plateau for approximately 100 km before following the Yukon River into the Dawson Range, a range of unglaciated mountains, for another 100 km. The route turns west into the White River valley for 50 km then up the Ledu River valley and into Alaska. The route follows along the Ledu River to the rivers headwater, over a topographic divide, and down into the Tanana River Valley (approximately 90 km). The route then follows the Tanana River to its terminus at Delta Junction. Note that the unglaciated terrain is highly dissected by streams forming a very rough terrain.

The route generally travels through glaciofluvial and fluvial deposits in the base of the valleys and colluvium in the unglaciated uplands of Alaska and Yukon. Eolian sand and silt is found in the valleys in the unglaciated uplands. The following is a summary of the terrain traversed by the route in Segment 5:

Table 20 Segment 5: Classification

Terrain Unit	Percentage of Route	Total Distance (Kilometres)
Glaciofluvial Moraine	35%	209
Colluvium	32%	191
Fluvial	18%	107
Eolian Sand	5%	30
Moraine	4%	24
Bedrock	2%	12
Moraine Veneer	1%	6
Organic	1%	6

The type of bedrock along the route consists of quartzite, gneiss, schist, basalt, granodiorite, granite, dacite tuff and volcanic breccias at the following proportions:

- Schist at 51%
- Granite at 14%
- Gneiss at 14%
- Granodiorite at 9%
- Volcanic breccia at 5%
- Basalt at 5%
- Dacite Tuff at 1%
- Quartzite at 1%.

The depth to bedrock ranges from at surface to 30 m.

The features that will require civil structures along the route include 91 unnamed creeks, 28 named creeks and the following rivers:

- Yukon River
- Selwyn River
- White River
- South Fork Ladue River
- Ladue River (two separate crossings)
- Tanana River
- Tok River
- Robertson River
- Little Gerstle River, and
- Gerstle River.

Other features include 33 road crossings, five highway crossings and seven crossings of an above ground pipeline.

1.6 Bridges and Structures

The section of the report will review the structural aspects of bridges and culverts and other major structures within the alignment proposed for the Alberta to Alaska Railway Preliminary Feasibility Study. Along that alignment, there are an anticipated 70 major railway bridges at water crossings. This section will summarize the estimated requirements for rail carrying structures, the process used to arrive at estimates and some suggestions for consideration in applying these numbers and for moving into the next phase of the project.

1.6.1 Estimated Bridge Requirements

1.6.1.1 Final Alignment Specified

As has been previously mentioned in the Route Selection sections, the starting point for all aspects of the right of way and railroad is the mapping. As with the previous sections the bridge and structural requirements were based on the available public data sets and information contained in previous studies along parts of the proposed corridor. The determination of structures was an iterative and collaborative effort between the Bridge, the Route Selection/Track & Civil and the Hydrology teams. The bridge team was the final user of the alignment but was integral in the process of best determining the final alignments. A number of information sources were used including:

- Color coded topography mapping
- 10 m Contours
- Aerial photos
- Existing maps
- Ground and track profiles
- Hydrologic data Hydro data, all locations; including major river segments and the tables provided by the Hydrology team for proposed bridge, tunnel and viaduct locations.

1.6.1.2 Bridge Parameters

For each of the 70 required railway bridges, three values were estimated as described and illustrated in the figure below. The input values of track elevation and hydrology data are shown with the final bridge parameters in Appendix B-4.

The **Out to Out Length** is the length of the bridge from the extension of the slope line to a point behind the abutments where that line intersects the track. This length is in meters.

The **Open Area** is the area of the bridge opening between the track elevation and the ground profile. It is given in m^2.

The **Total Span Length** is the sum of the length of the spans selected to bridge the crossing. For this estimate, 5 typical spans were selected:

- A 12 m Double Voided Box (DVB) span, 1.2 m deep
- A 20 m Deck Plate Girder (DPG) span, 2.0 m deep
- A 30 m DPG span, 3.0 m deep
- An 80 m Deck Truss (DT) span, 12.0 m deep
- An 80 m Through Truss (TT) span, 1.2 m deep.

The depth is measured from the base of rail to the underside of the span. The selection of spans was dependent on:

- The length of the crossing required
- The clear depth between track elevation and high water level (HWL)
- Depth of opening at abutments
- If a viaduct structure is expected.

The Out to Out length is typically longer than the Total span length, because it includes the length of track beyond the abutments; however, there may be an exception where the selected standard span lengths could not be combined to make the exact required opening length and a slightly longer bridge is required.

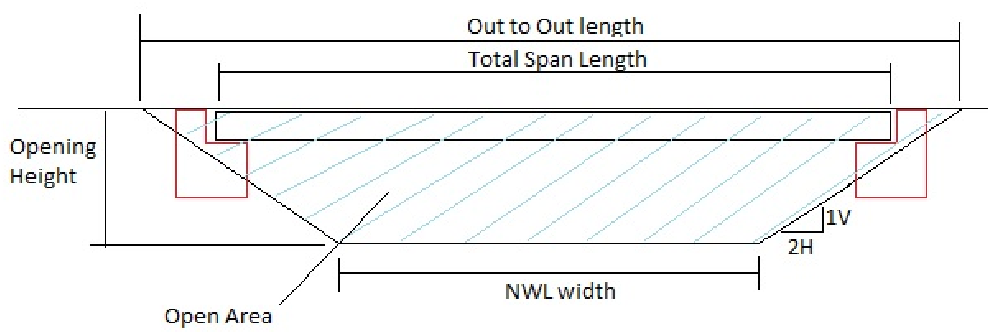


Figure 28 Typical Bridge Parameters

Bridges Estimates Based on Hydrologic Data

For bridges with a regular opening, there are three parameters which are based on mathematical formulae with the input values of opening height, and normal water level width along the track. The following is the definition of those values.

For these bridges:

$$Open\ Area = wh + 2h^2$$

$$Out\ to\ Out\ Length = w + 4h$$

Where:

w = Normal Water Level (NWL) width along the track (m)
h = opening height (m)

Span length was determined to be the minimum possible to fit inside abutments with a maximum height from groundline on the face of the abutment to base of rail of 12.0 m and a slope of 2 horizontal: 1 vertical (2H:1V).

1.6.1.3 Bridges Estimated From Track and Ground Profiles

In many cases, the track and ground profiles indicate that the bridge crosses a large valley in addition to the actual water crossing. Where this is the case, the assumption of a 2H:1V slope from the NWL width was insufficient to define the required length of bridge. In these cases, the three parameters were extrapolated from the profile drawings.

For these bridges, the location of the abutments was set to be where the distance from the ground line to the base of rail is approximately 12.0 m. The target Total span length was then extrapolated from there.

By assuming a 2H:1V slope from the front of the abutments to the track elevation, the Out to Out length was then calculated as:

$$Out\ to\ Out\ Length = Total\ span\ length + 2 * 2 * 12$$

The opening area was also extrapolated from the profile, and rounded to the nearest 100 m^2.

1.6.1.4 Substructure Review

On receipt of geomorphology data for the alignment, conceptual substructure requirements and associated structural restrictions or possibilities could be established.

1.6.1.5 Optimization of Standard Spans

At this phase, 5 typical spans were selected for use in determining the span layouts. In addition to alignment refinements, information on construction methods, shipping restrictions, project timeline, and design criteria could be applied to a more analytical determination of optimum span types and lengths.

1.7 Tidewater Terminal

The Valdez Marine Terminal marks the end of the Trans Alaskan Pipeline System. Located in the northeast corner of Prince William Sound, the Terminal lies on more than 1,000 acres of land. The facility was designed for loading crude oil onto tankers and holding crude oil so that North Slope production can continue without impact from the marine transportation system. There are 14 storage tanks in service, facilities to measure the incoming oil, two functional loading berths, and a power plant. Figures 29 and 30 depict the tidewater terminal location. [1]

Figure 29 Valdez Terminal – Aerial View

[1] *Port Valdez Company http://portvaldezco.com/*

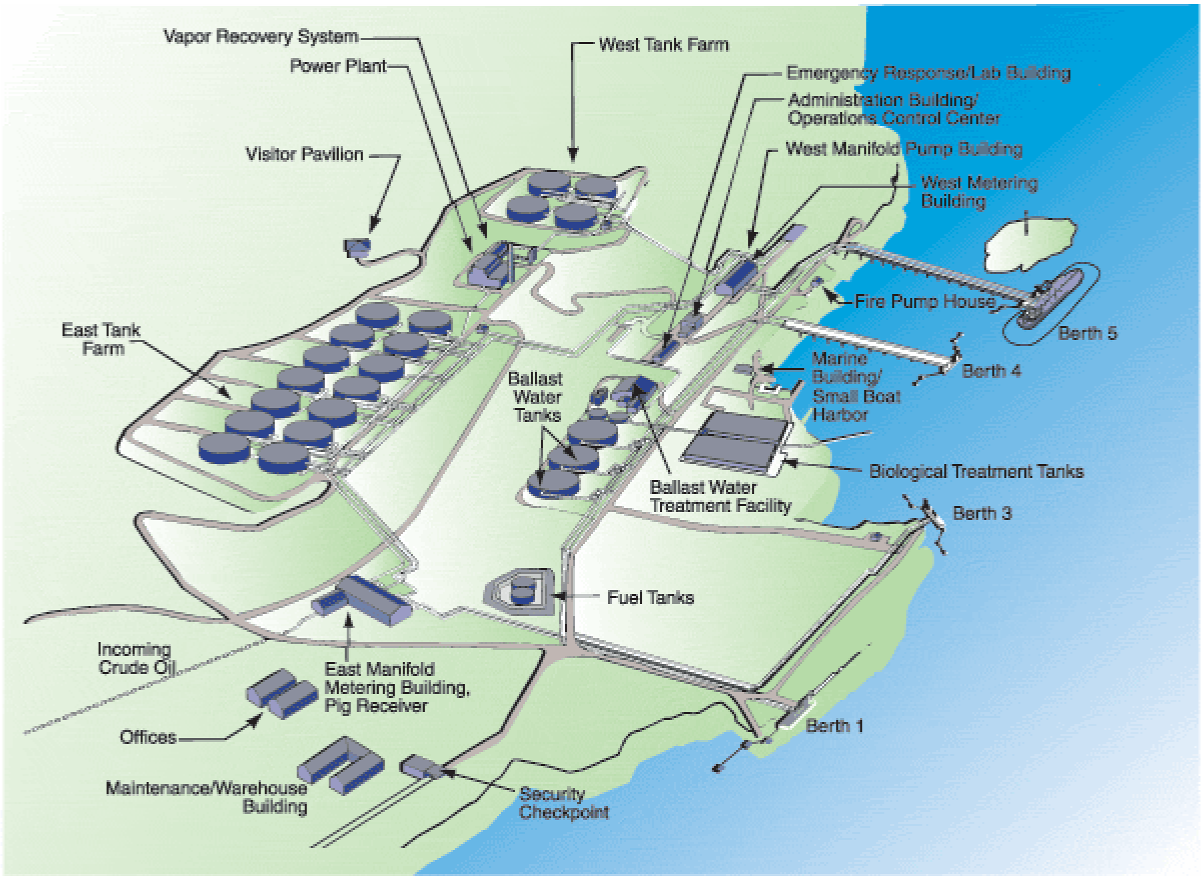


Figure 30 Valdez Terminal Schematic

At the Terminal, crude oil is measured and stored, then loaded onto tankers and sent to market. Tankers tie into a berth, where they hook into loading arms to take on crude oil. Before loading begins, crews protect the surrounding waters by placing an oil spill containment boom around the berth and tanker. The Valdez Marine Terminal also has a facility to purify storm water, other Terminal drainage water, and primarily ballast water – the water that fills tankers' hulls to stabilize them before they take on crude cargo. The Ballast Water Treatment System sends oily water through multiple processes to strip it of any hydrocarbons.

1.7.1 Basic Information for the Tidewater Terminal

- Located in Port Valdez, the northern most ice-free port in the U.S.
- Total area – 1,000 acres
- Cost to build - $1.4 billion
- Elevation – sea level to 660 ft. All facilities except berths 15 ft. or higher
- 18 storage tanks constructed, 14 in service as of January 1, 2012
- Current holding capacity in crude oil with 14 tanks – 7.13 million bbl.
- Two functional loading berths with vapor recovery capacity.

Alyeska does not own the tankers loaded with crude at the Valdez Marine Terminal. They are owned by shipping companies who contract with producers to carry crude oil to market. The entire berthing and loading process takes about a day to complete, and the largest tankers carry up to 2 million barrels of oil. More than 20,000 tankers have loaded at the VMT since 1977.

Tankers carry ballast on the way to the Valdez Marine Terminal. Ballast is water taken on by a tanker to stabilize the ship when it is not carrying crude oil. Today, many tankers have separate tanks for water and cargo, but some still carry ballast and crude oil in the same compartment. The Terminal's Ballast Water Treatment Plant can process any ballast contaminated with oil.

Alyeska, through its Ship Escort/Response Vessel System, is the primary oil spill response contractor for the shipping companies and provides response equipment and personnel in the event of threat of an oil spill. The SERVS duty office works closely with the U.S. Coast Guard to monitor vessel traffic and ensure tankers have a safe route through Prince William Sound. The U.S. Coast Guard will close the Port of Valdez in extreme inclement weather, and Alyeska does not load crude oil if the wind speed exceeds 40 knots.

1.7.2 Pipeline Low Flow Impact Study [2]

"The Low Flow Impact Study is a critical step toward addressing the many challenges associated with declining throughput," said Tom Barrett. "I want to thank Pat McDevitt and the study team for a job well done." "The study findings make it clear that the technical challenges compound and increase in complexity as throughput declines. The simplest, most direct and cost effective path to dealing with these challenges is to stop the decline by adding more oil." The LoFIS identified potential challenges with throughput levels between 600,000 and 300,000 barrels per day (BPD). Figure 31 is a graph of the pipeline flows since 1979. Potential challenges include:

- Water, present in oils as small droplets, is expected to separate out in a layer at the bottom of the pipe at 500,000 BPD and lower. Separated water will increase the potential for ice formation and corrosion.
- Wax build up in the pipeline is present at current throughput levels and will continue to increase as throughput declines.
- As throughput drops below 550,000 BPD, oil temperature will have the potential to drop below the freezing point of water and form ice in the pipeline during the winter months. Ice could damage pumps and equipment.
- Crude oil temperatures at 350,000 BPD could allow soils surrounding buried sections of the pipeline to freeze, which would create the potential for ice lenses. Ice lenses could cause movement and damage the pipeline via frost heaves.

Mitigation measures recommended in the study include:

- Minimize the impact of temperature decline by adding heat and insulation.
- Modify the water and temperature specifications for crude oil entering TAPS.
- Adjust the pipeline pigging program as throughput declines.

[2] *Low Flow Impact Study http://www.alyeska-pipe.com/TAPS/PipelineOperations/LowFlow*

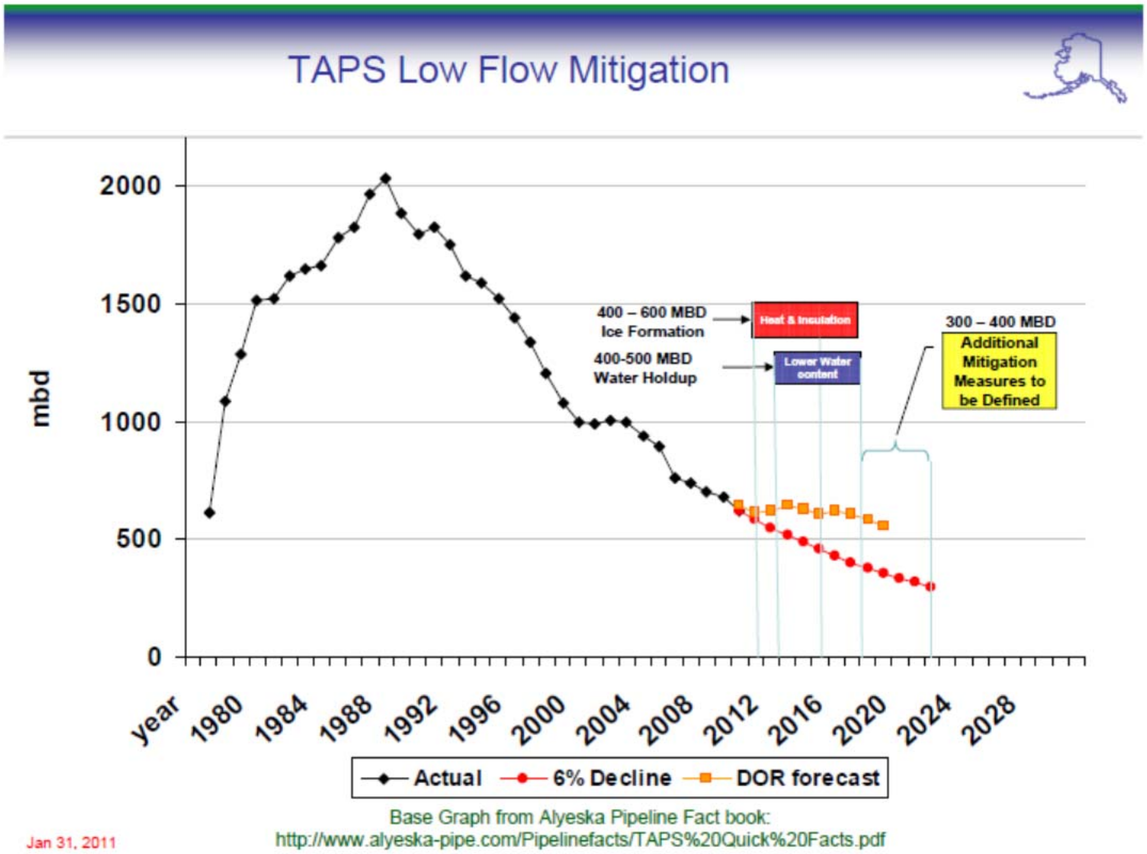


Figure 31 Annual Product Movement in the TAPS System