Design and construction of a 160-Metre-Long Wood Bridge in Mistissini, Québec

Design und Konstruktion von einer 160 Meter Langen Holzbrücke in Mistissini, Québec

Conception et construction d'un 160 Mètre longue ponts de bois à Mistissini, Québec

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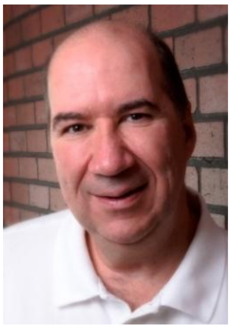

Grégoire Richard

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ABSTRACT: This article outlines the process and challenges involved in designing and building a new structure to span the Uupaachikus Pass in Mistissini, Quebec. The 160-metre-long bridge was designed using semi-continuous arches made of glued laminated wood (glulam) girders. The bridge is 9.25 metres wide and has spans of 37, 43, 43 and 37 metres. The Glulam Bridge features straight girders with a maximum length of 24 metres attached to 15-metre arched girders by means of steel plate assemblies. The arches are connected to the piers and abutments using pins.

KEYWORDS: Bridge, Mistissini, Glulam, Arch, Curved girders

1. Introduction

The project is located in Mistissini, Québec, Canada, about 600km north-east of Québec city.

The construction of a bridge, crossing the Uupaachikus pass, west of Mistissini has two main objectives:

- access of to a larger territory for the Cree community, and;
- access to a large gravel pit in order to satisfy the growing demand for gravel used in the construction projects of the community.

This project also includes the construction of an access road to the gravel pit and the prolonging of Main Street to the bridge. The project was completed on behalf of the Cree community of Mistissini.

In this project, Dessau offered three different possible deck structures: steel-concrete, steel-wood and glulam wood. Because of the remote location of the project, in Northern Quebec, the design favoured the use of local materials. The raw materials in such areas are about 25% more expensive than in larger cities. A traditional bridge (steel-concrete of steel-wood) would have been slightly more expensive than the glulam solution.

In the end, Chantier Chibougamau, located 90 km from the construction site, was awarded the contract for the wood structure, ensuring that the wood used on this project came from sustainable forests in the surrounding area.

Finally, the architectural and environmental aspects of the glulam solution were greatly appreciated by the client.

This article deals with the various challenges faced by the engineers during the design and construction phases and provides details of how the glulam structure was designed and built. The first part of the article focuses on the design of the bridge, whereas the second part approaches the construction of the bridge itself.

2. Carbon footprint

It is interesting to compare the carbon footprint of the wood structure and a structure with a mixed steel/concrete deck. Since we studied both solutions to compare construction costs, it is fairly easy to calculate the carbon footprint of the two solutions using the quantities established.

The values used to calculate carbon emissions are taken from the Athena Sustainable Materials Institute [1]. Athena performs life cycle assessments of all types of construction materials. We compared these values to those produced by ADEME [2] (*"Agence de l'Environnement et de la Maîtrise de l'Énergie"*, a government agency reporting to France's ministry of the environment, sustainable development and energy). We used the Athena values, as they better represent typical Canadian production.

For wood, the manufacturer, Nordic [3], commissioned an analysis of the manufacturing process. The value we have used is thus taken directly from that study.

The following values show the weight of equivalent CO_2 per m³ of material produced and transported. The values take into account the carbon emissions from carbon dioxide (CO_2) and methane (CH_4), the main gases responsible for the greenhouse effect. Upon analysis, it appears that the emissions stem primarily from carbon dioxide, as methane emissions are very low, on the order of 0.1% that of CO_2.

Table 1: CO_2 equivalent emissions by material

Materials	Athena (kg/m³)	Athena Average (kg/m³)	ADEME (t)
Concrete 30MPa	301		
Concrete 60MPa	304	302	209
Reinforcing steel	4,697		
Hot-rolled plates	8,968		
Galvanized steel plates	13,678		
Hardware	11,543	9,721	8,580
Bituminous concrete	127	127	145
Nordic LAM timber	-765 [3]	-765 [3]	-825

It is clear that the wood manufacturing process absorbs rather than emits CO_2. Wood can be considered a carbon sink only if it comes from a sustainably renewed forest and the life expectancy of the structure is sufficiently high (generally 100 years). These conditions are met on our project.

We compared the wood bridge solution with the equivalent mixed steel/concrete solution. The quantities estimated for the mixed steel/concrete bridge were taken from the preliminary design, in which we proposed various solutions to our client.

Table 2: CO_2 equivalent emissions for wood bridge

Materials	Volume (m³)	Unit emissions (kg/m³)	Total emissions (t)
Wood deck	1283	-765	-981
Steel	4.6	8 968	41
Galvanized steel	0.6	13 678	8
Foundations reinforcing steel	12.4	4 697	58
Foundations concrete	1197	301	360
Bitumen	112	145	16
Total			**-497**

Table 3: CO_2 equivalent emissions for steel-concrete bridge

Materials	Volume (m³)	Unit emissions (kg/m³)	Total emissions (t)
Concrete deck	370.0	301	111
Concrete sidewalks	103.2	304	31
Deck reinforcing steel	8.0	4 697	38
Steel girders	37.8	8 968	339
Hardware	1.9	11 543	22
Foundations reinforcing steel	12.4	4 697	58
Foundations concrete	1197.0	301	360
Bitumen	72.8	145	11
Total			**+969**

Overall, the total carbon emissions for the wood bridge were negative, which is a very good result.

The total difference between the two solutions is 1,472 tonnes of CO_2, equivalent emissions, which is equal to the CO_2 emitted in combustion 640,000 litres of gas.

3. Geometry

The geometric design of the Mistissini Glulam Bridge itself is innovative. The glulam girder and arch assembly creates a series of semi-continuous arches. The connectors between the arched girders and the piers and abutments are designed to provide a pivot-type connection. The connectors between glulam girder segments are designed to transfer shear, compression and bending forces. The properties of glulam make it possible to eliminate expansion joints over the 160-metre bridge, giving the structure greater durability. All of the bridge bearings are fixed which distributes the seismic effects over all foundation units, thus reducing their cost.

The wood structure is protected by waterproof decking, including different layers of plywood and a waterproof membrane. The bridge has a steel guardrail and a steel plate covered with Bimagrip to cover the wood CLT walkway. The wood curb is also covered with a steel plate.

3.1. Architectural design

Figure 1: General view of the bridge

In order to span the 160 metres of the Uupaachikus pass, we chose four (4) spans of 37, 43, 43 and 37 metres. The arches had been added in order to minimise the effects of the interior spans and adding an architectural feature, key aspect to our concept.

3.2. Geometrical design

As shown in Figure 2, the division of the spans (37/43 m) has been set in order to balance the stresses and deflections of the bridge in each span.

The Mistissini bridge features straight girders with a maximum length of 24 metres attached to 15-metre arched girders by means of steel plate assemblies.

The radius of the arches is limited by the maximal bending of the 38mm wood lamellas in the manufacturing process of the glulam arches. Lower radiuses are possible with thinner lamellas but with a higher manufacturing cost. For our bridge, the minimal radius of the arches is 10.8 m using 38 mm lamellas.

As shown in Figure 3, the arched girders are paired, and placed between two straight girders. Each one of these systems represents one of four main girders.

The section of the bridge is determined by the two traffic lanes and the sidewalk for a total width of 9.25 m. The addition of a fence was requested by the client for security reasons for the pedestrians.

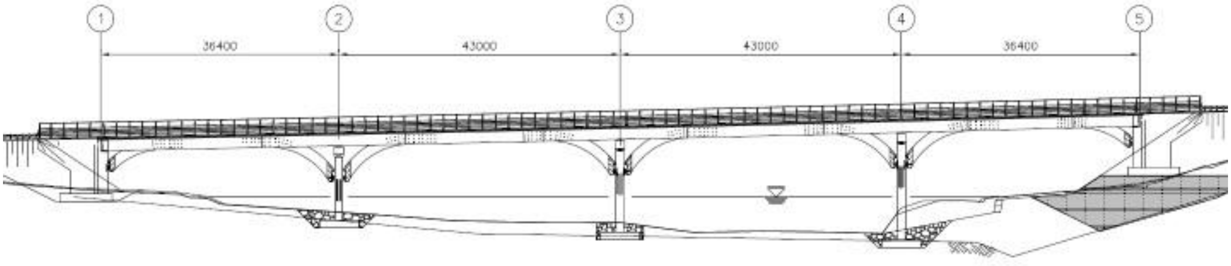

Figure 2: Elevation of the bridge

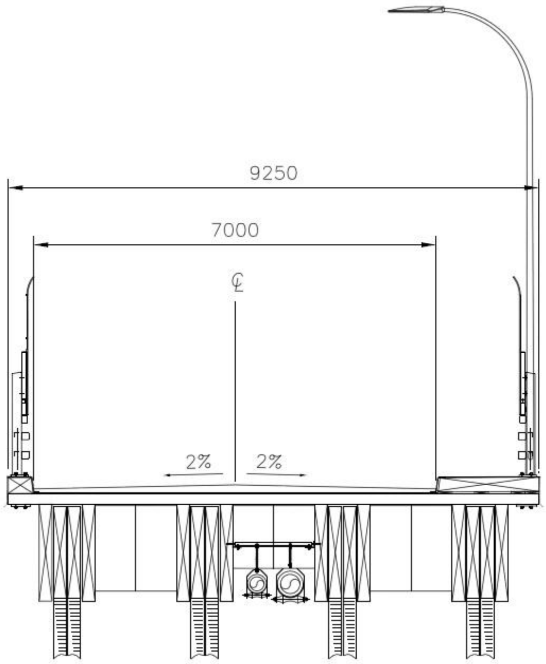

Figure 3: Section view of the bridge

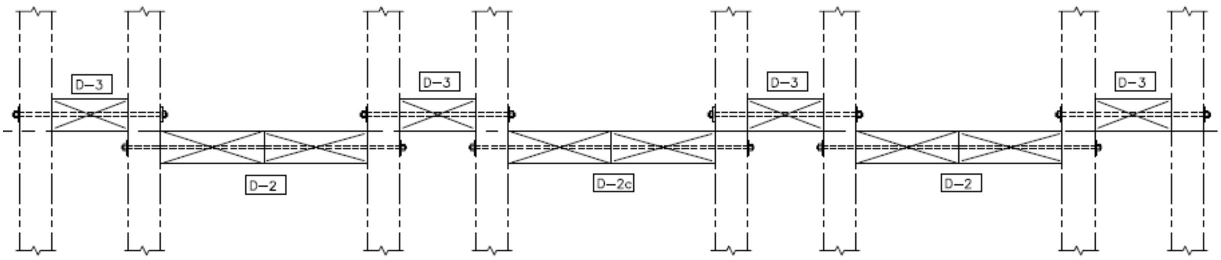

Figure 4: Position of alternate diaphragms

Since the timbers manufactured by Chantiers Chibougamau cannot be longer than 24 metres, the splices have been strategically combined with the junctions of the arches. The length of the straight portion of the arches was adjusted to accommodate this assembly strategy.

Since the effects of temperature on wood are less than the effects of temperature on steel and concrete (thermal expansion coefficient for wood of 5.0x10e-06), we opted for fixed supports on all the sections to eliminate the need for expansion joints.

Note that moisture levels increase in winter and drop in summer, which cancels out some of the thermal effects.

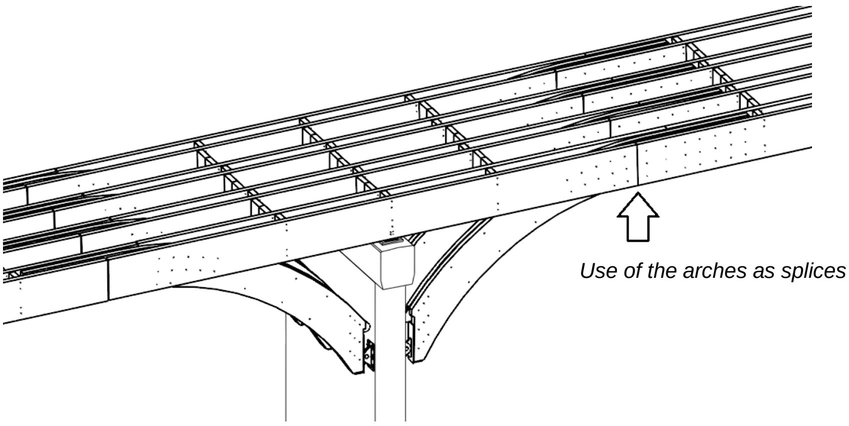

Figure 5: Splicing strategy

We fastened the bottom of the arches to the pier using pins. Note the additional anchoring above the pin to facilitate installation and eventual maintenance or repair of the pins, as seen on Figure 6.

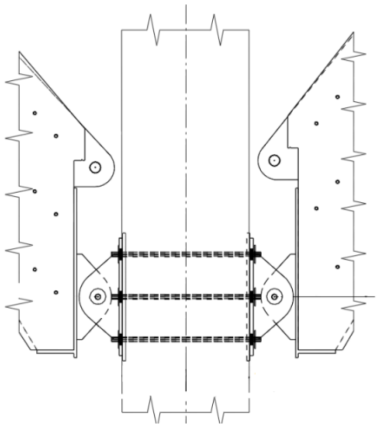

Figure 6: Connecting the arches using pins

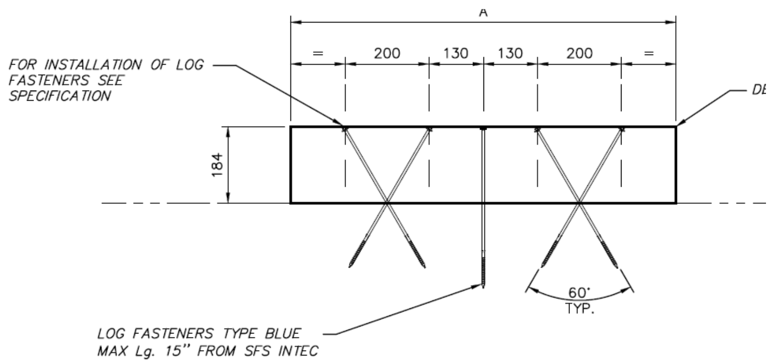

Figure 7: Fastening the deck to the main girders

The bridge deck is made up of glued-laminated boards (184x921) applied across the full width of the bridge, i.e. 9,250 mm. The boards are fastened using 15" Blue Max fasteners. Five fasteners are used above each girder, or 40 fasteners per board.

Because the deck is straight, the side-to-side slope is adjusted using bituminous concrete. The thickness of the bituminous concrete thus varies from 65 mm to 135 mm in the middle of the lanes.

3.3. Main features

3.3.1. Arches

The arches for sections 1 and 5 are identical but with different angles, because the bridge slopes by about 2.5 %. The other arches are divided into two identical groups, those pointing east and those pointing west.

Due to the shape and placement of the arched girders with the continuous girders, bending moment is always negative in the arches, meaning that bending reduces the intrados bending radius. Even if the girder is curved, the bending effect does not penalize the bending resistance.

In fact, this effect probably increases the value of the shear resistance because moments increase the perpendicular compression between the lams.

However, Canadian standards S6-06 – Chapter 9 [4] do not consider this effect to increase shear strength.

3.3.2. Steel plate assemblies

Calculation of the connections between the straight girders and the arches requires very specific attention. Because we are using the ends of the arches as splices, we must transfer all strain along the straight girders and the arches.

The design of the steel plate assemblies is based on CSA O86-9 –Section 10.9.4.2 [1] and uses Φ6x60mm annularly threaded nails. Each plate uses 38 nails. This assembly allows developing 70 kN in all directions. There is only one type of connecting plate, and only the number of plates used in the splice zone varies in order to resist shear where the parts interface.

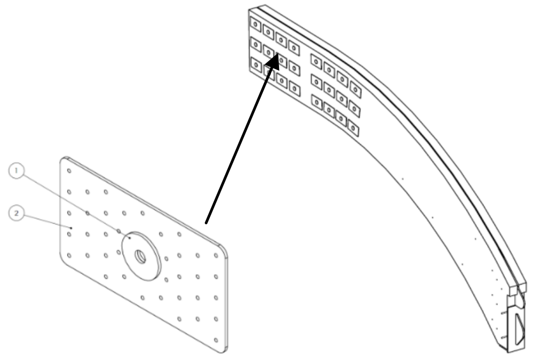

Figure 8: Steel plate assembly and arch

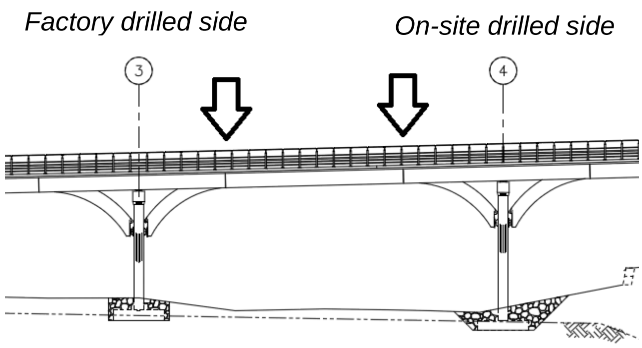

Figure 9: Pre-drilled and on-site drilled connectors

Since wood warps vertically to a substantial degree, we selected an arrangement that allows the wood to warp without causing horizontal cracking. If we had used steel plates along the entire top, significant tension loading perpendicular to the grain would have been induced when the humidity changed. According to the above calculations, we have estimated a maximum change in the height of the main girders of 10 mm.

The spreading out of the connectors makes it possible to achieve the desired strength while avoiding tension loading perpendicular to the grain.

The crushing strength or transverse crack resistance would be much too low to develop shear forces at the splices.

Another important detail, shown in Figure 9, is that the assembly principle selected dictates that the arches be assembled first and then fastened to the straight girders at the end. Because the geodesic positioning of the piers and abutments may vary from the theoretical positions by a few millimetres, we pre-drilled all connectors on the left side but not the right side. The right side of the connectors and girders must be drilled on site in order to adjust the position of the straight girders during construction. In this way, we can provide an assembly tolerance of about 20 mm.

3.3.3. Waterproofing deck

Water is the main enemy of wood structures. For this reason we have to take extra measures in the details of the bridge in order to minimize water infiltration. We combined several construction details to obtain a waterproofing deck.

The longitudinal slope of the bridge is on average 2.5%, favouring a fast flow to a drain on the west side of the bridge, in the abutment. Drains are known to leak and cause infiltration problems. Therefore, having no drains on the bridge reduces the possibility of infiltration.

The deck is composed of several layers of materials detailed in Figure 10 and 11. Using a bituminous coating, several sheets of membrane, a 20 mm marine plywood (that can be replaced if needed in future renovations); we are able to reduce to a maximum the possibility of water penetration and stagnation in the wood.

Because of the variable opening (up to 7 mm) between the transversal glulam deck panels, the plywood main function is to prevent the bituminous coating to crack over the openings.

The curb is manufactured by one glulam panel (460 mm x 260 mm) covered by a membrane overlaid by the deck membrane, and covered by a flashing on the outside and a steel plate protecting the wood from the snowplow. The sidewalk is based on the same principle but the surface is covered with a Bimagrip non-slip coating in order to give proper adherence to pedestrians.

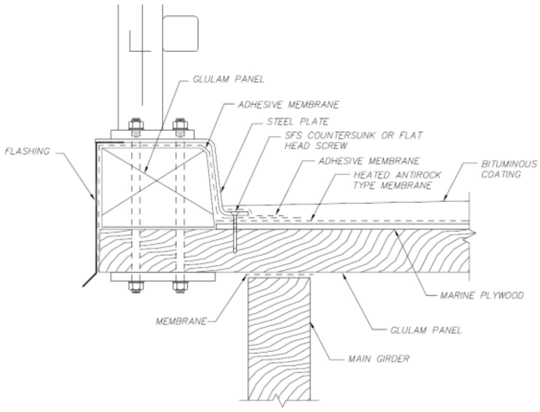

Figure 10: Detail of the waterproofing deck, curb side

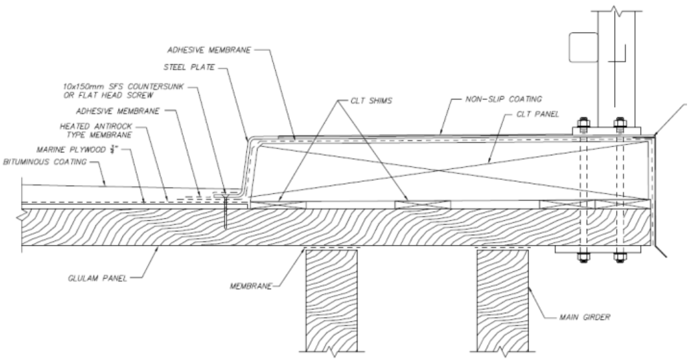

Figure 11: Detail of the waterproofing deck, side-walk side

The whole assembly is bolted via the anchors of the barriers, and forms a solid set capable to resist to truck impacts.

In order to prevent water for accumulating between the assemblies all the girders are spaced 25 mm. In the places where there is no or little stress, a 25 mm spacing washer is used. In the high stressed areas of the girders, this function was included in the steel plate assemblies. The steel plate is 12.5 mm thick, and there is one on each side of the girder, ensuring a 25 mm gap.

4. Design

4.1. Standards and design criteria

Canadian standards S6-06 [6] (in particular [4]), and O86-09 [7] were used to design the bridge. The properties of the materials supplied by Chantiers Chibougamau are provided for dry conditions of operation, so we opted to calculate the resistances using [7] for moist conditions.

In addition, [4] provides the properties of all species of wood and includes reduction coefficients for use in moist conditions, which explains the lack of K_S factors in the capacity calculations. According to [7], the KS factor is the condition of use coefficient. The values change if it is a dry environment or a moist environment and depending on the type of resistance calculated.

In the end, there were a few differences between the capacities calculated using [7] and those calculated using [4]. Among other things, the calculation of Mr affected by lateral slope in accordance with standard [4] is more permissive than with [7]. As for the Vr value, it is more permissive when calculated in accordance with [7] than [4].

The Mr value calculated in accordance with [4] is approximately 12% higher than the value obtained using [7]. The difference is directly related to the conditions of use coefficients (0.9/0.8 = 1.125) for moist environments for the two standards.

The Vr value calculated using [7] is approximately 23% higher than the value calculated according to [4]. That is the only value in [4] that is lower than the value calculated using [7]. The Vr value calculated using [4] does not appear to be up to date and should be revised.

Note that the design of the connections was calculated using [7], as dictated by [4]. Since the KS coefficients appear in the calculation of the connectors, we must therefore:

– Use a value of K_S = 1.0 (dry environment) if using material properties identified in [4];
– Use K_S values for moist environments if using material properties identified for dry conditions.

4.2. Dynamic load allowance and deflection

The dynamic load allowances indicated in sections 3.8.4.5.2 and 3.8.4.5.3 of standard S6-06 are multiplied by 0.70, which is an advantage over other types of construction.

The deflection criteria for the deck components are established in accordance with S6-06 – section 9.4.2 [8]; they do not take into consideration the dynamic load allowance and limit deflection to L/400.

The overall deflection across the entire structure is limited to the values indicated in the Manuel de conception des structures (design handbook issued by Ministry of Transportation of Quebec). For our project, deflection is limited to L/1,000, because there is a walkway and the deflection assessed using the vibration frequency of the bridge is lower. Contrary to what is specified in section [8], we considered dynamic load allowances in assessing the overall deflection. Note that for a span of 43 metres, the maximum deflection is 43 mm for a maximum ratio of L/1,000 and 108 mm for a ratio of L/400, which we believe is too permissive, since we have a bituminous concrete running surface and we wanted to maintain a sealed deck.

In the end, the overall deflection achieved is 38 mm, which is close to the allowed values, and the deflection of the planking is also about L/1,000, i.e. better than L/400 specified in [8].

4.3. Design of the deck

As described earlier, the bridge deck is made up of glued-laminated boards (184x921) applied to the entire width of the bridge, i.e. 9,250 mm. The structural capacity was assessed using two different methods.

4.3.1. First assessment:

An overall size of 184x921 (flat) is used, and then the Vr and Mr values are calculated;

Using the cross-section of the bridge, we model the 184x921 size, resting on the main girders. The shear force and the bending moment are calculated using a model that simulates passage of a 87.5 kN wheel (maximum load of a #4 axle) in the deck traffic zone. The strain is approximately 61% for shear and 29% for bending;

The deflection of the piece is calculated at the most critical position and maximum deflection of L/400 must be met, in accordance with [8]. The deflection identified was less than 0.7 mm, while the allowable limit was 3.6 mm.

4.3.2. Second assessment:

Since the board is relatively wide (921 mm), we modelled the 184x921 board using finished pieces to verify the torsion effect on the timber when a wheel is located on the edge of the timber. The deflection thus obtained was approximately 1 millimetre.
The stresses in the wood were also below the allowed stresses (about 11 MPa).

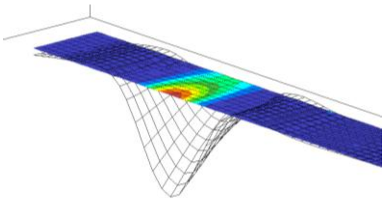

Figure 12: Warping and stress under wheel (250mm x 600mm)

4.4. Transverse warping of the timber

One of the peculiarities of wood is that it warps when there are changes in humidity. When moisture content changes (between manufacturing and installation or in winter and summer), the wood warps.

The moisture content during manufacturing is 12%. The tangential and radial shrinkage in the process used by Chantiers Chibougamau is 6% of the fibre saturation level (30% moisture) when oven dry (0% moisture).

The percentage change can be calculated as follows:

$$\Delta_{vt}\% = \frac{6\%}{30\%} \times (initial.moisture\% - 12\%) \qquad (1)$$

For instance, with wood moisture content of 14%, the change in volume compared with manufacturing (12%) is as follows:

$$\Delta_{vt}\% = 0.2 \times (14\% - 12\%) = 0.4\% \qquad (2)$$

For a timber that is 921 mm wide, its change in width would thus be 0.4%*921 = 3.7 mm.

For lengthwise shrinkage, the principle is the same, except that the change is much less significant:

$$\Delta_{vl}\% = 0.006667 \times (initial.moisture\% - 12\%) \qquad (3)$$

For a timber 9,250 mm long, the change in length is thus 0.01333%*9250 mm = 1.2 mm.

In the experience of Chantiers Chibougamau, the moisture content during manufacturing is 12%, during installation 13 to 14% depending on the weather conditions, and during the life of the bridge between 13% and 16%. Using this information and applying it to the components of our bridge, we calculated the changes as indicated in Table 4.

The change must be taken into account when designing the various construction details.

For installation of the boards (184x921), the gap between the boards during installation therefore depends on the moisture content of the timbers (or their size) and on the predicted change in volume. Table 7 indicates the gaps to use for installation, depending on the moisture content of the timbers or their size at installation.

Table 4: Changes in timber sizes depending on moisture

Point in time	Moisture content (%)	Dvt (%)	Width of timber (mm)
Starting moisture	12%		921
Installation moisture	14%	0.40%	925
Min. moisture (structure)	13%	0.20%	923
Max. moisture (structure)	16%	0.80%	928
Structure Max-Min			**5.5**

Note that if we install the timbers with no gaps during construction and the timbers have a moisture content of 13%, knowing that the moisture of the wood tends to go a maximum of 16%, the unrestrained expansion of the planking would be 960 mm. It is therefore wise to include gaps during installation to avoid any damage to the structure.

Table 5: Gap between timbers according to moisture content

Moisture content (%)	Width (mm)	Gap (mm)
12	921	9
13	923	7
14	925	5
15	927	3
16	928	2

4.5. Steel components

Calculation of the strength of the steel bolts and plates is in compliance with the specifications of [6].

4.6. Seismic bridge design

The bridge category is "Other" in [6], which is the least demanding, and the seismic zone is fairly low, i.e. A=0.036. The design of the foundations is thus based on R=1 and I=1. In addition, as all the support structures are fixed, the spreading of the seismic loads is optimal.

A multimodal spectral analysis was performed to determine the strain on the structure and foundations.

The ice loads are higher and dictate the design of the foundations. The support structures and pins are fixed, so they play an important role as a stabilizer against upheaval of the foundations due to ice ride-up.

4.7. Support structures

The support structures are made of conventional bonded steel reinforced elastomeric. The assembly is obviously different with a steel girder bridge. The vertical and transverse loads are lower, however, because the deck is about one-third the weight of a steel/concrete deck. In our case, because the pins take a sizeable amount of the deck strain, the reaction of the supports is about 10% of the reaction of a steel/concrete bridge.

4.8. Hyperstatic lifting

The arrangement of the straight girders and the arches means that lifting of the bridge requires greater force in relation to the reaction of the support structures. For example, on the most critical pier, the sum of the reactions due to permanent loads is 600 kN. However, to lift the bridge at this spot by 5 mm, we would need six lifting points of 160 kN, or a total of 960 kN. The lifting force is low and creates very little strain on the structure.

By comparison, the steel/concrete structural equivalent has a total lifting force of about 5,600 kN, more than five times the lifting value required for the wood bridge.

5. Construction

5.1. Foundations

The abutments and piers are reinforced concrete and rest on shallow foundations. The foundation soil is comprised of very rigid till.

The abutments are located on the shore of the Uupaachickus pass, whereas the foundations of the piers are underwater. The use of anti-leaching concrete was mandatory in the footings of the piers because of the water infiltration in cofferdams. The cofferdams were then dried and led to pour the central part of the pier using standard concrete.

Due to access difficulties in the west bank of the pass, the contractor had to use a few innovative solutions for pouring the concrete, such as placing the mixer trucks on barges or using a helicopter to fly the concrete over the pass.

A few issues with the concrete foundations were raised, most of all imprecision of the connections between the concrete and the wood. The anchors were offsets from 10 mm to 50 mm in every direction depending on the foundation unit, for example abutment 1 vertical anchors offset East whereas pier 2 vertical anchors offset North-West.

Even with the design that allowed some adjustments in the assembly (pre-drilled / drilled on-site holes, oblong holes in the anchoring plates of the arches) the contractor did not use these adjustments to his benefit. This led to a more complicated assembly.

5.2. Manufacture of the glulam girders

Because of the remote location of the project, in Northern Quebec, the design favoured the use of local materials. Chantier Chibougamau, located 90 km from the construction site, was awarded the contract for the wood structure, ensuring that the wood used on this project came from sustainable forests in the surrounding area.

The materials used in this project are Nordic Lam for glulam girders and panels, and Nordic X-lam for CLT panels. Information given by Nordic:

Nordic Lam is a glue-laminated structural timber product (glulam) made of black spruce and used as beams, headers, rafters, purlins, columns, studs and decking in buildings and other types of construction.

Product composition on the basis of 1m3 of glulam output at the mill gate:

- Wood portion: 1 m3 (417 kg on oven dry basis)
- Resin: 12.34 kg (Polyurethane and isocyanate)
- Lumber wrap: 0.46 kg (HDPE)

In order to ensure the best manufacturing quality, several actions were required by Chantier Chibougamau. The main quality control was supervision from a Dessau technician, experienced in wood manufacture. This technician was on site during the main steps of the manufacturing.

Dessau also required Chantier Chibougamau to make a full assembly of one of the main girders, including two arches and two strait girders. This test proved that the manufacture was correct and the assembly would not lead to problems during the final assembly on-site.

Overall the manufacture of the glulam girders and deck was a success except for the steel works, and the threaded rods in particular. These rods are the rods that transfer the efforts from arches to girders and girders to diaphragms. Some of these rods were not perfectly aligned, resulting in the necessity to enlarge the holes in the steel plate assemblies and girders. This was done on-site and led to delays in the assembly.

5.3. Assembly of the girders and deck

The erection of the girders started around pier #4. The first step of the contractor was to assemble two straight girders, on the ground, using diaphragms. The objective of this assembly is to have some lateral stability of the girders during the following steps. The assembly is then being mounted on the pier #4, using a crane. It is placed in the correct position using scaffolding and the arches are placed in the right position. The arches are first pinned to the pier and then brought to the correct position using cranes. The straights and arched girders are assembled using the rods passing through the diaphragms. The process is repeated until all the arches and straights girders are placed on pier #4, as shown in Figure 13.

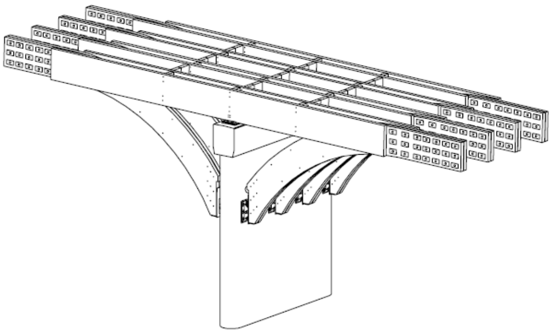

Figure 13: assembly completed at Pier #4

The contractor then began to assemble the girders between pier #4 and abutment #5, as shown in Figure 14.

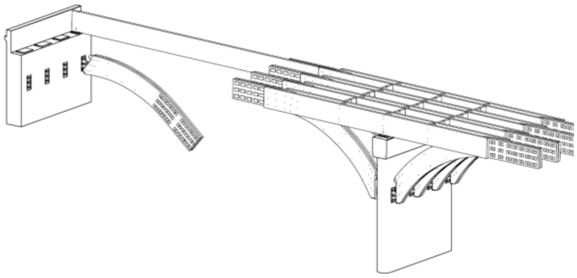

Figure 14: Beginning of assembly at abutment #5

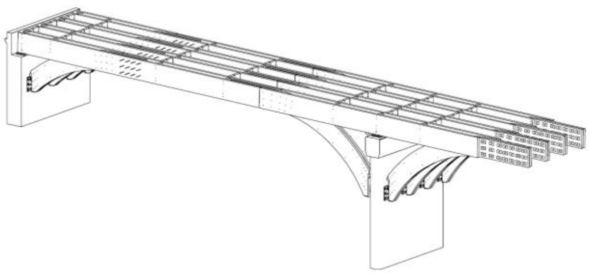

Figure 15: Assembly completed at abutment #5 and pier #4

The process is the same as the pier #4, pin the arched girders, and then assembling them with the straight ones. The process is repeated until all the arches and straights girders are placed.

The same process is repeated at pier #2 and abutment #1. The pier #3 is then attached to the 4-5 assembly before attached to the 1-2 assembly.

We had issues with the elevations of the girders. Being that a land surveyor was not on site, the contractor could not precisely position the girders. This led to differences from 20 to 50mm higher at mid-span of 2-3 and around 20mm lower between piers 3-4. In fact, the straight girder over pier #3, had rotated during the mounting. This problem has been corrected because it was caught early and Dessau gave a solution to the contractor. The steel plates, on which the pins are mounted, have oblong holes, permitting to replace them if needed. The plates were lifted from the left side of the #3 pier, and lowered on the right side. The position of the girders was then corrected.

There is still, 50mm lower point at mid-span of the pier #4 and abutment #5 spans. It was not possible to correct this deflection given that the girders were already assembled when the land surveyor gave the measures. To correct this deflection would mean dismount this span and then reassemble which is impossible at this stage of the construction. The road will be corrected around this low area, using more paving.

During the erection, the most important lesson we learned is that a specialized wood assembling contractor is the best solution if possible. Our contractor had no experience with assembling wood structures and did not seek help from a specialized contractor for this part of the project. It would have probably been a more effective solution for this complex structure. In our future projects, we will require the contractor, if possible, to use experienced workforce in wood structures.

Furthermore, we should have requested the presence of a land surveyor on site during the whole assembly.

5.4. End of construction works

All these small problems during the construction of the foundations and the assembly of the glulam structure, once added led to delays that were problematic. Due to the temperature when the works were finished, the deck membrane was not installed (need to be over 0°C), as well as the paving. This resulted in the inability for the client to use the bridge in the winter.

All the on-site works are suspended during the winter 2013/2014, due to weather conditions. All works have been completed in the spring/summer 2014.

A series of inspections will occur before on-site works resume in order assessing the potential damages due to water infiltration, because of the absence of the membrane.

6. Conclusions

The use of glulam gave us the opportunity to conceive a 160-metre-long bridge across Uupaachikus pass in Mistissini, with great aesthetical features. The cost of the bridge is affordable; it is even less than an equivalent steel-concrete bridge.

Figure 16: View of the bridge from east bank

7. Acknowledgement

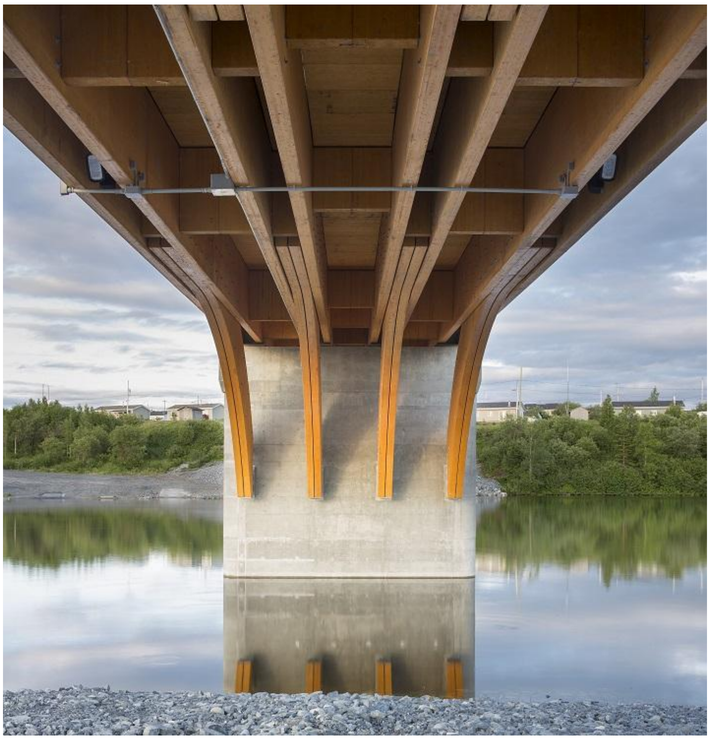

Figure 17: View of the structural system of the bridge, below the deck

First, we would like to acknowledge our client the Cree Nation of Mistissini, and in particular Emmett MacLeod, Director of Municipal Services and Jean Bénac, for their confidence in our capacity to conceive such an innovative project.

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Constructions BSL Inc:
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