Ontario Wood Bridge Reference Guide

By Moses Structural Engineers and Brown & Co. Engineering Ltd. for the Canadian Wood Council and the Ontario Ministry of Natural Resources and Forestry

CONTACT

FORWARD

Timber bridges have a long history of construction and use throughout North America, including Ontario, for roadways, railways and logging roads. The Canadian Highway Bridge Design Code (CHBDC), together with the Canadian Wood Council publication *Wood Highway Bridges* from 1992 are typically referenced by designers of timber bridges in Ontario. This new reference is intended to provide updated background information for designers as they embark on proposing and designing timber highway bridges for primary and secondary roads.

This reference is divided into three parts:

Part 1 – Wood Bridges – Design and Use

Part 2 – Opportunities & Current Limitations

Part 3 – Design Examples

Part 1 provides background information on topics including wood materials, bridge systems, prefabrication, durability and species availability. Details of costs, construction cycle and sustainability are also provided. Part 1 concludes with examples of a variety of completed highway bridges from North America and Europe.

Part 2 of this reference is intended to provide designers and authorities with highlights of the current edition of the CHBDC on subjects related to the wood highway bridges, including areas that will require future development in the code. Additional references to other resources for advancing practitioner knowledge of and advancing the state of the art in wood bridge design are provided.

Part 3 has two fully worked design examples of a two-lane 18-m span wood highway bridge designed in accordance with the latest provisions of the CHBDC and the best available information from current literature. Each example is based on a single-span, simply-supported glued-laminated girder bridge. One bridge has a glued-laminated deck and the other has a stress-laminated deck. These examples are intended to help designers understand the key issues as they undertake wood highway bridge design. Durability through detailing and choice of materials is discussed.

TABLE OF CONTENTS

DRAWINGS – WOOD BRIDGE DESIGN EXAMPLE (inserted at the back)

1.   Glued-laminated deck option drawings

     SWB-01 – General Arrangement

     SWB-05 – Timber Framing I

     SWB-06 – Timber Framing II

     SWB-07 – Timber Deck I

     SWB-08 – Timber Barrier I

     SWB-09 – Timber Barrier II

2.   Stress-laminated deck option drawings

     SWA-01 – General Arrangement

     SWA-05 – Timber Framing I

     SWA-06 – Timber Framing II

     SWA-07 – Stress-laminated Timber Deck I

     SWA-08 – Timber Barrier I

     SWA-09 – Timber Barrier II

Part One: Wood Bridges – Design and Use

1.1. INTRODUCTION

In many people's minds, the mention of timber bridges conjures images of covered bridges on country roads, or grand railway trestles and even some landmark bridges that are visually expressive. Timber bridges take many forms – in fact, the majority are quite innocuous, providing road access to remote locations for activities such as logging. Still, others are on highway systems throughout Canada and the United States, oftentimes unrecognizable since the timber structure lies beneath the paved roadway. Timber bridges remain an important part of our network of roads and highways.

The benefits of building modern timber bridges include (from Leslie, 2012):
1. Lower initial cost particularly for remote areas;
2. Speed of construction and use of prefabrication;
3. Use of sustainable materials;
4. Lighter foundations; and
5. Aesthetics.

Of the roughly 70,000 bridges on public roads in Canada it is unclear how many are timber. Data from the United States indicate that less than 2% of their bridges are timber. In the United States, new highway timber bridges continue to be built. Using a variety of types of glulam timber construction for spans as long as 85 m, timber bridges have been constructed and are viable options for current heavy truck loading requirements (Gilham 2013).

It is apparent that when designers and authorities have been familiar with the benefits of timber bridges, and are technically knowledgeable in the design of wood bridges, we tend to see more of this type of bridge built. Historically, the number of timber bridges in Canada has been encouraging. Currently, in Ontario there are 157 timber bridges and culverts in service with the Ministry of Transportation Ontario (MTO) (Mermigas, 2017) although the number may actually be as high as 250 timber bridges out of the 3000 bridges in Ontario (Krisciunas et al. 2010). British Columbia has approximately 500 timber bridges out of approximately 2800 and currently have more planned. Quebec has been building new timber bridges as well. According to Leslie Marshall "Nova Scotia has hundreds of timber bridges, and half of Prince Edward Island's bridges are timber" (Leslie M., 2012).

Wood bridges take on many forms, from the simplest log bridge to many types of trussed bridges – the style and design have evolved significantly over the years depending on technology, skill and availability of materials. "Covered Bridges and the Birth of American Engineering" (Christianson *et al.*, 2015) explores the designs of over eighteen different types of wood truss configurations used historically for wood bridges including the most basic king post trusses to more complicated town lattices and structurally efficient Pratt trusses. Truss bridges with the trusses above the deck (so called 'though-truss') provide a great opportunity to build a roof over the roadway. Trussed wood bridges from the 18th and 19th centuries had spans in the range of 30 m to 60 m (100 to 200 feet). The presence of the roofs is the main reason these bridges still exist today – the roof protects the wood structure beneath it from rain, snow and sun.

West Montrose Bridge, Ontario, D. Moses

Trussed bridges allow for longer spans compared to simple girder bridges. Many examples of trussed timber bridges for roadways have been built for well over a century. For those with no roofs, the structure below the deck is relatively protected from weathering, however the timber members and connections above deck require attention to detail to provide drainage and localized protection of the structure.

As described in detail in Timber Bridges Design, Construction, Inspection, and Maintenance (Ritter, 1990), the Howe truss was patented in the United States in 1840 based on stress analysis which was a major leap in timber bridge technology because it was based on more than 'trial-and-error' and introduced cast iron components to the design (chords and diagonals were timber and verticals were cast iron). Shortly after, the Pratt truss was patented, also using cast iron members. In Ontario, the Sioux Narrows Bridge, constructed in 1936 remained in service for almost 70 years. It is a box Howe truss with a 64 m main span built from solid sawn Douglas fir timbers.

As time passed, cast iron bridges became popular with less and less timber being constructed. Initially, timber bridges were about half the cost of cast iron bridges, but by the mid-1930's, steel bridges could be built economically then reinforced concrete became more and more common for bridges. In the mid-twentieth century, most efforts into research and advancement of bridge concepts focussed on steel and concrete construction.

In contrast to the mainstream, Ontario continued to build wood bridges, particularly in the northern parts of the province. Research activities continued at the Ontario Ministry of Transportation and Canadian universities where they pressed ahead with novel timber bridge designs during the 1970's to 1990's. Research using new engineered wood products such as glulam timber, parallel strand lumber (PSL), glass fibre reinforcing, high performance concrete and epoxies and even composite concrete-log bridges continued to evolve (Krisciunas *et al.* 2010). Research continues, though at a lesser pace, in Canada.

Norwegian Bridge with member protection. Photo Credit: FPInnovations

1.1.1. SCOPE

Part 1 of this reference guide provides background on many topics related to wood bridges including:

- Types of wood bridge systems
- Wood construction technology
- Wood bridge design considerations (national and provincial requirements)
- Prefabrication
- Service life and durability
- Benefits (cost, construction cycle and sustainability)

The topics are followed by examples of wood bridges from Canada and around the world.

1.2. WOOD BRIDGE SYSTEMS AND TECHNOLOGY

The following sections describe the various wood products and systems available, treatments, fasteners and hybrid systems.

1.2.1. WOOD BRIDGE MATERIALS

Solid Sawn
Solid sawn members can be either dimensional lumber (sizes ranging from 38 mm x 89 mm to 89 mm x 286 mm) used for deck laminations, or heavy timbers (ranging from 140 mm x 140 mm to 191 mm x 191 mm) for girders or floors beams.

Glued-laminated timber (glulam)
Glulam is typically made of dimensional lumber (38 mm thick) and glued together to form deep members (See Figure 1.2.1.1.) that are more economical than using a large solid sawn lumber. Glulam is commonly used for stringers or girders, but can also be placed on its side to make a glulam deck or concrete-glulam composite deck. Glulam can also be curved and for standard glulam laminations they can be curved to a radius as small as 8.4 m – smaller radii can be achieved with thinner (non-standard) laminations (designers should be aware that not all suppliers can supply thinner laminations and that it will be more expensive).

Figure 1.2.1.1. Glulam Member (Source: Figure 31L, section 4.1, FPI/TWBC)

Laminated veneer lumber (LVL)

LVL is manufactured from veneers all oriented in one direction. LVL is commonly produced in 44 mm wide sections. LVL is approved for dry use only (i.e. for outdoor conditions it cannot be exposed and must be adequately protected). In wood bridge applications, LVL is usually used for stressed T- or box-sections (see Section 2.2) but must be carefully protected from moisture.

Parallel Strand Lumber (PSL)

Parallel strand lumber is a structural composite lumber (SCL) developed in Canada in the 1980's and became commercially available in 1990. PSL is made from the same veneers as LVL, but the veneers are sliced into thin long strands (approximately 3 mm x 12 mm x 1000 mm long) prior to pressing. The strands are pressed together with a resin. PSL can be treated but is normally covered by a protective wearing surface. PSL can be built into in T- or box-sections.

Composites

The most common composite material used for wood bridges is a concrete-wood laminated deck. The two materials are mechanically bonded to act as one member. See Section 1.2.2 for more detail.

1.2.2. WOOD BRIDGE SYSTEMS

The following sub-sections describe the various bridge systems used in wood bridges as they pertain to decks, the super structure, and the sub-structure.

1.2.2.1. DECKS

Decks are a crucial component of bridges, not only for structural strength, but also for protection of the bridge substructure. For information on protection and treatment of decks, see Section 1.4. There are many different types of decks used for wood bridges, as follows:

Longitudinal nail-laminated (LNL) Decks
Longitudinal nail-laminated decks are built from dimensional lumber laid side-by-side and nailed together to form a wood deck (or slab). Wood laminations span in the same direction as the flow of traffic. Lumber used for the deck ranges from 38 to 89 mm in thickness by 89 to 286 mm in depth. Most commonly available lengths are up to 4.9 m. This is also applicable for TNL, LSL, TSL, and WCC decks (described below).

The use of longitudinal nail-laminated decks in Canada is limited due to the common occurrence of deck delamination. Deck delamination can occur in LNL decks from some planks receiving more load than adjacent planks directly under wheel loads. These planks will deflect more under the higher concentrated load, causing the nails to bear more heavily and partially withdraw, effectively loosening the deck and delaminating the planks.

Longitudinal nail-laminated decks span along the length of the bridge, in the direction of traffic flow and between supports.

Transverse nail-laminated (TNL) decks
Transverse nail-laminated (TNL) decks are similiar to longitudinal nail-laminated (LNL) decks but span across the width of the bridge. TNL decks are also built from dimensional lumber laid side-by-side and nailed together to form the wood deck (or slab). Wood laminations span perpendicular to the flow of traffic. TNL decks are more common than LNL decks since they are less susceptible to delamination. Unlike an LNL deck, the TNL deck requires support from girders or stringers. Figure 1.2.1.2. illustrates the difference between LNL and TNL decks.

Figure 2.2
Longitudinal and transverse laminated decks

Longitudinal laminated deck

Traffic direction

Transverse laminated deck

Traffic direction
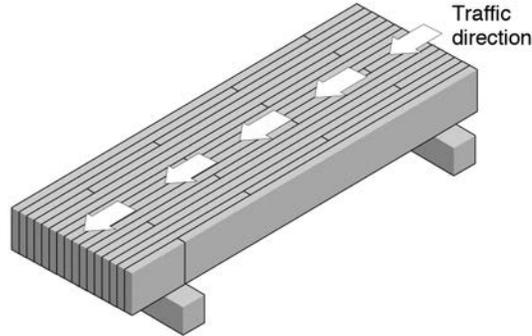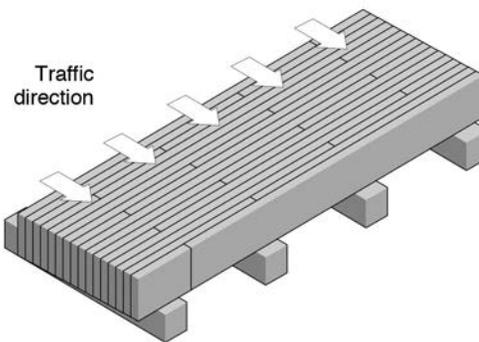
Figure 1.2.1.2. Longitudinal and transverse laminated decks
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1992))

Longitudinal stress-laminated (LSL) decks

Longitudinal stress-laminated decks are built from dimensional lumber laid side-by-side and nailed together to form a wood deck. In addition to the nailing, post-tensioned steel bars are installed through the deck. The post-tensioning helps to reduces plank delamination and improves load sharing. Holes are pre-drilled through the wood planks and pressure treated after drilling for improved durability. LSL deck are more common than TSL decks (See Figure 1.2.1.3.).
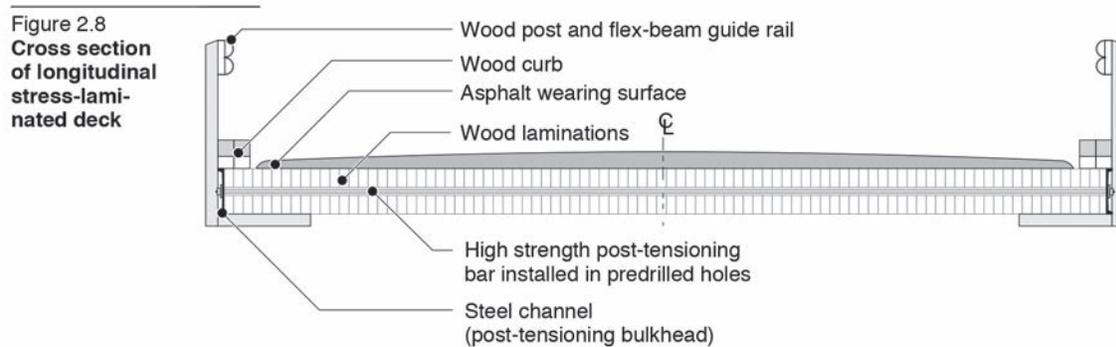
Figure 1.2.1.3. Stress-laminated deck
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

Transverse stress-laminated (TSL) decks

Transverse stress-laminated decks are built from dimensional lumber laid side-by-side and nailed together to form a wood deck. In addition to the nailing, post-tensioned steel bars are installed through the deck. Very few wood bridge decks use TSL decks. TSL is similiar to the LSL deck, but the post-tensioning bars are positioned in the same direction as the flow of traffic. This is problematic as it means the anchorage point is located under the road (higher chances of contact with salt and water) making it difficult to access for re-tightening and maintenance of the bars.

Wood-concrete composite (WCC) decks

Wood-concrete composite decks are built from dimensional lumber laid side-by-side and nailed together to form a wood deck. The laminations can be either longitudinal or stress-laminated. The concrete is mechanically bonded to the laminated wood deck, to create a composite slab. (See Figure 1.2.1.4. below). The concrete provides a buffer between the vehicle wheels and wood planks, and provides better load distribution to the wood planks (significantly reducing the likelihood of plank delamination). While the wood laminations are not affected by road salts, the concrete portion is susceptible to salts and must be designed and protected appropriately. Detailing between the concrete and wood is also required to ensure that water does not get trapped between the two material interfaces.
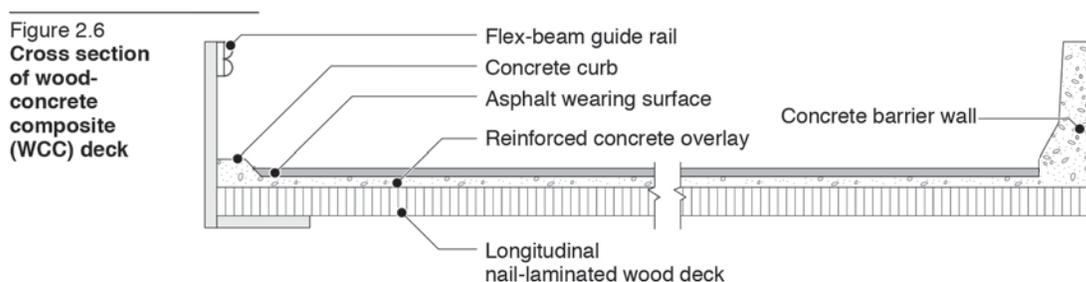
Figure 1.2.1.4. Wood-concrete composite deck
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

Note that other composite concrete-wood deck systems have been developed in other countries, typically using a connector between the wood and concrete for shear transfer with consideration of thermal and swelling changes in the materials.

Floor beam decks

The most common form of construction for floor beam decks are solid sawn timbers. The system consists of a plank surface on heavy timbers oriented transversely and supported on longitudinal wood or steel girders (See Figure 1.2.1.5. below). The decks are most commonly found in park and forest roads and municipal roads. Floor beam bridges can support heavy loads, but the planks may need replacing every few years. Typical floor beams are 184 x 184 mm or 200 x 250 mm (depending on girder spacing). Typical planks sizes are 64 x 184 mm up to 100 x 250 mm.

LONGITUDINAL PLANKS

TRANSVERSE FLOOR BEAM

LONGITUDINAL GIRDER
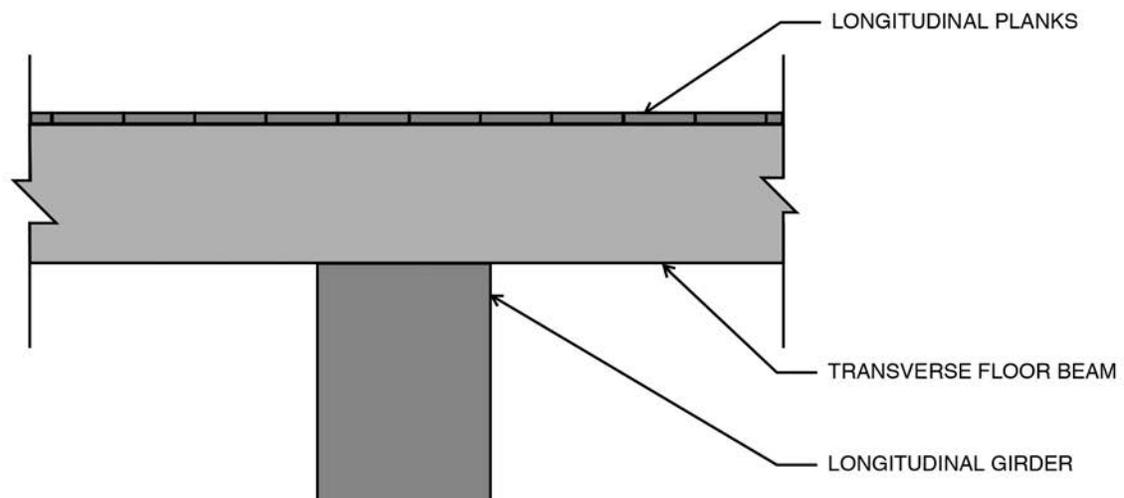
Figure 1.2.1.5. Floor beam deck (Moses Structural Engineers Inc.)

Two-layer plank decks

The most common form of construction of two-layer plank decks are solid sawn timbers. This system consists of two alternating layers of planks supported on floor beams. The two layers help to provide a more dimensionally stable deck and better load distribution (See Figure 1.2.1.6.).
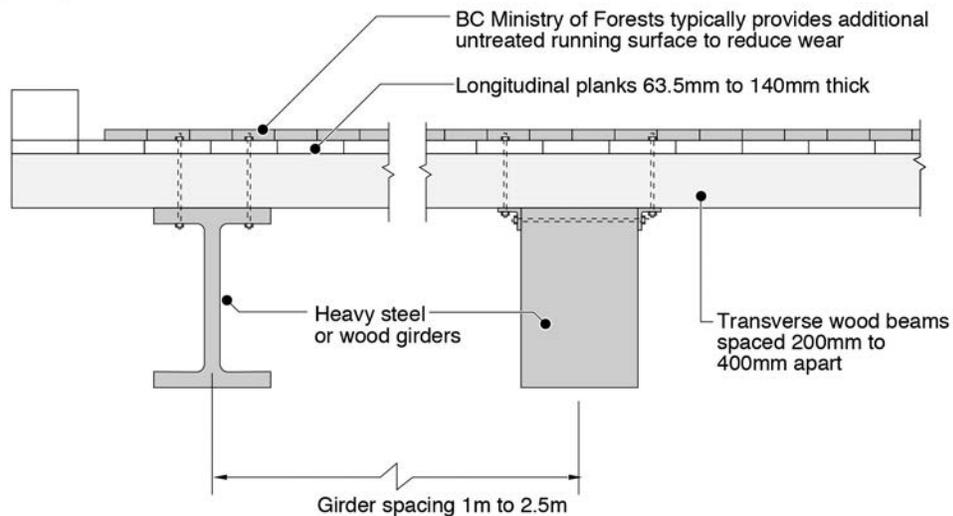
Figure 1.2.1.6. Two-layer plank deck
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

1.2.2.2. SUPER STRUCTURES

Log beams

Perhaps the oldest method of forming a bridge span, modern log beam bridges are constructed by placing round logs alternating from base to top (i.e. if the base of the log is wider than the top, to keep a uniform width the base and tops are alternated) and binding them together with steel cables. The deck, if solid sawn dimensional lumber, is fastened to the logs, or alternatively soil and rocks can be placed on the logs. Log beams commonly reach spans of 6 m to 18 m (this is primarily limited by the availability of diameter and length of the trees), however spans up to 30 m have been constructed. The logs typically are not pressure treated and have generally served as temporary structures with a 10-20 service life.

Stressed log bridges

Stressed log bridges became a solution for using discarded wood poles and efficient wood use (the concept can also be used for new poles). The round logs are cut along two parallel faces to create a flat surface. They are pressed together against their flat faces and then are "tied" together with a post-tensioning system (See Figure 1.2.2.1).

The post-tensioning is similar to stress laminated decks, but with fundamental differences. Stress laminated decks use nails to connect the laminations together, but this cannot be accomplished with stressed log bridges due to the large size of the logs. This results in a higher likelihood of experiencing more prestress loss.

A method to reduce the amount of prestress loss is to use a more flexible material than traditional prestressing steel. One solution to this has been to use Aramid fibre which has a tensile capacity of about 1.25 times of prestressing steel and about 40% of the modulus of elasticity. Aramid fibre is available in different sizes under the trade name PARAFIL rope, which has flexibility similar to rope and can be wrapped around small radii. Other synthetic fibres exist but are often composite with epoxy and can be too stiff to wrap around small radii. A note of caution that PARAFIL rope should be insulated when hot-mix asphalt is used, since it is not recommended for temperatures 136°C or higher.
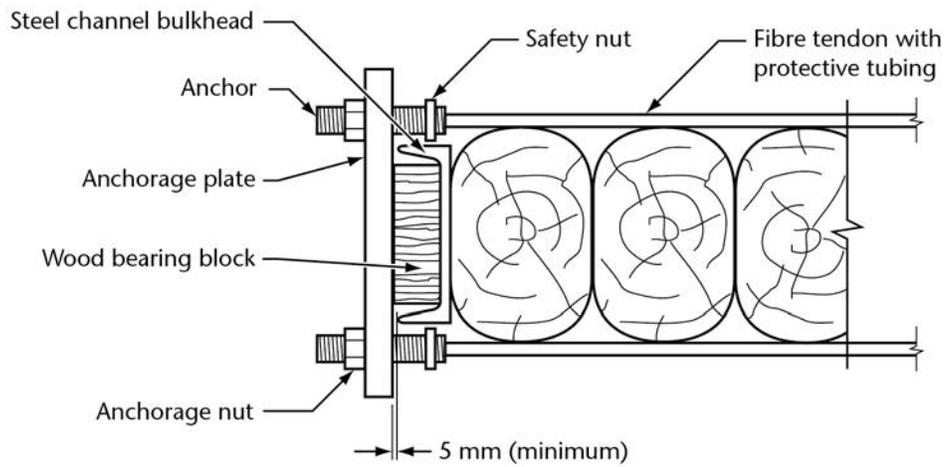
Figure 16.9
Post-tensioning system for stressed log bridges
(See Clause 16.9.3.)

Figure 1.2.2.1. Stress log cross-section
(Source: Clause 16.9.3., Figure 16.9. Post-tensioning system for stressed log bridges –
Canadian Highway Bridge Design Code. © 2017 Canadian Standards Association)

Trusses

Timber trusses, shown in Figure 1.2.2.2., can achieve longer clear spans for timber bridges. Trusses can support the bridge deck at various elevations (e.g. top chord or through truss, bottom chord or deck truss, or somewhere in between – pony truss). Truss members using either glulam timber or solid sawn timbers can be built in a wide variety of styles. The Sioux Narrows Bridge is one historical example of a solid sawn truss bridge in Ontario (refer to Section 1.6.1 for more detail). There are also other examples of truss bridges that can be found in Section 1.6.4. Figure 1.2.2.2. illustrates different truss configurations.
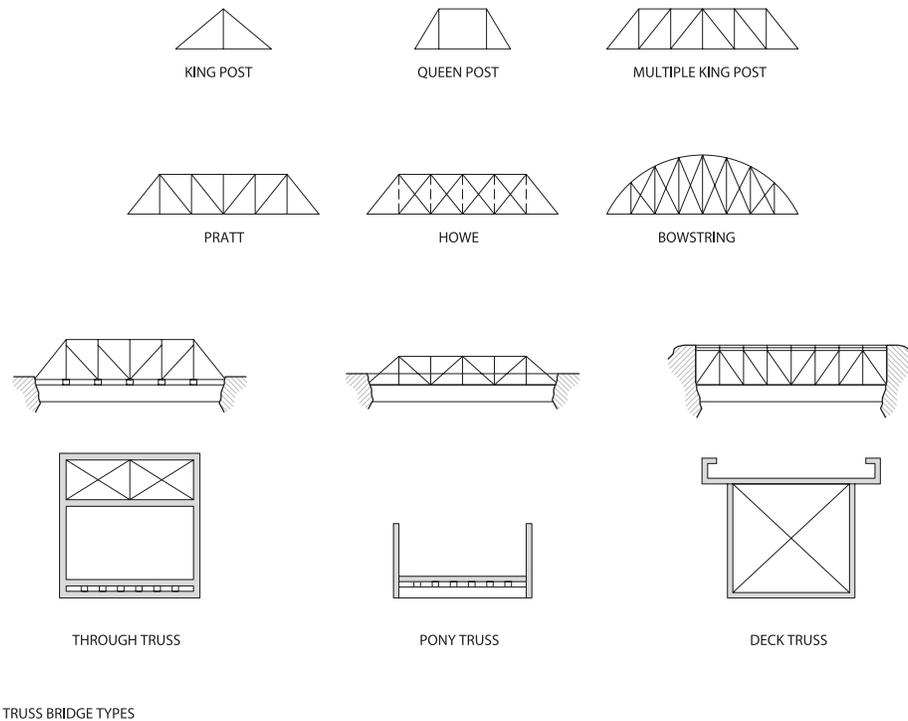
Figure 1.2.2.2. Truss Configurations

Bowstring Truss

Bowstring trusses have a continuous curved glulam timber top chord or a series of straight segmented solid sawn timber members that form the top chord. The bottom chord is normally straight and web members are vertical and diagonal. Bowstring trusses tend to be most economical for spans up to 30 m.

Parallel-chord Truss

Parallel-chord trusses can be constructed as through-trusses or deck trusses and can reach spans of up to 75 m. Parallel-chord trusses are an economical solution where vertical clearance is not constrained.

Trestles

A trestle bridge is a system that consists of simply-supported beams, decks or trusses that are supported on repeating timber bents. The bents are made of timber piles or solid sawn frames. Trestles are best situated when long clear spans are not required. Trestles are not commonly used currently due to their higher cost associated with more material being used, and since longer spans can be achieved using other systems. Additionally, in locations over waterways, if the bridge bents are too intrusive, a trestle will not be a viable solution.

Suspension Bridges

Suspension bridges most commonly are constructed with a timber deck supported from flexible steel cables that are supported by timber towers. These types of bridges can reach clear spans over 150 m.

Multi-beam Solid Sawn Stringer and Glulam Girders

Solid sawn timber stringers are available in up to 12 m lengths and are therefore typically used for simply-supported spans (See Figure 1.2.2.3.). Stringer spacing can reach up to 750 mm. The span of solid sawn timbers is limited by the availability of large timbers and capacity since allowable design stresses are reduced for large dimensional solid sawn timbers. Larger solid sawn members will also require adequate drying prior to installation.

Glulam timber girders offer a variety of much larger member sizes and are commonly used for spans between 10 m and 25 m (50 m spans have been reached). With proper treatment, glulam girders can provide a service life of over 50 years. During manufacturing, cambers can also be incorporated to help counter deflections from service loads and creep effects. Glulam girder spacing can reach up to 1.8 m (See Figure 1.2.2.4.). More information on glulam stringers and girders can be found here: http://www.woodcenter.org/library/videolibrary.cfm

As with all bridge types, protection of the substructure for durability requires careful attention to details for preventing water accumulation. Overhanging the deck is one approach to protect the girders and stringers from direct rain impact. Refer to Section 1.4.0 for more information on durability and protection details.

Figure 3.1
Cross section of a sawn wood stringer bridge

Wood post and flex-beam guide rail
Wood curbs
Asphalt wearing surface
Transverse laminated wood deck
Sawn wood stringers
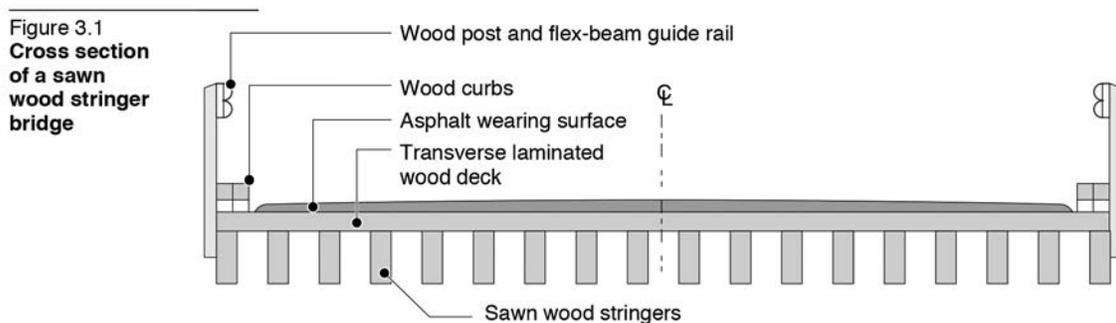
Figure 1.2.2.3. Wood stringer bridge
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))
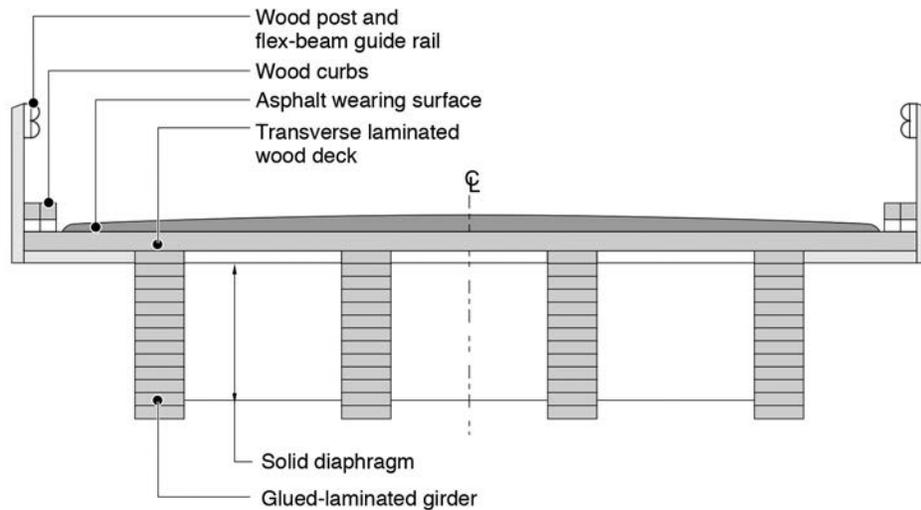
Figure 1.2.2.4. Glulam girder bridge
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

Stressed T-and Box-sections

A stressed T-beam deck consists of a longitudinally stress laminated deck with a deep LVL laminates or glulam timber members spaced every six to eight deck laminations (See Figure 1.2.2.5.)

A stressed box-section consists of an upper and lower longitudinally stress laminated deck with a deep LVL lamination or glulam member spaced every six to eight laminations, connecting the two decks (See Figure 1.2.2.6.). The designer should be cautioned when designing with LVL as it is only approved for dry conditions.
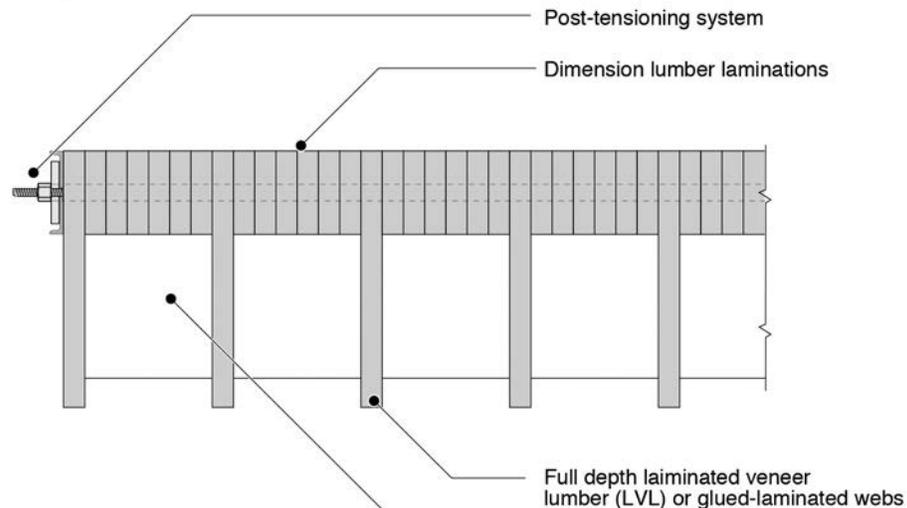
Figure 1.2.2.5. LVL T-beam
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

Figure 8.5
Cellular integrated deck

Spacer deck elements are regular dimension lumber
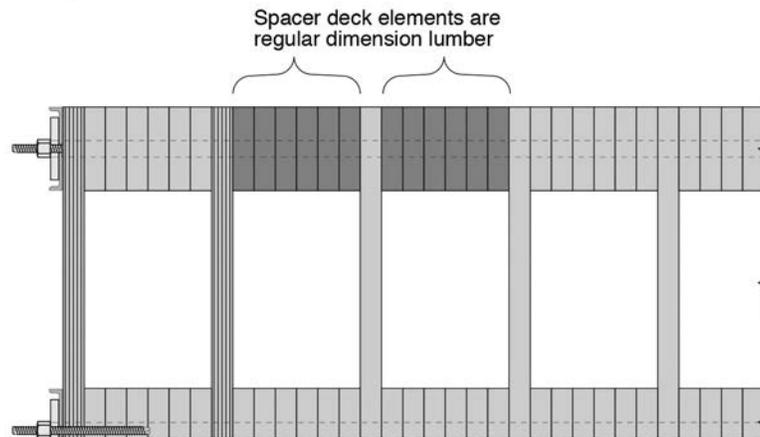
Figure 1.2.2.6. LVL box-bridge
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

Glulam Arches

Glulam arches can be used to achieve larger clear spans. Arches can support the bridge deck from below or by suspension using rods and transverse girders below the deck. The arch can either be two- or three-hinged. A two-hinge requires a continuous arch from end to end (or with moment splices at intermediate points). A three-hinge arch has a hinge at the mid-span of the arch. For examples (See Figure 1.2.2.7. below and Sections 1.6.2, 1.6.3, and 1.6.4).

MOMENT SPLICE
(IF REQUIRED DUE TO FABRICATION AND TRANSPORTATION RESTRAINTS)

HINGE

TWO-HINGE ARCH

HINGES
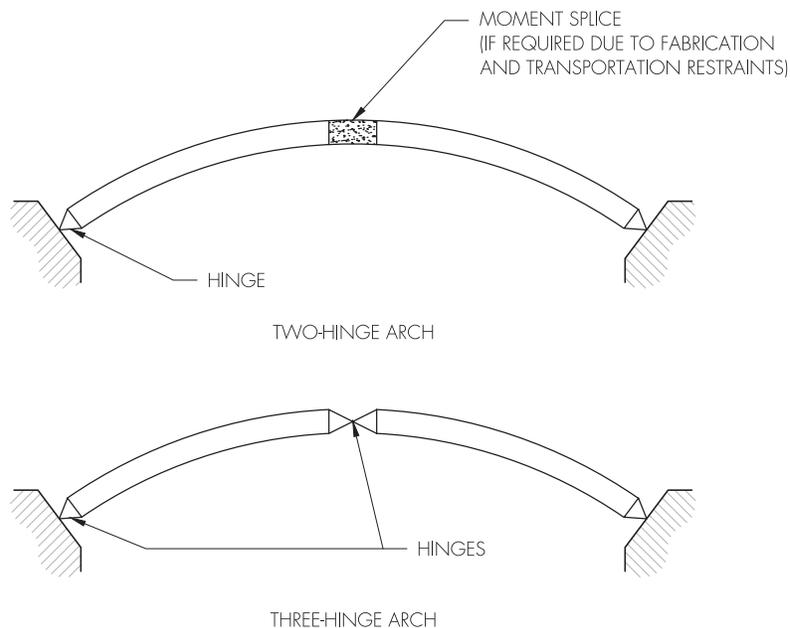
THREE-HINGE ARCH

Figure 1.2.2.7. Two and three-hinge arch bridges

1.2.2.3. SUB-STRUCTURES

Abutments

Most common abutments used today are concrete abutments, but there have been bridges built with timber abutments. Abutments are used to support the ends of bridges and retain embankment soils. One type of timber abutment is a pressure treated solid sawn or pressure treated glulam timber spread footing that is placed on the embankment. This can only be used in situations where the foundation is of high quality (low settlement, little erosion and scour). Alternatively, post abutments, built from a series of posts (wood or other materials), support the superstructure and are connected to a spread footing and provide a backwall and wingwall for retaining the embankment. If soil conditions cannot support post abutments, then timber piles can be used.

Bents

Bents are used for intermediate support between abutments. For relatively short bents, timber piles can be used for the bent structure (the height is generally governed by the available length of the timber). Timber piles penetrate the soil. For heights beyond those that can be achieved with timber piles, timber frames can be built. Frames can also incorporate braces to provide lateral stability. Timber frames can be supported on either footings or piles.

1.2.2.4. OTHER SYSTEMS

In addition to the systems and materials described above, there are other systems that can be considered for wood bridges. Fibre-reinforced polymer (FRP) can be used as an alternative to stressed wood decks or a method to repair existing decks for improved strength. CAN/CSA S6 – Canadian Highway Bridge Design Code currently permits the use of aramid-fibre-reinforced polymer (AFRP), glass-fibre-reinforced polymer (GFRP), and aramid ropes for stressed wood deck design.

Cross-laminated timber (CLT) or composite concrete-CLT can be an alternative to deck design. CLT an engineered wood product that is assembled by alternating the direction of dimensional lumber in a series of layers (top and bottom layers have the same orientation). Alternating the layers provides improved dimensional stability. However, CLT is not recommended for exterior use because the wood used is not treated. The use in wood bridge design would require proper detailing and protection from water ingress. For Canadian designers there is the added challenge that CLT is currently not recognized in CAN/CSA S6 – Canadian Highway Bridge Design Code (CHBDC).

1.3. WOOD BRIDGE DESIGN CONSIDERATIONS

Design considerations for wood bridges in Canada include the design standards and guidelines, methods of construction and issues related to durability, as discussed in this section.

1.3.1. CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC)

CAN/CSA S6 Canadian Highway Bridge Design Code (CHBDC) outlines the criteria for all bridges designed in Canada. CAN/CSA S6 is based on limit states design. A review of sections of the CHBDC related specifically to wood bridges can be found in Part Two of this document.

1.3.2. MTO ONTARIO PROVINCIAL STANDARD SPECIFICATION

The MTO Ontario Provincial Standard Specification (OPSS) is an 8-volume set of standards for roads and public works in Ontario. Design and construction of bridges in Ontario must meet the requirements of the OPSS and CAN/CSA S6. Specifications related to materials and construction are listed in "Volume 1 – Municipal and Provincial Common General and Construction Specifications" in Section 907 "Construction Specification for Structural Wood Systems."

1.3.3. ONTARIO HERITAGE BRIDGE GUIDELINES

The Ontario Heritage Bridge Guidelines (OHBG) provide direction for heritage road bridges owned by the province. OHBG does not govern over CHBDC however, Article 1.4.2.8 of the CHBDC states that the appearance of the finished structure must be considered with its surroundings. The OHBG requires an environmental assessment be completed before any work is done on a heritage bridge.

Not all bridges are heritage bridges. The MTO uses a scoring system based on criteria to determine if a bridge is a provincial heritage bridge. The OHBG lists provincial heritage bridges, two of which are timber bridges: the Sioux Narrows bridge and the Duchesnay Creek bridge. If a bridge is not found on this list, at least one of the following criteria needs to be met for a Cultural Heritage Evaluation Report to begin:

1. The structure is listed in the document: *Heritage Bridges: Identification and Assessment Guide, Ontario 1945-1965* (List A, B or C) as a candidate structure,
2. The structure is over 40 years old and is not screened in the 1945-1965 Guide (mentioned above), or
3. The structure is locally or regionally unusual.

Whether or not a bridge is a heritage bridge, the design of rehabilitation or replacement of the bridge must take into consideration factors such as structural adequacy. The OHBG outlines eight conservation options required for heritage bridge rehabilitation:

1. Retention of existing bridge with no major modifications undertaken;
2. Restoration of missing or deteriorated elements where physical or documentary evidence (e.g. photographs or drawings) exists for their design;
3. Retention of existing bridge with sympathetic modification;
4. Retention of existing bridge with sympathetically designed new structure in proximity;
5. Retention of existing bridge no longer in use for vehicular purposes but adapted for a new use. For example, prohibiting vehicle or restricting truck traffic or adapting for pedestrian walkways, cycle paths, scenic viewing, etc.;
6. Retention of bridge as a heritage monument for viewing purposes only;
7. Relocation of smaller, lighter single span bridges to an appropriate new site for continued use or adaptive re-use; and
8. Bridge removal and replacement with a sympathetically designed structure
   a. Where possible, salvage elements/members of bridge for incorporation into new structure or for future conservation work or displays
   b. Undertake full recording and documentation of existing structure.

1.3.4. PREFABRICATION

Prefabrication offers several positive attributes for timber bridge construction.

- Less time spent on site – faster installation using pre-assembled components
- Pressure treatment with preservatives in advance of installation but after holes are drilled and notches are cut – this is a more durable product compared to site applied topical preservative treatment of site drilled holes and site cut notches
- High precision of offsite fabrication resulting in better tolerance control
- Ability to create complex geometry (or slight variations in members for horizontal curves or bridge slopes)

Design considerations for prefabrication

- Transportation size limits (may need to design moment connections or other member splices)
- Preservative treatment facility size limits (may need to design moment connections or other member splices)
- Availability of materials and species

1.3.5. PRESTRESSING

The stress-laminated decks discussed earlier use the technique of prestressing wood elements. This method was developed in Ontario in the 1970's and came into new construction practice in the 1980's. Wood elements are post-tensioned after the deck is assembled. The post-tensioning is provided by high-strength steel bars that are installed through predrilled holes through the wood element (See Figure 1.3.5.1.). The bars are anchored at the ends with a steel anchorage plate that typically bear against a steel channel. The bars can also be located above and below the wood elements with a bearing block at the edge of the wood element (See Figure 1.3.5.2.).

Prestressing a wood deck will provide better stability of the deck because the wood elements are pressed tightly together which has the effect of better load sharing and can increase the moment capacity of a deck. When combined with a concrete composite slab, these decks can reach spans up to 30 m.

Due to long term shrinkage and creep effects in the wood, there can be a loss of prestress with time. This can be resolved by re-tightening the bars. CHBDC Clause 9.23.3.4 requires that initial tightening, at the time of construction shall have two stressing operations at least 12 hours apart, followed by restressing at least two weeks after the initial stressing, and then another restressing at least four weeks after the first restressing. At no point between any of the restressing cycles shall the ambient temperature reach below 0°C.
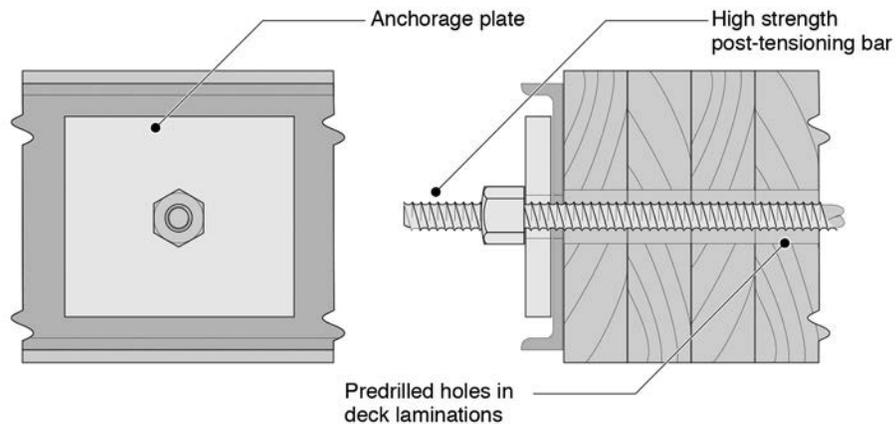
Figure 1.3.5.1. Prestressing anchorage detail through the wood deck
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))
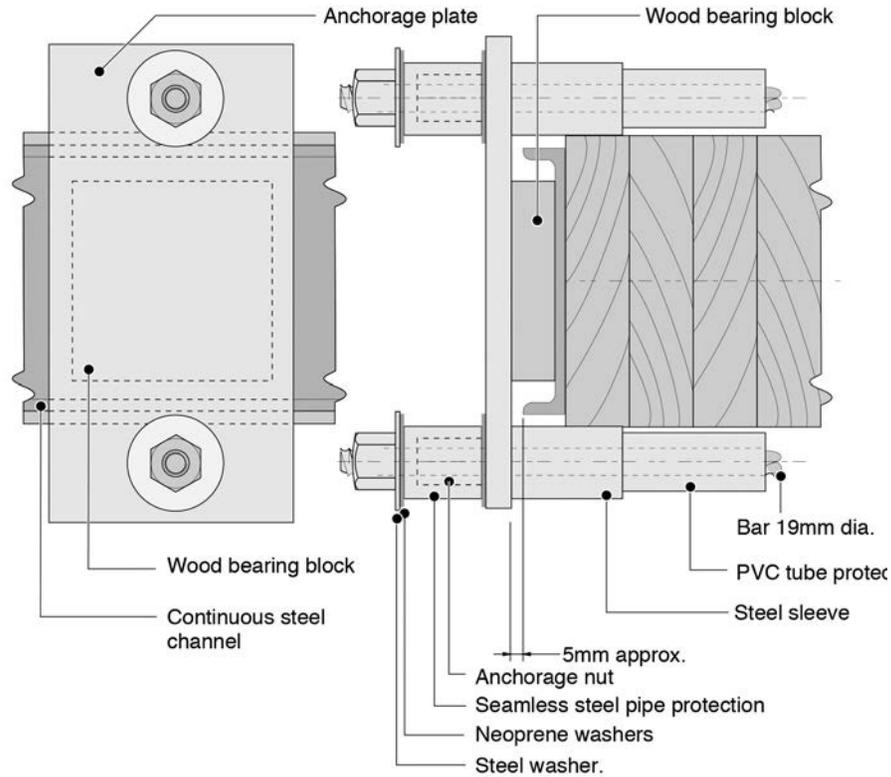
Figure 1.3.5.1. Prestressing anchorage detail outside of the wood deck
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

1.3.6. SERVICE LIFE AND DURABILITY

The required service life for any bridge in Canada is 75 years. This does not mean it is maintenance-free for 75 years, but that the main structure can withstand 75 years or more of loading. It should be expected that repairs and maintenance will be more frequent than this. For example, an asphalt wearing layer on the bridge deck may need to be replaced every 15 or 20 years. Regardless of material, it is not the structure that governs the service life, rather it is the detailing for a durable design that governs the service life of a bridge. Three main criteria that address durability in bridges are protection by design, pressure treatment, and replaceable elements. These criteria are further explained in Section 1.4.

1.3.7. SPECIES

Two main wood species used for wood bridge design in Canada are Douglas Fir (D.Fir) and Spruce-Pine-Fir (SPF). Other species like Hem-Fir and Northern are also recognized as structural wood species in CSA O86, however, they are less commonly used. The same is true for glulam members. The focus here will be on D.Fir and SPF materials, comparing their attributes. These are summarized in Table 3.7.1 below.

Table 3.7.1. Comparison of D.Fir and SPF wood species in bridge design

Category	D.Fir	SPF
Code recognized	D.Fir is recognized in the CHBDC and CSA O86 for both glulam and solid sawn timbers.	CHBDC does not currently recognize SPF for glulam, although it is recognized in CSA O86. SPF is recognized for solid sawn timber in the CHBDC and CSA O86.
Pressure treatment*	Unsuitable for some pressure treatments.	Suitable for many pressure treatments.
Structural properties	Compared to SPF, D.Fir (solid sawn and glulam) in general has higher moment and shear capacities and higher stiffness.	SPF has lower capacities and stiffness than D.Fir
Availability	Most commonly available in British Columbia.	Most commonly available in Ontario and other eastern provinces.
Appearance	Darker in appearance than most softwoods, but without treatment it will "grey" with time.	Light in appearance, but without treatment it will "grey" with time.

*There are some wood species such as cedar that do not require pressure treatments for durability. However, these naturally durable species are not recognized in the CHBDC.

1.3.8. INTERNATIONAL RECENT PRACTICES

Norway commonly uses stress-laminated glulam decks and glulam arches or trusses which can reach spans of up to 70 m (refer to Section 1.6.0 for example bridges). It is also common practice in Norway to "dual" treat the glulam members, typically first with a copper-based preservative, and then impregnation with creosote. They will also provide copper flashings. This results in a highly durable and long-lasting bridge. See Subsection 1.6.4 for more examples of international wood bridges.

1.4. WOOD BRIDGE PROTECTION

Moisture from rain and snow is one source of increased moisture content in timber components. Tree leaves and branches can also trap moisture against the timbers, which can lead to decay. Similarly, birds and other animals can leave droppings which can work in a similar way to trap moisture and promote decay.

Protection by design

A bridge can be designed such that it is inherently self-protecting by deflecting water away from the structural elements. Covered road bridges were built throughout Canada and the United States for this reason as shown in Figure 1.4.1. Installing a roof overhead is an excellent way to shed water and snow away from the main bridge structure. The roof will require occasional replacement, just like a building, but this is a minor cost and disruption compared to repairing the main bridge structure. The roof slows the movement of moisture to the structural elements and allows the timber to dry out between wetting cycles. The roof also prevents organic materials such as leaves and branches from building up on the timber components where they would trap moisture.

Figure 1.4.1. Timber Pedestrian Bridge in Guelph, Ontario (Photo Credit: E. Chau)

Another strategy is to shed water by sloping the members using arches or skewed elements. Figure 1.4.2. is an example of a wood bridge with all structural members sloped. While this may require more forethought in the structural layout, it prevents the accumulation of moisture on the timbers, thereby allowing them to dry out after rainfall or snowfall events. CHBDC Clause 2.3.2.3 specifies, in general terms, for all materials, that detailing shall provide free air circulation and avoid pockets and crevices. However, no details specific to timber bridges are provided. In Europe, good practice in timber bridge design is to use techniques that keep the timber dry and allow it to dry if it does get wet.

Figure 1.4.2. McCulloch Trestle foot bridge with all members sloped. (Photo Credit: D. Moses)

Another method of protecting the timbers is to create overhangs using the bridge deck. See Section 1.6 for examples of bridges with deck overhangs. Since the deck is likely to be covered with a wearing surface layer of concrete or asphalt with a membrane that protects the timber below it, the deck itself can be used to prevent water contact with the supporting structure below. Reliance on this method requires ongoing maintenance and inspection of the deck (from above and below) to ensure that there are no leaks. And the designer must consider potential pathways for water that short-circuit this protection, such as the attachment of guard rails that pass through the protective membrane, or unprotected sidewalk wearing surfaces.

The use of metal flashings is another method of preventing direct contact of water with timbers. This is not always practical and does add to the initial cost and maintenance costs, however, if designed and installed correctly, flashings will help to extend the life-expectancy of timbers. Critical to the installation of flashings is a connection design for the flashing to the timber that prevents moisture from being trapped beneath the flashing and allow the timber to breathe (i.e. provide air flow to encourage drying). Refer to Section 1.6.4 for photos of Norwegian bridges that use flashing in combination with pressure treatment.

A good approach to timber bridge design will use a combination of all the above. In addition, the use of preservative treatments will add to the long-term durability of the bridge.

Preservative treatment

Pressure treating timber bridges is a requirement of the CHBDC. All wood components of a bridge require pressure treatment, such as guardrails, decks, beams, girders, arches, trusses, the substructure (trestles or bents), etc. Unfortunately, it is not always clear for the designer what pressure treatment should be used as there are many considerations such as: wood species, type of wood product, type of exposure (air or ground contact), intended service life (is it a temporary or long-term bridge), availability of treatments, local health regulations, effect of treatment chemical on steel components, expected type of decay (fungal or insect) and effects of possible toxicity to local environmental conditions (Humphries *et al.*, 2009). As shown in Table 1, the type of exposure often dictates the types of preservatives that can be used (note the variety of available preservatives). As the footnotes to the table indicate, the CSA standards may have specific requirements that differ from this table.

Type of exposure	Preservative
Seawater	Creosote, chromated copper arsenate (CCA), Ammoniacal copper zinc arsenate (ACZA)
Fresh water or terrestrial piles	All above plus pentachlorophenol, oilborne copper naphthenate alkaline copper quat (ACQ) and copper azole (CA-B)
Critical ground contact	All above plus ESR-1721, ESR-1980 and ESR-2325
Ground contact	All above plus acid copper chromate (ACC), waterborne copper napthenate, ESR-2325, ESR-2500 and ESR-2500-B
Above-ground, fresh water	All above plus copper xyligen (CX-A), 4,5-dichloro-2-N-octyl-4-isothiazolin-3-one and imidacloprid (EL2), propiconazole-tebuconazole-imicloprid (PTI)and ESR-2067
Dry or occasionally damp	All above plus SBX (borates)

**(Adapted from Specifying Alternatives to Conventional Treatments,
Wood Design & Building, Winter 2017.)**

Preservative treatments can be oil-borne or water-borne. Oil-borne preservatives are the traditional types of treatments. Water-borne treatments normally have a copper component and are typically used on timbers which may have direct pedestrian contact. The chemical treatment for preservatives may off-gas, or leach, and should be a consideration during the design process. Disposing off-cuts or components that are removed during retrofit work may also need to be considered.

CHBDC states that one of the following preservative treatments must be used for timber bridges (Clause 9.17.1):

a. creosote;
b. pentachlorophenol in Type A hydrocarbon solvent;
c. copper naphthenate in Type A hydrocarbon solvent;
d. chromated copper arsenate (CCA);
e. ammoniacal copper zinc arsenate (ACZA);
f. alkaline copper quaternary (ACQ); or
g. copper azole type B (CA-B).

The standard requires that the oil-borne preservatives must be used in all locations except where pedestrians may come into contact with the timbers in a walkway. This is because the oil-borne preservatives have an odour and may not leave a clean surface for human contact. Only the last four treatments are water-borne (CCA, ACZA, ACQ and CA-B) and are permitted for pedestrian contact per Clause 9.17.2. For more detailed information on various treatment types, see Humphries *et al.*, 2009, and Morris *et al.*, 2009.

Odor or oily surface a concern?	Exclude creosote, pentachlorophenol in heavy solvent, oilborne copper napthenate. Check with coatings suppliers regarding painting or staining.
Frequent human hand contact?	Exclude CCA, ACZA and those listed above
Free of color?	PTI, EL2 and ESR-2067
Use in building interiors	All except creosote, pentachlorophenol and copper naphthenate. However, regulatory agencies strictly limit allowed interior uses of ACZA, CCA and ACC.
Little or no additional corrosion of fasteners?	Creosote, pentachlorophenol in heavy or light solvent, copper naphthenate, oxine copper, borates

(Adapted from Wood Preservatives: Choosing the right one, Wood Design & Building, Spring 2009.)

There is another layer to the requirements for permissible pressure treatments. The preservative standard, CSA O80, references the Health Canada Pest Management Regulatory Authority (PMRA) which limits the use of certain preservatives. As a result, creosote is not permitted except for railroad ties, utility poles and marine pilings. PMRA requirements are subject to change, so designers must ensure they are compliant with the latest PMRA requirements by checking regularly.

If most oil-based preservative are less friendly, why can't water-based preservatives be used instead? The primary concern is that water-borne preservatives affect the moisture content of timbers and can cause excessive checking and dimensional instability, particularly in large timbers, or additional loss of prestress in post-tensioned decks. Steel connectors and components can be affected by the water-borne preservatives initially and in the long-term. The designer must check for compatibility of all preservatives with the steel hardware, (even if hot-dipped galvanized) to ensure corrosion is inhibited. CHBDC provides requirements for the preservatives used that come in contact with stainless or hot-dipped galvanized steel.

Other structural composite lumber products, such as laminated veneer lumber (LVL) and parallel strand lumber (PSL), if used in a bridge, must be treated. However, the treatment is per US standards – the Canadian standard CSA O80 does not currently cover treatment of these products. Clause 9.17.5 and 9.17.6 require that bore samples must be taken of the timbers after treatment to ensure proper penetration of the preservative. Presumably the bore holes would require field treatment. Field treatments can also be done for notches or drilled holes done on site, but it is best practice to have all the notches and holes prefabricated and apply the pressure treatment after these are done to provide maximum durability.

Figure 1.4.3. Waterfront wave decks, Toronto. Yellow cedar glulam timber girders.
Photo provided courtesy of the Ontario Wood *WORKS!* Wood Design Awards program.

Replaceable elements

Designers might want to consider how the future replacement of bridge decks will be carried out when they design the layout and connection of the deck to the sub-structure. In addition, if part of the sub-structure deteriorates earlier than expected, designers should consider the method of replacement of single components to avoid having to extensively retrofit or replace a bridge before it reaches its service life. The bridge should be designed such that at some point in its future, a single element can be replaced relatively easily, without significant disruption or cost.

Inspection and maintenance are normal parts of the life-cycle routine of bridges. For timber bridges, the deck is likely to require repair and/or replacement during the 75-year service life. From the CHBDC Commentary a typical service life of 5 to 10 years is expected for a timber wearing layer, and 15 to 20 years is expected for an asphalt wearing layer. The timber wearing layer and the asphalt wearing layer are examples of protection elements which are not the same as the main structure.

1.5. BENEFITS OF WOOD BRIDGES

Why build a bridge out of wood, when it is common in Ontario to build a bridge out of steel or concrete? Aesthetics could be one reason; however, timber bridges are quite often low profile and drivers are unaware that the structure beneath them is wood. There are landmark examples of timber bridges in Canada which have their own aesthetic and help to define a sense of place (they "fit in" with their surrounding environment. Whether architects are involved or not in bridge design, exposed wood is often very visually pleasing. Wood bridges offer a great opportunity for structural and architectural expression. Aesthetics is a requirement by the CHBDC, Clause 1.4.2.8 states that:

> *"Structures shall be simple and graceful in form, shall intrude minimally on desirable scenery, and shall exhibit an integrity in which the function of components is explicit in their form and their size realistically reflects necessary strength. Visual discontinuities or abrupt changes shall be avoided"*

Aesthetics are an influencing factor for a bridge, some other common themes regarding potential benefits for designers and society are cost, construction cycle, and sustainability (see Section 1.5.1, 1.5.2, and 1.5.3). More often than not, it is economy and access, however, that make wood bridges an attractive design option.

1.5.1. COST

Wood bridges can offer a cost competitive alternative to conventional steel and concrete bridges. A large driving factor in its cost effectiveness is its lighter self weight compared to steel and concrete. This corresponds to the following cost benefits:

- Smaller foundations
- Smaller earthquake loads – less complex connections to substructure
- Smaller temporary structures and cranes
- Quicker installation (especially when paired with prefabrication), that saves on overhead costs (although, this can be dependant on the experience of the builders)
- Lower transportation costs (transporting less load)

An important decision-making tool for designers during the design process is costing. Many bridges built in Ontario are funded by the government, and designs can often be decidedly solely on cost. When choosing between systems or elements of the structure an understanding of cost implications can help with the initial design decisions. For example, understanding that by volume solid sawn dimensional lumber is less expensive than glulam lumber, the designer will try to work with solid sawn before considering glulam. However, initial material cost, is only one aspect of costing, design efficiency is another consideration. For example, one material type may be cheaper up front, but going with the more expensive material could eliminate the number of supports required, or reduce the number of connections required, which could out-weigh the initial cost difference. However, designers will need more than just broad comments on cost-effectiveness – they require pricing corresponding to specific systems and bridge types. For example, is an arch bridge more cost effective than using truss? There is not a lot of information about this in North America.

Direct and indirect reports of cost of construction and long-term maintenance costs tend to be favourable for timber bridges. A study by Behr et al. in 1991 investigated the cost of three bridge options in New England for spans of 20-, 40- and 60-feet. Using cost estimates by contractors for three designs (timber, steel/concrete and prestressed concrete), the authors found costs were competitive for timber and less expensive than prestressed concrete for initial costs for these short-span timber bridges in that region. In their analysis, they found that timber contractors pricing was much lower than pricing by general contractors who were unfamiliar with timber bridge construction.

In another reference, the 1990 U.S. publication *Timber Bridges* states:

> *"…timber has continued to be economically competitive with other bridge materials, both on a first-cost basis and a life-cycle basis."*

More recently, and in Canada, the designers of the Mistissini Bridge in Quebec noted that having an experienced wood contractor would have likely made the erection sequencing more efficient (Lefebvre, Richard, 2014).

1.5.2. CONSTRUCTION CYCLE

Three areas contributing to the construction cycle of bridges are the speed of placement, the size of the foundations, and the repair/replacement. The overall dead load of timber bridges is generally less than the design of concrete or steel bridges. This makes erection easier and potentially quicker, lessens the lift load for temporary jack supports, reduces the seismic load and substructure connections/foundations. If smaller, lightweight timber bridges are developed, they could offer an opportunity for fast deployment and minimal disruption to traffic. Prefabrication can aid to further reduce erection or replacement time on site.

1.5.3. SUSTAINABILITY

The Transportation Association of Canada printed a short primer on Sustainability Considerations for Bridges (TAC, 2015). The concept of sustainability is described as "interconnected goals of preserving and protecting the environment and preserving the ability of society to sustain itself. Supporting the natural, social and economic systems of today and the future (e.g. clean air and water, or safe and efficient transportation system)" (TAC, 2015). The TAC suggests the following objectives to provide better sustainability of virgin material use:

- Optimize waste stream
- Reduce energy use
- Reduce emission to air
- Maintain or improve hydrologic regime characteristics
- Maintain biodiversity
- Engage community values and sense of place
- Improve safety
- Improve access and mobility
- Improve local economy
- Increase lifecycle efficiency
- Promote innovation

TAC suggests the following sustainability practices to help achieve some the sustainability objectives noted above (**bolded** practices can be achieved with the use of a wood bridge structure):

- Addressing scour
- **Bridge aesthetics**
- Bridge lifecycle cost analysis
- Durability
- Embrace public participation
- Hazardous material reduction
- Importance of safety for bridge users
- Improve material reuse and recycling
- Improve the quality of receiving waters
- Increase local procurement
- Lifecycle assessment
- Maintain or improve access for bridge users (pedestrians and cyclists)
- Maintain or improve access for transit
- Maintain or improve aquatic ecosystems
- Maintain or improve terrestrial ecosystems
- **Material reduction**
- **Reduce and divert construction waste**
- **Reduce bridge greenhouse gas emissions**
- **Reduce construction traffic delay**
- **Reduce fossil fuel combustion in construction, maintenance and deconstruction**
- **Reduce noise pollution**
- Resilience

Environmental Declarations

A method to provide a transparent means on the sustainability of a product is an Environmental Product Declaration (EPD). An EPD is a standardised report on environmental performance for products, materials, and services. How an EPD is conducted is detailed by ISO 21930 and is verified by a third party (Evans, 2015). This is a relatively new standard in North America, and so far, is only made available by large industries or manufacturers, but this can be expected to grow with peaking interest in environmental impacts of design. Information presented in an EPD is based on findings from a life cycle analysis (LCA). This includes both energy and off-gassing and embodied environmental impacts (such as the amount of fresh water that goes into production). In North America, there are EPDs for several wood products for solid sawn and glulam, it would be beneficial to have EPDs on wood treatment chemicals to aid designers.

Municipal Class Environmental Assessment

Designers should consult with local municipalities to determine if municipal environmental assessments have been carried out or are required for bridge construction.

Sustainable design

Although this may not be a large contributing factor in the material choice for a bridge, wood bridges offer the possibility of a net negative greenhouse gas emission.

The green building movement often uses wood (mass timber) as a means to offset the carbon emissions of the building. The same benefit could be promoted with timber bridges. Ironically, the metric normally attached to carbon offset in buildings is the number of annual car trips – a similar measure for bridges could be adopted with the equivalent number of cars passing over the bridge.

Recently, the design and construction of a 160-Metre-Long Wood Bridge in Mistissini, Québec (Lefebvre, Richard, 2014) used a life cycle assessment comparing the carbon emissions from the design wood bridge and compared it with the carbon emissions of an equivalent steel-concrete bridge. The wood deck alone had -981 tonnes of CO_2 and had a net total emissions of -497 tonnes CO_2. The steel-concrete bridge had a total +969 tonnes CO_2 emissions (the wood deck alone could offset the emissions of the steel-concrete bridge).

1.6. WOOD BRIDGE EXAMPLES

The following are examples wood bridges in Ontario, Canada, the United States and other countries to illustrate the broad range of spans, systems, treatments, and a range of aesthetics from utilitarian to picturesque.

1.6.1. ONTARIO BRIDGE EXAMPLES

Mather Creek Bridge

Photo Credit: MTO

Mather Creek Bridge was constructed in 2008 along highway 71 in Ontario. The deck is a longitudinally laminated parallel strand lumber (PSL) slab with a composite precast concrete deck. The deck is supported on steel pile abutments. It spans approximately 23 m.

Sioux Narrows Bridge

Sioux Narrows Bridge was originally constructed in 1936 in Kenora Ontario. It has a main span of 64 m and a total length of 120 m. Solid sawn D.Fir members formed a box Howe truss. The truss supports wood king post floor beams. The floor beams support longitudinal stringers that support a transverse laminated wood deck with a 25 mm asphalt wearing surface.

- Truss compression chord 190 mm x 483 mm, bottom chord 190 mm x 533 mm
- Truss diagonals vary in widths of 184 mm to 289 mm and depths of 241 mm to 394 mm
- Floor beams are 64 mm to 95 mm wide by 337 mm deep
- King posts taper from 292 mm to 356 mm to 178 mm x 279 mm
- Floor beams 115 mm x 285 mm two span continuous
- 140 mm deep deck

It has since been replaced in 2006/2007 with a steel truss bridge cladded in timber to mimic the original appearance (Taylor, 1987).

Kabaigon River Bridge

Kabaigon River bridge was originally construction in 1979 in Atikokan, Ontario. It was a post-tensioned, longitudinally nail laminated deck. In 2015 it was replaced with a precast concrete box girder bridge on a steel substructure.

North Pagwachuan River Bridge

Photo Credit: MTO

The North Pagwachuan River Bridge is a single span bridge constructed in 1993 on Highway 17 in Northern Ontario (about 400 km from Thunder Bay). It is a longitudinally laminated timber deck (38 mm x 286 mm D.Fir) with transverse post tensioning. The deck is supported on steel girders, spanning approximately 50 m. This bridge was a prototype of a composite wood-steel bridge (Krisciunas). Concrete and steel bridge options would have been difficult to implement due to the remote location. All the wood was prefabricated and treated before arriving on site. The substructure is steel piles with timber lagging, due to poor soils and the need for an alternative to concrete (Krisciunas).

Hoiles Creek Bridge

Photo Credit: MTO

The Hoiles Creek Bridge is a single span bridge constructed in 1994 on Highway 11 in Ontario. The deck is comprised of longitudinally laminated timbers with transverse post-tensioning. The deck is supported on steel girders. The system used is similar to the North Pagwachuan Bridge (described above). The bridge spans approximately 30 m.

Nestor Falls Bridge

Photo Credit: MTO

Nestor Falls Bridge was constructed in 2009 near Kenora, Ontario. The deck is a longitudinally laminated PSL slab with a composite precast concrete deck. The bridge is a single span of approximately 15.7 m.

Silver Falls Creek Bridge

Photo Credit: MTO

Silver Falls Creek Bridge was reconstructed in 2014/2015 in Ontario. The deck is a longitudinally laminated PSL slab that is supported by steel sheet pile abutments. The bridge is a single span, spanning approximately 8 m.

Pennock Creek Culvert

Photo Credit: MTO

Pennock Creek Culvert was constructed in 2012 along highway 17 in Ontario. The deck is a laminated PSL deck supported on steel sheet pile abutments. The bridge is a single span, spanning approximately 8 m.

Duchesnay Creek Bridge

The Duchesnay Creek Bridge was constructed in 1937 in North Bay Ontario and has recently been proposed to be replaced. The bridge was comprised of a laminated timber deck truss system with three trusses across the width of the deck, supported on concrete piers and timber piles. The replacement bridge is proposed to be a concrete deck on steel girders (Stantec, 2013).

Providence Road Bridge

Photo: Design by Wood Research and Development and Installation by Timber Restoration Systems

The Providence Road Bridge was replaced in 2013 due to deterioration and decay. It is a five-span vehicle overpass in Oshawa, Ontario. The original bridge was a transverse timber deck supported on 15 timber stringers (per span) that were supported on timber caps. The replacement bridge consists of a transverse glulam deck supported on curved glulam girders. The timber guard was also crash-tested. All of the timber components of the bridge were prefabricated and pressure treated and site installed.

Dickinson Road Bridge

Photo: Design by Wood Research and Development and Installation by Timber Restoration Systems

The Dickinson Road Bridge was replaced in 2013/2014 due to safety concerns in Port Hope, Ontario. It is a five-span vehicle overpass bridge supported on two abutments and six pile bents. The replaced bridge consists of a transverse glulam deck supported on curved glulam girders. Timber guards were crash-tested. All the timber components were prefabricated and pressure treated, then installed on site.

Snake Road Bridge

Photo: Design by Wood Research and Development and Installation by Timber Restoration Systems

The Snake Road Bridge was originally constructed in 1912 and was retrofitted and replaced in 2013/2014 due to inadequate support of the bridge (no frost protection that lead to decay of some on the sub piles). The bridge is a seven-span vehicle overpass between Hamilton and Burlington, Ontario. The replaced bridge is comprised of a transverse glulam deck supported on curved glulam girders. The timber guard rails were crash-tested. All wood components were prefabricated and pressure treated and then assembled on site.

1.6.2. OTHER CANADIAN BRIDGE EXAMPLES

Carney Mill Road Bridge

Photo Credit: British Columbia Department of Highway Bridge Design

The Carny Mill Road Bridge was constructed in 2010 in British Columbia. It is a 3-pin glulam arch bridge and spans about 21.4 m. The deck is longitudinally laminated timber, supported on steel floor beams. The glulam components were treated with pentachlorophenol (Farden, 2010).

Lillooet Bridge

Photo Credit: British Columbia Department of Highway Bridge Design

The Lillooet suspension bridge was built in 1946 in British Columbia. It has since been transferred and serves as a pedestrian only bridge with a steel truss retrofit. The bridge consisted of a timber deck supported on timber singers. The suspension span was approximately 103 m and the timber bents supported up to 46 m on one end.

Answer Creek Bridge

Photo Credit: British Columbia Department of Highway Bridge Design

The Answer Creek Bridge is on Vancouver Island, British Columbia. It is a non-composite concrete deck supported on glulam girders that were treated with pentachlorophenol (Farnden, 2010).

Large Creek Bridge

Photo Credit: British Columbia Department of Highway Bridge Design

The Large Creek Bridge is on Vancouver Island, British Columbia, and is almost identical to the Answer Creek Bridge (described above). It is a non-composite concrete deck supported on glulam girders pressure treated with pentachlorophenol (Farnden, 2010).

Mistissini Bridge

Photo Credit: Stephane Groleau

The Mistissini Bridge is in Mistissini, Quebec. Due to the remote location of the project (about 600 km north-east of Quebec City) there was lenience to use local materials. Using glulam over a steel-concrete or steel-wood was found to be slightly less expensive, however, the "local" supplier provided wood from a sustainable forest. During the assessment of the design options, CO_2 emissions was one area of consideration. The designers found that the CO_2 emission from the proposed glulam bridge would be -497 tons, meaning these are avoided emissions. The steel-concrete bridge solution would result in net emissions of +969 tons of CO_2. The bridge spans 160 m with intermediate supports at 37 – 43 m. The glulam girders are spliced at connections to the glulam arches. The bridge is constructed with glulam panels 184 mm deep x 921 mm wide. The girders and arches are supported by concrete piers (Lefebvre, Richard, 2014).

Guardian Bridge Rapid Construction Inc.

Photo Credit: Guardian Bridge Rapid Construction Inc.

Above is an example of a timber bridge in Canada that used the Guardian Bridge technology. Guardian Bridge prefabricates a bridge that is externally reinforced with a fibreglass composite to improve durability, strength and erection time compared to similar style bridges and structural members. This technology can also be used for non-timber bridges with similar benefits. Although, typically used for retrofits or repairs for deteriorating bridges, this can be applied for new bridges as well.

1.6.3. AMERICAN BRIDGE EXAMPLES

Lower Burnett Road Bridge

Photo courtesy of Western Wood Structures Inc. Tualatin, Oregon

The Lower Burnett Road Bridge is a vehicle and pedestrian bridge in Buckley, Washington. This bridge is a curved glulam three arch bridge with a 198 m horizontal curve. The deck is a longitudinal glulam deck supported on timber bents that are supported by the main arches. The use of arches was chosen to reduce the number of piers to meet the AASHTO aesthetic guidelines and limit the exposure to scouring potential from the overflow of the South Prairie Creek. One challenging aspect was the 28 unique bents due to the horizontal curve which was overcome with the aid of prefabrication (Gilham, 2013).

Overpeck Park Bridges

Photo courtesy of Western Wood Structures Inc. Tualatin, Oregon

The Overpeck Park Bridge in Bergen County, New Jersey is a vehicular bridge designed for AASHTO HS20 with a 25 percent overload (400 kN or 90,000 lb vehicle). The bridge consists of two identical three-pin glulam arches, each spanning approximately 42 m. The deck is a 223 mm thick longitudinal glulam supported on transverse and diagonal glulam braces. The braces are also used for lateral support of the arches. The arches are 362 mm x 1524 mm deep with a peak height of 9.6 m. Due to the size of the arches, the arches were designed with moment splices so reduce the arch segment sizes small enough to fit in a pressure-treating cylinder. (Gilham, 2013).

1.6.4. INTERNATIONAL BRIDGE EXAMPLES

Tynset Bridge

Tynset Bridge in Norway is a three-span arch bridge. The largest span is 70 m and the arch is comprised of a glulam truss with 12 steel hangers connecting to the steel beams supporting the deck. The deck is a stress laminated timber deck. The other two arches span 26.5 m with a three-pin curved glulam arch. The bridge supports normal road traffic and pedestrian/bicycle traffic. The smaller arches are protected by design (i.e. no pressure treatment), but the glulam truss arch is pressure treated.

Skogsrud Bridge

Photo Credit: FPInnovations

A Norwegian vehicular bridge constructed in 2007 with a 37 m span and 49 m length. It is a three-pin glulam arch with stress laminated glulam deck, supported on transverse steel beams. The glulam arches are treated with a copper-based treatment and then coated in oil-based paint, the deck is treated with copper-based treatment and then impregnated with creosote. In addition to the wood treatments, the arches have metal flashings to protect against water infiltration.

Tretten Bridge

Photo Credit: FPInnovations

A Norwegian vehicular bridge constructed in 2012. It is a three-span bridge with the largest span of 70.2 m and total length of 148 m. The bridge is a glulam truss girder with a stress laminated glulam deck, supported on transverse steel beams. Both the girder truss and the deck are dual-treated with copper-based preservatives and then impregnated with creosote. In addition to the treatment, the top and bottom chords of the truss have a copper flashing to protect against water infiltration. The bridge is a replacement of an original steel bridge. The new glulam bridge was a lighter alternative, allowed reuse of the substructure and was delivered quickly to site.

Evenstad Bridge

Photo Credit: FPInnovations

A Norwegian vehicular bridge constructed in 1996. It is an equal five span bridge with a total length of 180 m. Each span has a glulam arch truss and stress-laminated glulam deck supported on transverse steel beams. Both the deck and glulam arches were dual-treated with copper-based preservatives and then treated with creosote. The top and bottom chords, and diagonals of the glulam truss are covered in copper flashings.

Kjollsaeter Bridge
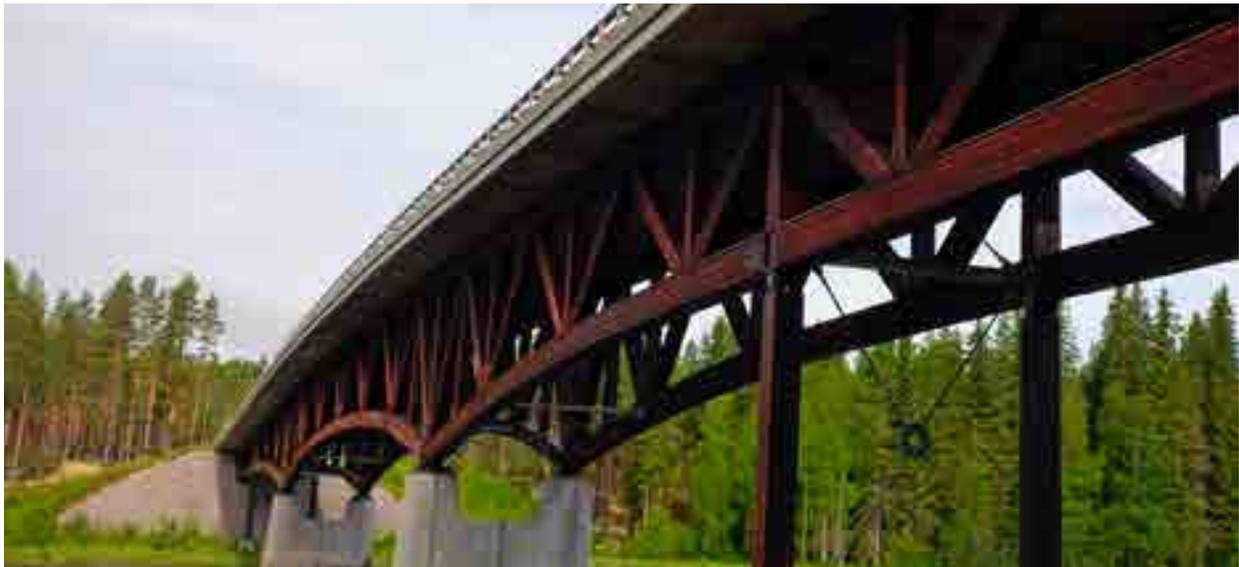
Photo Credit: FPInnovations

A Norwegian military traffic bridge constructed in 2005/2006. The bridge has a maximum span of 45 m and a total length of 145 m. The bridge is designed for military vehicles of 100 tons (and is quite possibly one of the heaviest loaded timber bridges in the world). The bridge is a glulam truss girder supporting a reinforced concrete deck. All timber elements of the truss are dual-treated with a copper-based preservative and then treated with creosote.

Flisa Bridge

Photo Credit: FPInnovations

A Norwegian vehicular bridge constructed in 2003. It is a three-span bridge with the largest span being 70 m. The bridge is an arched glulam truss girder with a glulam stress laminated deck support on transverse steel beams. Both the glulam truss and deck were dual-treated with CCA and then creosote. In addition to treatment, the top and bottom chords are covered in a copper flashing.

1.7. REFERENCES

1. Behr, R. A., Cundy, E. J., & Goodspeed, C. H. (1990). Cost comparison of timber, steel, and Prestressed concrete bridges. Journal of Structural Engineering, 116(12), 3448–3457.

2. Bell, K., & Karlsrud, E. Large Glulam Arch Bridges – A Feasibility Study(Rep.). Trondheim, Norway.

3. Christianson, J., Marston, C. H., Barker, J., Bennett, L., Conwill, J. D., Duwadi, S. R., Gaspar, D.A, O'Connor, R., Reckard, M.K., Sangree R.H., (2015). Covered bridges and the birth of American engineering. Washington, D.C.: National Park Service, Historic American Engineering Record.

4. CSA (2014) CSA O86-14 Engineering Design in Wood. Mississauga: Canadian Standards Association.

5. CSA (2006) CAN/CSA-S6-06 Canadian Highway Bridge Design Code. Mississauga: Canadian Standards Association.

6. CSA (2006). S6.1-06 Commentary on CAN/CSA-S6-06, Canadian Highway Bridge Design Code. Mississauga: Canadian Standards Association.

7. Cramahe Community Association – Website. (n.d.). Retrieved March 21, 2017, from https://www.cramahecommunityassociation.ca/

8. Evans, L. (2015). Wood and Environmental Product Declarations. reThink Wood.

9. Farnden, G. (2010). 2010 CWPA Conference. Bridge Timbers.

10. FPInnovations (2014). FPI/TWBC – Technical Guide for the Design and Construction of Tall Wood

11. Buildings in Canada (2014 ed.). Pointe-Claire, QC: FPInnovations.

12. Gilham, P. (2013). Stretching the Limits – Modern Timber Bridge Case Studies. In Timber Bridge Conference. Tualatin, Oregon.

13. Humphries, M., Lebow, S., Moses, D. (2009). Wood Preservatives Choosing the right one. Wood Design and Building Magazine

14. Krisciunas, R. (n.d.). Ontario's Experience with Composite Wood/Steel Bridges(pp. 313-320, Rep.). Thunder Bay, ON: Minsitry of Transportation.

15. Lefebvre, D., & Richard, G. (2014). Design and construction of a 160-Metre-Long Wood Bridge in Mistissini, Quebec.

16. Mohammad, M., & Natural Resources Canada. (2014, September). Innovative Timber Bridges: Technical Tour of Norwegian Timber Bridges. FP Innovations.

17. Morris, P. Durability of Wood Bridges. FPInnovations.

18. Morris, P., Moses, D., & Lebow, S. (2009). Specifying Alternatives to Conventional Treatments. Wood Design and Building Magazine.

19. Ontario Heritage Bridge Guidelines For Provincially Owned Bridges. (2008). Ministry of Transportation.

20. Ritter, M. (1990). Timber Bridges Design, Construction, Inspection, and Maintenance. Washington, DC.

21. Stantec Consulting Ltd. (2013). Structural Planning Report Duchesnay Creek Bridge Site 43-067, Highway 17B. North Bay, Ontario: Ministry of Transportation.

22. Taylor, R. (1987). Load Testing and Rehabilitation of the Sioux Narrows Bridge. The Research and Development Branch Ontario Ministry of Transportation.

23. Taylor, R. J., & Keenan, F. J. (1992). Wood Highway Bridges. Canadian Wood Council.

24. Tingley, D., Keller, R., Arthur, R., Hunter, M., & Legg, C. (2015). Developing the Market for Solid Sawn and Engineered Wood Products in North American Bridges. Canadian Wood Council.

25. Transportation Association of Canada TAC (2015). Primer on The Sustainability Considerations for Bridges.

26. "TRS – Timber Restoration Services Canada". TRS – Timber Restoration Services Canada. N.p., 2017. Web. 21 Mar. 2017. https://www.timberrestorationservicescanada.com

Part Two: Opportunities & Current Limitations

2.1. INTRODUCTION

Timber bridges have a long history of construction and use throughout North America, including Ontario, for roadways, railways and logging roads. The style and span of bridges varies greatly depending on the application. For example, timber trestle bridges were common at the end of the 19th century and into the beginning of the 20th century in hard to reach locations with very deep valleys to avoid having to fill the valleys with earth. Trestle bridges relied heavily on ample timber resources and in some cases, were considered to be temporary. But that is just one type of timber bridge.

Currently, in Ontario there are 157 timber bridges and culverts in service with the Ministry of Transportation Ontario (MTO) (Mermigas, 2017). Between 2009 and 2014, 172 new bridges were tendered, of which only one was timber. Most of the timber bridges on public highways in Ontario were built prior to 1980. Of the new bridges tendered in that same period, between 2009 and 2014, more than half of them (56%) had an average span of 30 m or less. There appears to be an excellent potential market for wood bridges which tend to be in that same ranges of spans.

For comparison, from the US National Bridge Inventory, the total number of bridges in their inventory as of 2015 was 362,624. Of these, 39,038 were wood (about 11%). States with the greatest percentage of wood bridges (number of wood bridges divided by total number of bridges in each state) were: Alaska (41%), Montana (29%) and North Dakota (21%). The total square metres of deck area as of 2015 was 369 million. Of this, wood bridges account for 4.3 million square metres (about 1%). This implies that timber is used primarily for relatively small bridges. The same states ranked highest for percentage of timber deck surface area relative to total deck area (Ritter, Duwadi, Lee, 1996).

A study in the United States in 1987 found that nationwide, with over 575,000 bridges in their inventory (including steel, concrete and timber bridges), over one-third of bridges that fall outside of federal support were structurally deficient (Ritter, Duwadi, Lee, 1996). The authors' data also showed that timber bridges were cost competitive with other materials "both on a first-cost basis and a life-cycle cost basis."

In Ontario, the Ministry of Transportation and universities have researched numerous improvements and new ideas in timber bridge technology over the decades building many proof-of-concept bridges and monitoring them. Historically, there have been periods with almost no new wood bridge construction or research – the record is erratic and has been dependent on funding and having true champions of wood bridges.

The reasons for these cycles are not completely clear. However, government funding of bridge construction and research has faced ever-changing priorities. Authorities and their designers tended to move to appease current priorities for bridge types and materials. Researchers move towards topics that are more likely to be awarded funding, and if bridges are not a hot topic, or more out of favour, they may be more likely to pursue other topics. Compounding this, with leaner budgets, bridge authorities typically waver from trying to innovate and tend towards known designs and methods which in turn cause designers, suppliers and builders to conform to a limited set of bridge types. In a chicken-and-egg cycle, with less information, less likelihood of getting research funding, less design knowledge, less builder familiarity, the interest in wood bridges wanes. When designers and authorities are educated, and when research on simplifying the design process or answering common designer questions is easily accessible to designers, interest and capacity to build wood bridges expands.

Whereas the green building movement has helped move designers of buildings towards wood as an environmentally-friendly, sustainable method of construction, bridge authorities and designers are not motivated by environmentalism – their needs are simple: build a bridge that is reliable, long-lasting and cost effective. In a sense, this is an informal life-cycle cost analysis, self-regulated through the basic need to be cost effective and durable, without the environment explicitly in the equation. The main driver for green buildings has been to reduce operating energy and to improve human comfort. Turning this around, bridges have no operating energy requirements. Compared to buildings, which have mechanical and electrical systems that consume operating energy, bridges rank very highly on embodied energy relative to all energy – this makes bridge construction in wood a potential for net zero construction, i.e. the sequestered carbon in a bridge could potentially outweigh all the energy required to construct the bridge in the first place.

The main issues preventing a significant uptake in wood bridge construction stem from three main sources:

Issue 1: Challenges in the bridge design code;
Issue 2: Lack of education around bridges for authorities, designers and builders; and
Issue 3: Limited information on the cost of wood bridges.

Issue 1: The bridge design code affords many options for wood bridges to be designed and built. There are some technical challenges which stem from a combination of technical improvements that can simplify design to the ease at which a designer can propose and implement an innovative or novel design. But there are instances in the bridge design code where clearly education is the obstacle. A review of current standards is the primary focus of this document.

Issue 2: While the bridge design code has a lot of information in it, the code itself is not a substitute for education. It is assumed that engineers who use the code are already competent. Education around topics such as appropriate choice of wood system, spans, methods of analysis and durability are crucial to ensure designers understand the principles of design.

Detail on topics such as preservative technologies and detailing for long-term performance are not easy to glean from just a read through of the bridge design code. Preservative treatment has terminology and many details and take time to learn. Another area that is not obvious to the reader of the bridge code is how to decide the appropriate type and location ideally suited for timber bridges, and determining the optimum use of this typology. Finding this information is not always easy. Where is the designer to turn? Are there fully worked design examples of specific bridge types? Are additional courses available and if so, by whom?

Design examples can offer immediate solutions to specific problems. On the other hand, clear judgement of design issues and design decisions only come with time and experience, and without a deep pool of knowledge and experience, where can a designer turn for help? And with that help, how will they assess the relative cost of various wood bridge options offered by the code?

Issue 3: Cost studies have been carried out over the decades comparing wood bridges to steel, concrete and pre-stressed concrete. In general, wood has been shown to be the more economical option at the time of construction. Long-term performance and maintenance cost data is harder to find. The number of years a bridge lasts will be part of the life-cycle cost equation because the replacement and disruption costs during the replacement phase can be significant. The costs are dependent on the use, span and type of wood bridge. To determine a 'sweet spot' for wood bridges requires data on historical costs for construction, maintenance and replacement.

These three major topics (bridge code, education and cost) are covered in more detail in this report, with emphasis on the Canadian Highway Bridge Design Code (CHBDC).

2.2. REVIEW OF CAN/CSA S6 – CANADIAN HIGHWAY BRIDGE DESIGN CODE

The following is a review of the CAN/CSA Standard S6-14 and its commentary for requirements for timber bridges and their relevance for potential limitations on their design and use.

2.2.1. CAN/CSA S6 – SECTION 1 – GENERAL

This section has no requirements that are specific to timber bridges.

Clause 1.4.2.3 specifies that bridges built to this standard are required to have a service life of 75 years.

2.2.2. CAN/CSA S6 – SECTION 2 – DURABILITY

This section has no requirements that are specific to timber bridges.

Clause 2.3.2 provides guidance on the importance of detailing with durability in mind. This includes providing drainage to avoid water accumulation and the long-term performance of materials. The material-specific sections for concrete, steel and wood provide more specific provisions for each material. Section 9 – Wood Structures states the requirements for preservative treatment of wood and is discussed in detail later in section 2.9.8 of this report.

2.2.3. CAN/CSA S6 – SECTION 3 – LOADS

Applied loads and load factors in accordance with the limit states design philosophy are listed in Section 3 of the CHBDC.

Dynamic Load Allowance
The main requirement that is specific to timber bridges is the allowable reduction of 30% in the Dynamic Load Allowance for wood components compared to other materials, as noted in Clause 3.8.4.5.4. The Commentary on this clause states "bridges fabricated largely of wood, composite concrete and wood, and transversely or longitudinally laminated wood decks" fall under this category. This reduction is possible because of the improved performance of wood under short-term dynamic loads and the higher damping performance of long-span wood structures.

Shrinkage and swelling
Clause 3.9.1 states that temperature, shrinkage, and creep effects need not be analysed for conventional wood structures, "but shrinkage and swelling that are perpendicular to the grain are due to moisture changes shall be considered." Differential temperatures in wood are not a concern, however, shrinkage effects in the perpendicular-to-grain direction from moisture changes can be considerable. Commentary C3.9.1 states that designs should not restrain movement in timber bridges to avoid the build-up of internal stresses.

2.2.4. CAN/CSA S6 – SECTION 4 – SEISMIC DESIGN

This section has no requirements that are specific to timber bridges.

Seismic design requirements for concrete and steel bridges are provided in detail. However, timber bridges are not listed. Design for earthquake loading and resistance of timber bridges is still required per the standard, however, there are no specific clauses provided for timber as there are for concrete and steel on topics such as ductility and diaphragm design.

Generally, it is the design of the piers and the connection to the piers that are considered for seismic design. Common practice is to support timber bridges on concrete piers and abutments, however, wood piers are also an option. Clause 9.15 of the standard on connectors for wood provides many options for designers, but Section 4 – Seismic Design is silent on detailing connections in wood for the resistance of seismic loads for wood piers/piles/bracing.

Seismic design provisions for wood should be an item for future developments to the CHBDC as this is a challenge for designers.

2.2.5. CAN/CSA S6 – SECTION 5 – METHODS OF ANALYSIS

The CHBDC is unique among design standards in that it prescribes methods of analysis in addition to minimum requirements for safety, serviceability, and durability. In other design standards, it is assumed that users of the standards have the competence to calculate forces and displacements produced in structures by loads and other actions.

The clauses of Section 5 that are most often used are those relating to simplified methods of analysis for longitudinal load effects (Clause 5.6). This section relates to a set of common structural systems (such as slab on steel girder), and provides a simpler means of calculating forces produced by the transverse distribution of live load in multiple-girder systems. These requirements are not mandatory and other methods of analysis can be used to calculate the effect of live load in these and other structural systems.

Clause 5.5.8 Transverse wood deck
This clause specifies requirements for transverse moment in laminated wood decks that span transversely between longitudinal girders or stringers. For the design of vertically laminated wood decks, it is permitted to ignore shear effects per Clause 9.7.5.

Clause 5.6 Simplified method of analysis for longitudinal load effects
This clause provides simplified methods for calculating moments and shear forces in longitudinal structural components due to dead load and highway live load. Most of this Clause pertains to the calculation of live load effects in longitudinal components such as girders. In most structural systems consisting of multiple parallel structural components such as girders, the transverse flexibility of the system results in live load effects that are greater than the total live load divided by the total number of girders. Clause 5.6 provides simplified formulas that enable these effects to be taken into account.

The systems incorporating wood components covered by Clause 5.6 are wood decks spanning longitudinally (i.e., wood decks without longitudinal girders of any material), wood decks supported on steel girders, and wood decks supported on wood girders. Clause 5.6.4.2 gives provisions for calculating the truck load fraction F_T to be applied to a metre of width of wood decks spanning longitudinally. Clause 5.6.4.3 gives general formulas for calculating truck load fraction F_T to be applied to a given girder in systems with wood decks supported on steel girders

and wood decks supported on wood girders. The formulas defined in Clause 5.6.4.3 require calculation of a parameter called "truck load distribution width", D_T. This parameter is calculated using Clause 5.6.7.2 and Table 5.8 for steel girders supporting timber decks, and Clause 5.6.7.5, Table 5.11, and Table 5.12 for wood girders supporting timber decks.

Other structural systems, such as wood girders supporting a concrete deck slab, are not yet covered by Clause 5.6.

Commentary C5.6 notes that in general this approach is conservative. More refined approaches such as a grillage model can result in less conservative analysis.
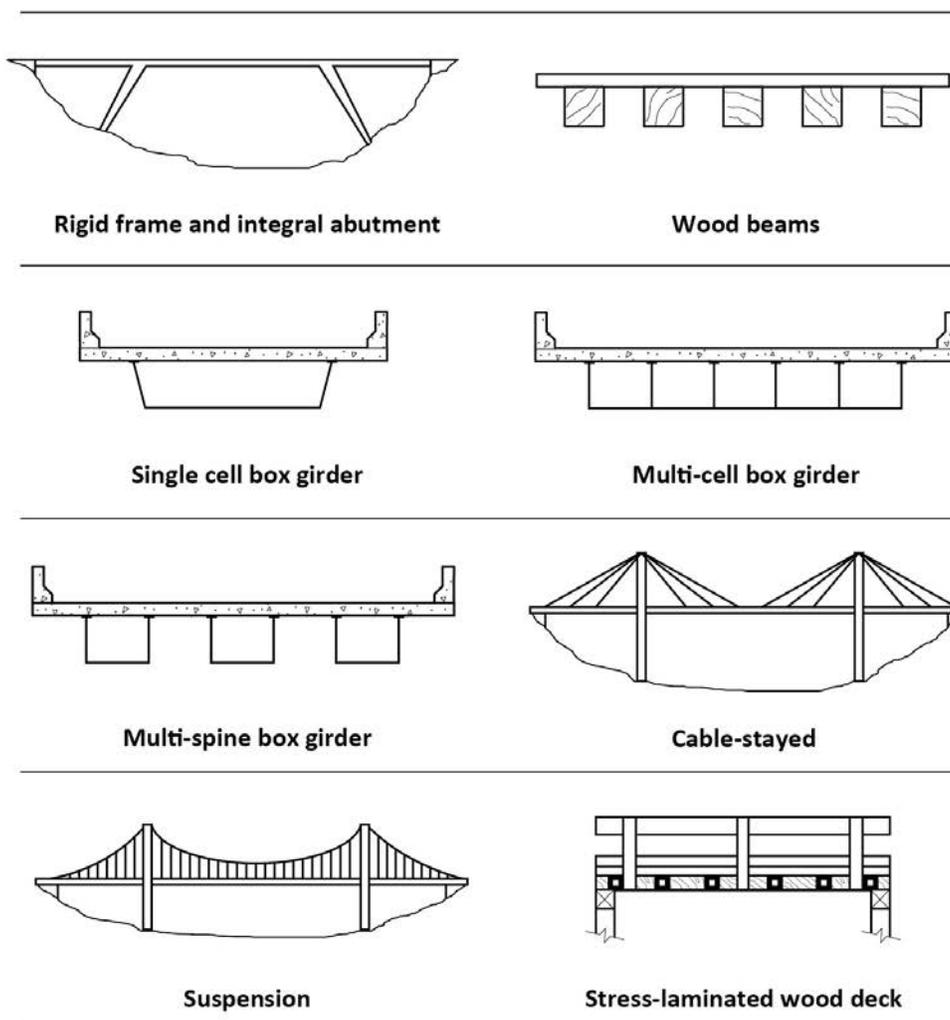
Various cross-sections and bridge types, image based on Figure C5.1 of CHBDC
(Source: Clause C5.1., Figure C.5.1 Representative cross-sections and elevations of bridge types –
Canadian Highway Bridge Design Code. © 2017 Canadian Standards Association)

On closer inspection, Clause 5.6.7.2, which has specific provisions for wood decks (i.e. laminated wood decks and wood plank decks) on steel girders, Table 5.8 specifies the value of D_T for moment calculations, for example, which varies between 3.0 and 3.75 for ULS, SLS and FLS in laminated decks but only 2.4 to 2.55 for wood plank decks.

Similarly, Clause 5.6.7.5 has provisions for wood decks on wood girder bridges or just wood deck bridges. Table 5.11 specifies D_T values for transverse laminated wood decks and wood plank decks supports by wood girders. The value of D_T for moment calculations varies between 2.6 and 3.6 for ULS, SLS and FLS limit states in laminated decks but only 2.4 to 2.55 for wood plank decks.

Clause 5.6.7.5 also refers to Table 5.12 which specifies D_T values for stress-laminated wood decks, nail-laminated wood decks and laminates of wood-concrete composite deck bridges. These decks are intended to span longitudinally. The value of D_T for moment calculations varies between 2.6 and 4.0 for ULS, SLS and FLS limit states for stress laminated decks, is 3.2 for wood-concrete composite decks and only 1.7 for nail-laminated wood decks. Note that Clause 9.21.3 limits the use of nail-laminated decks to only those cases where a composite wood-concrete deck is used to minimize loosening of nails which can lead to delamination and compromise the load sharing capacity of the deck. See notes for Section 9 – Wood Structures later in this report.

Clause 5.7 Analysis of decks

This section defines provisions for calculating forces in bridge decks spanning between primarily longitudinal elements such as girders. Clause 5.7.3 pertains specifically to wood decks.

The analysis to determine the moments in wood bridge decks is defined in Clause 5.7.3 for the following applications:

a) Transverse laminated wood decking on sawn timber stringers (Clause 5.7.3.1);
b) Transverse stress-laminated wood deck-on-girders (Clause 5.7.3.2);
c) Transverse nail-laminated wood deck-on-girders (Clause 5.7.3.3); and
d) Transverse wood plank deck-on-girders (Clause 5.7.3.4).

For these types of systems, formulas are given for the calculation of transverse bending moments in the timber deck. These formulas are either direct or expressed in terms of a width of deck over which wheel loads are to be distributed. These provisions are based on research performed by the MTO in the 1980s (references are provided in C5.7.3.1).

Annex A5.2 *Two-dimensional grillage analysis of wood floor systems* is provided as a non-mandatory section of the standard to aid designers with simplified methods of analysis of the three-dimensional deck and girder system for the following four bridge deck types:

a) Transverse laminated wood decks on longitudinal wood beams (Table A5.2.1);
b) Transverse laminated decks on longitudinal steel beams (Table A5.2.2);
c) Glued-laminated and transversely laminated prestressed decks (Table A.5.2.3); and
d) Composite concrete slabs on longitudinally laminated wood decks (Table A.5.2.4).

As noted above, the type of structural systems currently covered by this Annex are relatively limited. For example, wood girders with composite concrete decks are not covered.

Damping

Commentary Clause C5.11.1.4 regarding damping in dynamic analysis for vibration control notes that timber bridges have better responses to vibration compared to other materials due to a higher critical damping ratio: concrete construction 2%, welded and bolted steel construction 1%, and timber 5%.

2.2.6. CAN/CSA S6 – SECTION 6 – FOUNDATIONS AND GEOTECHNICAL SYSTEMS

This section has no requirements that are specific to timber bridges. There is one note related to splicing of wood piles (Clause 6.11.4.9) unrelated to bridge superstructures.

2.2.7. CAN/CSA S6 – SECTION 7 – BURIED STRUCTURES

This section has no requirements that are specific to timber bridges.

2.2.8. CAN/CSA S6 – SECTION 8 – CONCRETE STRUCTURES

This section has no requirements that are specific to timber bridges.

2.2.9. CAN/CSA S6 – SECTION 9 – WOOD STRUCTURES

Section 9 is specific to "wood components and their fastenings." The Clauses most relevant to this report are as follows.

2.2.9.1. CLAUSE 9.4 LIMIT STATES DESIGN

Clause 9.4 provides load and resistance factors for limit states design. Commentary C9.4.1 notes that only the Serviceability Limit State (SLS) and Ultimate Limit State (ULS) requirements are given for wood components however, no explicit Fatigue Limit State (FLS) requirements are given for wood. Designers should be aware that they may still need to consider FLS for steel components used in wood structures.

Clause 9.4.2 states that the vibration performance of the superstructure must be in accordance with Clause 3.4.4. These provisions relate primarily to the human perception of vibration.

2.2.9.2. CLAUSE 9.5 GENERAL DESIGN

Load-duration factor
Load-duration factors are specified in Clause 9.5.3 of CAN/CSA S6. (i.e. 0.7 for long-duration, 1.15 for short duration and 1.0 for all others). These are similar to the requirements in CSA O86 *Engineering Design in Wood*.

Service conditions factors
Clause 9.5.5 notes that the service condition factors for wet service conditions have been included in the design stresses in CAN/CSA S6. It is important to note that this is different from CSA O86 where the service condition factors are applied by the designer to the specified strengths of the wood materials.

Treatment factor
Clause 9.5.9 notes that the treatment factor for preservatives and incising has been included in the design stresses in CAN/CSA S6. Again, this is important to note that this is different from CSA O86 where the treatment factor is applied by the designer to the specified strengths of the wood materials. Also note that treatment factors typically apply for fire retardant treatments.

2.2.9.3. CLAUSE 9.6 FLEXURE

Flexural design is fairly consistent between CAN/CSA S6 and CSA O86. Designers should be aware that there is a slight difference in the calculation for glulam timber bending resistance. CAN/CSA S6 calculates Ck using E_{05} whereas CSA O86 uses 0.97E.

2.2.9.4. CLAUSE 9.7 SHEAR

Designers should be aware that the formulas for shear in Clause 9.7 of CAN/CSA S6 are provided in a different format than in CSA O86 *Engineering Design in Wood*. Both standards have two options for shear calculations. A simple formula is used for beams with volume less than 2.0 m^3 whereas a longer more detailed analysis is required for beams with volume over 2.0 m^3. (For example, a 6 m long 315x1064 girder is just over 2 m^3.) The long form solution in CAN/CSA S6 takes time to solve especially if there are many load cases and moving loads. CSA O86 provides tables of many typical conditions found in buildings to speed up the design process. CSA O86 also provides a table for moving loads. According to CAN/CSA S6 Commentary C9.7, the generalized formula was selected likely since timbers in bridges are relatively large and because bridge loading is normally a series of point loads, unlike buildings where uniformly distributed loads are the norm. The choice makes sense, however, implementation can be challenging.

Shear often controls the design of timber elements in bridge design, even with species that have higher specified shear strengths. It appears that CSA O86 is less conservative and easier to use for shear design. Designers should be aware that the formulas in CAN/CSA S6 become more challenging for multi-span design. The Commentary C9.7 includes one relatively simple design example for a pile cap.

2.2.9.5. CLAUSE 9.11 SOLID WOOD

Specified strengths and moduli of elasticity for all recognized species groups and grades of solid wood are provided in Tables 9.12, 9.13 and 9.14. The values in the tables have already been modified for wet service conditions. Table 9.12 for structural joists and planks footnote (e) notes that preservative treated and incised wood have been included in the design values. Tables 9.13 and 9.14 do not have this note, likely due to treatment factor of 1.0 for incised lumber in wet conditions for members 89 mm and larger. As noted in sub-section 2.4.2 above, designers should be aware that this is different from CSA O86.

2.2.9.6. CLAUSE 9.12 GLULAM TIMBER

Specified strengths and moduli of elasticity for Douglas fir glulam timber are provided in Tables 9.15 for six different grades. The table footnotes indicate values are based on wet service conditions however the table does not indicate that the treatment factor was applied, as noted in Clause 9.5.9. Designers should be aware that there is a discrepancy between the values of Table 9.15 of the CHBDC compared to the values provided in O86 (when multiplied by the wet service condition factor).

Designer should note that unlike solid sawn wood, and unlike CSA O86, no design values are provided for other species of glulam timber. No mention to this omission is provided in the standard or the commentary. Note that other species may be better for preservative treatment and have other useful properties.

Clause 9.12.4 specifies camber for long glulam timber spans to be twice the dead load deflection to account for creep. In addition, 1/600 of the span is added to the camber to account for sag that may be visually unappealing.

2.2.9.7. CLAUSE 9.15 FASTENINGS

Designers should be aware that certain connections in wood are not permitted. Clause 9.15.1 prohibits the use of glulam rivets where the bridge design life is greater than 2 years. It also prohibits truss nail plates (Commentary C9.15.1 refers to them only as truss plates) for bridges with a design life of more than 2 years. Steel nail plates are permitted if used in wood-concrete composite decks per Clause 9.22.2.2.3 (i.e. for splicing butt joints in wood laminates in concrete composite slab design).

It is worth noting that the European standard EN1995-2 Clause 8.1 prohibits the use of stapled or punched metal plate connections in wood bridges.

Fastenings (connectors) shall be designed with the load-duration factor specified in Clause 9.5.3 of CAN/CSA S6. (i.e. 0.7 for long-duration, 1.15 for short duration and 1.0 for all others). These are similar to the requirements in CSA O86 *Engineering Design in Wood*.

2.2.9.8. CLAUSE 9.17 DURABILITY

Design for durability of timber bridges is a concern for all designers and lack of education in this area may discourage designers from considering timber. There is a lack of standardized details for designers to use that show methods of shedding and preventing long-term water accumulation on timbers. And information on preservatives can be complicated to understand, with many options available. Compounding this is a note in the Commentary C9.17.1 which recommends that designers note the type of treatment on the construction drawings, meaning that the designers must understand how best to select a preservative.

Bridges built to CAN/CSA S6 are required to have a service life of 75 years (Clause 1.4.2.3). Many wood structures in Canada and around the world have been shown to last for this long and some significantly longer. Proper detailing for durability and protection of the structures is the key to their longevity. To achieve these long lifespans in bridges, design for durability should address three main criteria:

Protection by design: Provide protection of the timber elements by locating structural elements strategically (i.e. roofs, overhangs, flashings, sloped members);

Preservative treatment: Permanent treatment of timber elements to prevent decay from high moisture conditions and pest attack; and

Replaceable elements: Design a replaceable wearing surface with little or no effect on the main timber support structure.

Protection by design

A bridge can be designed such that it is inherently self-protecting by deflecting water away from the structural elements. Clause 2.3.2 of CAN/CSA S6 provides guidance on the importance of detailing with durability in mind. Section 1.4 of the Ontario Wood Bridge Reference Guide provides further explanation and examples of protection by design techniques.

Preservative treatment

The bridge code, CAN/CSA S6, references the CSA O80 standard series for preservative treatment. All timbers in bridges are required to be treated by one of the approved preservatives from the standards. It is currently difficult for a designer to assess the best treatment option since there are many parameters to consider (in addition to the minimum requirements dictated by the standards). The method of treatment and procedures and locations for incising timbers is not easily available for the average designer. CSA O80 is not a typical standard in an engineer's office.

There are other code requirements for preservatives as follows. The preservative standard, CSA O80, references the Health Canada Pest Management Regulatory Authority (PMRA) which limits the use of certain preservatives. As a result, creosote is not permitted except for railroad ties, utility poles and marine pilings, and designers must check the PMRA requirements regularly to ensure they are in compliance with the latest PMRA regulations. Likely most oil-borne preservatives are not a friendly as the water-borne preservatives, however, this is at odds with the requirements of the bridge code that restricts the use of water-borne preservatives except in areas in contact with pedestrians on walkways. The primary concern is that water-borne preservatives affect the moisture content of timbers and can cause excessive checking and dimensional instability, particularly in large timbers.

A note on species. Some species are easier to treat than others. Douglas fir is not particularly easy to treat. However, designers should be aware that Douglas fir is currently the only referenced species for design in CHBDC for glulam timber.

Steel connectors and components can be affected by the water-borne preservatives initially and in the long-term. The designer must check for compatibility of all preservatives with the steel hardware, even if hot-dipped galvanized) to ensure corrosion is inhibited. Clauses 9.17.11.1, 9.17.11.2 and 9.17.11.3 provide steel requirements for each of the types of preservatives for hot-dipped galvanized and stainless steel.

Clause 9.17.3 states the requirements for incising of timbers prior to preservative treatment. Incising creates additional pathways for the preservative to enter the timber. Incising must be noted on the drawings – designers should familiarize themselves with preservatives to know how to specify incising.

A typical requirement for preservative treatment is to cut notches and drill all holes prior to treatment so that the newly exposed surfaces can be treated. Preservative treatment may not reach the core of a timber and if new cuts or holes are created during construction (after treatment in a facility) the new holes may expose untreated areas. Prefabrication is ideal. Clause 9.17.4 and 9.17.7 of CAN/CSA S6 specify that if field treatment of cuts or holes is required, only creosote or copper naphthenate are acceptable (except that creosote is not permitted under the PMRA).

Wood piles also require preservative treatment in locations where they are exposed to air and above permanent water levels as required in Clause 9.17.8. Untreated wood piles are allowed if the entire pile is located "below a known permanent water level" in accordance with Clause 9.17.9. The commentary notes that in areas with flowing water, untreated piles may encounter decay.

Finally, preservative treatments are required for timber bridge decks. Stress-laminated timber decks require oil-borne preservatives (creosote, pentachlorophenol or copper naphthenate). Water-borne preservatives are allowed but due to concerns with dimensional stability, a sealer is noted in Clause 9.17.12 – designers require more details on the type and location and timing for applying the sealer and are not provided in the CHBDC standard or its Commentary. Dimensional stability is important for stress-laminated timber decks since the prestress in the deck will change as the laminations expand and contract with changes in moisture content and this could affect the performance of the deck.

This is a broad summary of requirements for preservatives. Other preservatives are available such as Hi-Clear II (Permethrin) which is mineral-spirit based and other water-borne preservatives, but no mention of those is made in the standard. In addition, naturally durable species are available which do not require preservative treatment, although the standard does not recognize these species for structural lumber. Additional notes about pressure treatment can be found in Section 4 of Timber Bridges in Ontario: Reference Guide.

Replaceable elements

Inspection and maintenance are normal parts of the life-cycle routine of bridges. For timber bridges, the deck is likely to require repair and/or replacement during the 75-year service life. The commentary on Chapter 2 of the standard, Table C2.1 provides anticipated service life of components. Exposed timber wearing surfaces have an expected 5- to 10-year service life. Asphaltic decks have a 15- to 20-year service life. Clause 9.24 specifies the asphalt wearing course required on timber bridge decks. Nail-laminated decks with asphalt require welded-wire fabric mesh in the asphalt to prevent the asphalt from breaking apart (see Commentary C9.24). Concrete-wood composite decks do provide protection to the wood and may not require additional asphalt; however, they may be susceptible to road salts. The timber components may not be affected by salts, but fasteners and reinforcement are. See Section 1.4 of Ontario Wood Bridge Reference Guide for more information about replaceable elements.

2.2.9.9. CLAUSE 9.21 NAIL-LAMINATED WOOD DECKS

Section 5 of CAN/CSA S6 provides analysis methods for longitudinal nail-laminated decks. However, Clause 9.21.3 limits the use of nail-laminated decks to only those cases where a composite wood-concrete deck is used to minimize loosening of nails which can lead to delamination and compromise the load sharing capacity of the deck. It also states that an "Approved alternative method of providing load sharing among the laminates is used," i.e. stress-laminated decks per Clause 9.22.

2.2.10. CAN/CSA S6 – SECTION 10 – STEEL STRUCTURES

This section has no requirements that are specific to timber bridges. Clause 10.10.1.3 notes that wood decks are not considered to provide lateral support to steel girders "unless the design and fastenings are designed for this purpose."

2.2.11. CAN/CSA S6 – SECTION 11 – JOINTS AND BEARINGS

This section has no requirements that are specific to timber bridges.

2.2.12. CAN/CSA S6 – SECTION 12 – BARRIERS AND HIGHWAY ACCESSORY SUPPORTS

This section has no requirements that are specific to timber bridges.

2.2.13. CAN/CSA S6 – SECTION 13 – MOVABLE BRIDGES

This section has no significant requirements related timber bridges.

2.2.14. CAN/CSA S6 – SECTION 14 – EVALUATION

Section 14 specifies requirements for the evaluation of existing wood bridges to assess the capacity for carrying a specified load. Clause 14.14.1.7 is specific to the evaluation of wood members based on site measurements and calculation and is to be used in conjunction with Section 9 – Wood Structures.

2.2.15. CAN/CSA S6 – SECTION 15 – REHABILITATION AND REPAIR

Section 15 is for all types of bridge materials. Clause 15.8.1.2 notes that wood members may be strengthened with fibre-reinforced polymer in accordance with Section 16.

2.2.16. CAN/CSA S6 – SECTION 16 – FIBRE-REINFORCED STRUCTURES

Stress-laminated wood decks may be fibre reinforced. Table 16.1 permits the use of fibre reinforcement using aramid fibre-reinforced polymer (AFRP), glass fibre-reinforced polymer GFRP and Aramid rope. Carbon fibre-reinforced polymer tendons, or CFRP, are not permitted, as noted in the Commentary to Table 16.1, because their high modulus of elasticity will result in "large loss of prestress due to the creep of wood."

Clause 16.9 provides the requirements for FRP post-tensioning of stressed wood decks. Stressed log bridge decks are also included in this category.

2.2.17. CAN/CSA S6 – SECTION 17 – ALUMINUM STRUCTURES

This section has no requirements that are specific to timber bridges. Clause 17.6.1(a) specifies that corrosion protection is required between aluminum and wood components.

2.3. EDUCATION – DOCUMENTS/RESOURCES FOR DESIGNERS

The primary resources for wood bridge designers have changed over the years. The main resource for many years was Modern Timber Bridges: Some Standards and Details, published by the Canadian Institute of Timber Construction (CITC) from Ottawa. This publication included information on materials (solid sawn and glulam), preservatives, and installation and had sample drawings for bridges spanning between 15-feet and 30-feet. The CITC maintained the document up to a third edition in 1970. The document referred to the CSA Standard S6, Design of Highway Bridges for loading requirements.

In 1979 the Ontario Highway Bridge Design Code (OHDBC) was published using the limit states design approach and covered topics on wood bridges. By 1988 the standard CAN/CSA-S6-88 Design of Highway Bridges for users across the country largely adopted parts of the OHBDC. With new research and practice in wood highway bridges, technologies such as laminated wood bridge decks and other wood products and assemblies became available but were not reflected in these standards until the 1983 edition of the OHBDC. After the publication of the 1991 OHBDC, the Canadian Wood Council published the book Wood Highway Bridges in 1992 to provide a simplified guide for designers that could be used alongside the bridge code. Subsequent codes effectively replaced the OHBDC with a single nationwide standard, published by CSA called the Canadian Highway Bridge Design Code (CHBDC) with editions in 2000, 2006 and 2014 (Taylor, Kennan, 1992).

The US Department of Agriculture Forest Service published Timber Bridges: Design, Construction, Inspection and Maintenance in 1990 with over 900 pages of information covering types of timber bridges, wood properties and preservatives, and design information per US standards.

Learning opportunities for practitioners interested in wood bridges are not often available. The Canadian Society for Civil Engineering (CSCE) hosts the Short and Medium Span Bridges Conference every few years since 1982 with presenters primarily from Canada and some international representation. There are typically a small number papers related to advances in timber bridges.

The International Conference on Timber Bridges (ICTB) was held for the first time in Lillehammer, Norway in 2010. The second conference was in 2013 in Las Vegas, and the most recent in Sweden in June 2017. This is the only international event focussed primarily on timber bridges.

Other conferences where some timber bridge related presentations have been made are the bi-annual World Conference on Timber Engineering (next conference will be in Seoul Korea 2018) and the annual Holzbau Forum in Germany.

In 2016 WoodWorks offered a seminar titled "Advanced Timber Bridge Maintenance, Restoration and Inspection Practices" targeted at inspectors, maintenance workers and engineers.

2.4. COSTS

The focus of this report has been on obstacles that may slow design or prevent designers from working with timber bridges. And while cost of bridges may appear to be an issue related to decision-makers at the policy level, an important decision-making tool for designers is costing during the design process. When choosing between systems, or elements in a structure, some understanding of costs can help with design decisions. For example, in building design, if a designer knows that solid sawn dimension lumber is cheaper than structural composite lumber (SCL) or glulam lumber, then the designer will work towards a solid sawn lumber solution before considering the SCL. In bridge design, a similar approach will help the designer make choices, but they require cost information for systems where the relative costs of systems are not readily understood. More information about costs of timber bridges can be found in Section 1.5.1 of Ontario Wood Bridge Reference Guide.

2.5. PERCEPTION

Some common themes regarding potential benefits and obstacles of timber bridges for designers and society are presented briefly.

Design examples to current standards

A design example of a bridge to the current CAN/CSA S6 would be helpful to provide more confidence for designers with little or no experience with timber bridges. An example of a glulam girder splice would be helpful to educate practitioners.

University Courses and Seminars

Lack of knowledge and experience is a big deterrent from designing and building wood bridges not only in Ontario but across Canada. Most Canadian universities do not offer a structural analysis course in wood (at most it may be a sub-topic in another course), not to mention wood bridge design. Topics on detailing, design for shrinkage, expansion are important and not clearly understood for wood. (Topics in shrinkage control and design for temperature changes are normally covered in courses and books for concrete and steel design.)

Aesthetics

Quite often, timber bridges are low profile and drivers are unaware that the structure beneath them is wood. But there are landmark examples of timber bridges in Canada which have their own aesthetic and help to define a sense of place. Whether architects are involved or not in bridge design, exposed wood is often very visually pleasing. Wood bridges are an opportunity for great structural and architectural expression.

New Concepts in Timber Bridge Design

Wood-concrete composite decks for longer spans have been built in Europe. The current CAN/CSA S6 provisions for wood-concrete composite decks are capable of only limited spans. Relatively new products and the use of prestressing may help timber bridges reach spans in the 30 m range.

Prefabrication and offsite construction

The overall dead load of timber bridges is generally less than the design of concrete or steel bridges. This makes erection easier and potentially quicker, lessens the lift load for temporary jack supports, reduces the seismic load and substructure connections/foundations. If smaller, lightweight timber bridges are developed, they could offer an opportunity for fast deployment and minimal disruption to traffic.

Long-term creep

Creep of beams and creep perpendicular-to-grain have been well researched. As noted above, glulam beams can be cambered to twice the calculated deflection due to dead loads to account for long-term creep. And creep perpendicular-to-grain can be a problem for stress-laminated bridge decks, however, this is resolved by the restressing at the intervals specified in Clause 9.23.3.4.

More recently, interest in longitudinally prestressed (post-tensioning) girders has revealed a new area where more research may be required (Lehan, 2012) to predict and deal with creep parallel-to-grain on long girder spans.

Bridge Type Selection and Spans

Assistance in determining appropriate spans and highway classes for various types of wood bridges would help designers. If there are regional differences between jurisdictions, this would be useful to know.

Wood bridges are generally used for shorter clear spans than concrete or steel. Solid sawn spans tend to be up to 7.5 m, and glulam 7.5 m – 15 m (Leslie, 2012). Wood trusses may be longer. New technologies with composite wood-concrete decks could be in the range of 30 m. (Lehan, 2012).

Lack of software

This may not be of great concern but software for steel and concrete bridges is available. Automating the application of moving loads and designing timber members for the complex shear calculation may be of use to designers. Most software packages do not have Canadian standards in them. One that does and can be used for bridges to CAN/CSA S6 is limited to designing wood decks on steel girders.

Inspection and maintenance

The Ontario Structure Inspection Manual (OSIM) has detailed information on procedures and measurements for existing timber bridges. This may be a useful training tool for learning about successful details in bridge design. It should also be reviewed periodically to ensure it reflects current knowledge and practice.

2.6. SUMMARY AND FUTURE RESEARCH

This report focussed on existing technologies currently addressed by the standard. However, there are many new and novel technologies being developed and tested here in Canada and around the world that will result in more potential options for bridge designers. The priority should be to address the concerns related to the current standard (some of which will require research) and then look for opportunities to evolve the standard to make it easier for the new technologies, once proven, to be adopted. Educational events related to the new technologies may be a useful vehicle for generating interest and promoting the opportunity for wood in highway bridge design.

2.7. SYMBOLS

ACQ   =   Alkaline copper quaternary

ACZA =   Ammoniacal copper zinc arsenate

AFRP =   Aramid fibre-reinforced polymer

CA-B =   Copper azole type B

CCA   =   Chromated copper arsenate

CFRP =   Carbon fibre-reinforced polymer

C_K =   Lateral Stability Factor variable for glulam compression resistance

CO_2 =   Carbon Dioxide

C_v =   Coefficient of shear load

D_T =   Truck load distribution width (in metres)

E   =   Modulus of Elasticity

FLS   =   Fatigue Limit State

FRP   =   Fibre reinforced polymer

GFRP =   Glass fibre-reinforced polymer

F_T =   Truck load fraction

LVL   =   Laminated veneer lumber

M_r =   Moment resistance

MTO   =   Ministry of Transportation

PMRA=   Health Canada Pest Management Regulatory Authority

PSL   =   Parallel strand lumber

SLS   =   Serviceability Limit State

ULS   =   Ultimate Limit State

2.8. REFERENCES

1. AASHTO LRFD bridge design specifications, customary U.S. Units (2012). Washington, DC: American Association of State Highway and Transportation Officials.

2. Behr, R. A., Cundy, E. J., & Goodspeed, C. H. (1990). Cost comparison of timber, steel, and Prestressed concrete bridges. Journal of Structural Engineering, 116(12), 3448–3457.

3. Canadian Institute of Timber Construction (1970). Modern Timber Bridges Some Standards and Details (Third ed.). Ottawa, Ontario: Canadian Institute of Timber Construction.

4. CSA (2014) CSA O86-14 Engineering Design in Wood. Mississauga: Canadian Standards Association.

5. CSA (2006) CAN/CSA-S6-06 Canadian Highway Bridge Design Code. Mississauga: Canadian Standards Association.

6. CSA (2006). S6.1-06 Commentary on CAN/CSA-S6-06, Canadian Highway Bridge Design Code. Mississauga: Canadian Standards Association.

7. Eurocode 5: Design of timber structures – Part 2: Bridges (2004). European Committee for Standardization.

8. Lefebvre, D., & Richard, G. (2014). Design and construction of a 160-Metre-Long Wood Bridge in Mistissini, Quebec.

9. Lehan, A. (2012). Development of a Slab-on-Girder Wood-Concrete Composite Highway Bridge

10. Leslie, M. (2012, May). Opportunities for Timber Bridges. Toronto.

11. Mermigas, K. (2017, January). Personal Communication with Head of Bridge Management for MTO.

12. Mohammad, M., & Natural Resources Canada. (2014, September). Innovative Timber Bridges: Technical Tour of Norwegian Timber Bridges. FP Innovations.

13. Morris, P., & FPInnovations. Durability of Wood Bridges

14. Morris, P., Moses, D., & Lebow, S. (2009). Specifying Alternatives to Conventional Treatments. Wood Design and Building Magazine.

15. Ontario Structure Inspection Manual: OSIM (2000). St. Catharines, ON: Ontario Ministry of Transportation, Bridge Office.

16. Ritter, M.A., Duwadi, S.R., Lee, P.D.H., ed(s). (1996). National conference on wood transportation structures. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. Tech. Rep. FPL- GTR-94. Madison, WI.

17. Sarisley, E. F. (1990). Construction methods and costs of Stress Laminated timber bridges. Journal of Construction Engineering and Management, 116(3), 432–447.

18. Taylor, R. J., & Keenan, F. J. (1992). Wood Highway Bridges. Canadian Wood Council.

19. Transportation Association of Canada TAC (2015). Primer on The Sustainability Considerations for Bridges.

Part Three:
Design Examples

3.1. INTRODUCTION

Two worked design examples of wood highway bridges are provided as follows:

1. A glued-laminated timber deck panels supported by glued-laminated timber girders, and
2. A longitudinal stress-laminated timber deck supported by glued-laminated timber girders.

The bridges are for illustrative purposes only and have not been constructed, but the design process presented herein is analogous to the design effort required to engineer such structures. While a specific structural system has been chosen for presentation in this example, the design philosophies may be extended to other wood highway bridges. Note that the design examples are written as stand-alone examples, so some information is repeated intentionally for the convenience of readers.

The design complies with the design criteria established by the CAN/CSA-S6-14 Canadian Highway Bridge Design Code (CHBDC). The 2019 version of this code was shortly forthcoming at the time of writing of this design example. As such, efforts were made to use the proposed updated equations, nomenclature, and clauses of the new code, to ensure compatibility for future designers. As referenced by the CHBDC, the connection design complies with CAN/CSA-O86-14 Engineering Design in Wood (CSA O86).

Some components of the example bridge concept are not currently well-covered by the CHBDC or CSA O86. Accordingly, other codes, standards, and publications were utilized, where necessary, to develop the design criteria presented herein. Refer to the references section for a complete list of these documents. It is recommended that the designer review and understand these documents before undertaking a real-life design.

These design examples include the design of superstructure elements, including structural loading, structural analysis, member design, connection design, and detailing for durability. They do not include substructure design.

The terms "vertical", "transverse", and "longitudinal" are used throughout this design example to indicate direction. Unless otherwise specified, these terms refer to the global directions of the bridge. "Vertical" refers to the direction parallel to gravity. As a subset of "vertical", "uplift" refers to the direction parallel to but in the opposite sense of gravity, while "downward" refers to the direction parallel to and in the direction of gravity. "Transverse" refers to the horizontal direction acting perpendicular to the direction of traffic. Finally, "longitudinal" refers to the horizontal direction acting parallel to the direction of traffic.

3.2. GLUED-LAMINATED TIMBER DECK PANELS ON GLUED-LAMINATED TIMBER GIRDERS

3.2.1. THE BRIDGE CONCEPT

3.2.1.1. DESCRIPTION

The bridge is a single-span, wood highway bridge carrying a two-lane highway 18 m across a stream. The bridge follows a tangent horizontal alignment and a 1.50% tangent vertical profile. The centreline of the highway is collinear with the centreline of the structure. Figures 3.2.1. and 3.2.2. illustrate an elevation view and the profile of the structure.
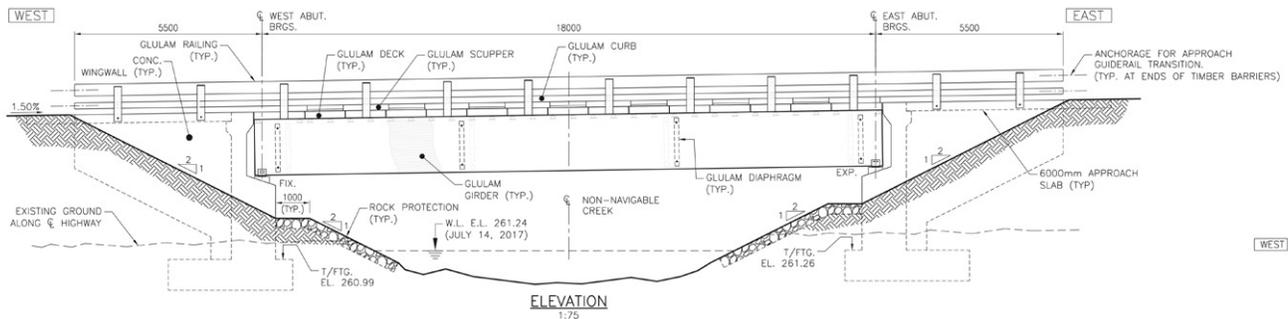
Figure 3.2.1. – Elevation of the structure
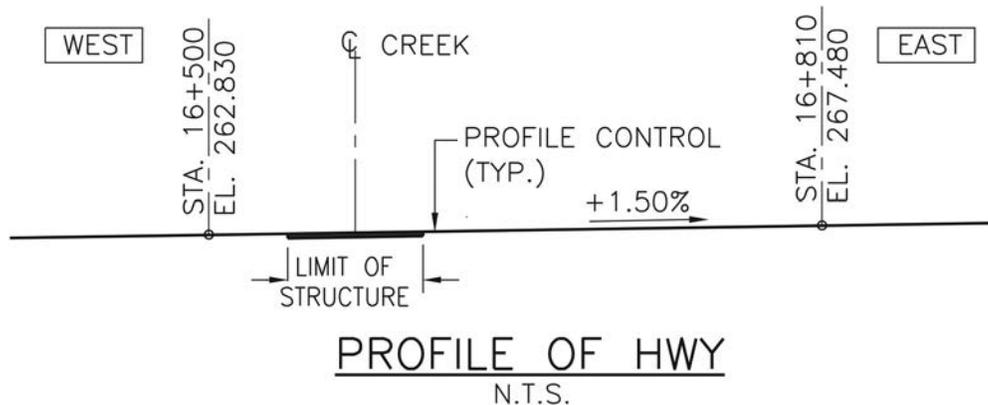
Figure 3.2.2. – Profile of the structure

The roadway cross-section was developed using the "Geometric Design Standard for Ontario Highways" document with 2002 revisions (Ontario Ministry of Transportation (MTO) 2002). It is valid for an undivided arterial road with a design speed limit of 110 km/h or less. This criterion describes the Trans-Canada Highway in many parts of the country. Figure 3.2.3. illustrates a typical cross-section of the roadway and structure.
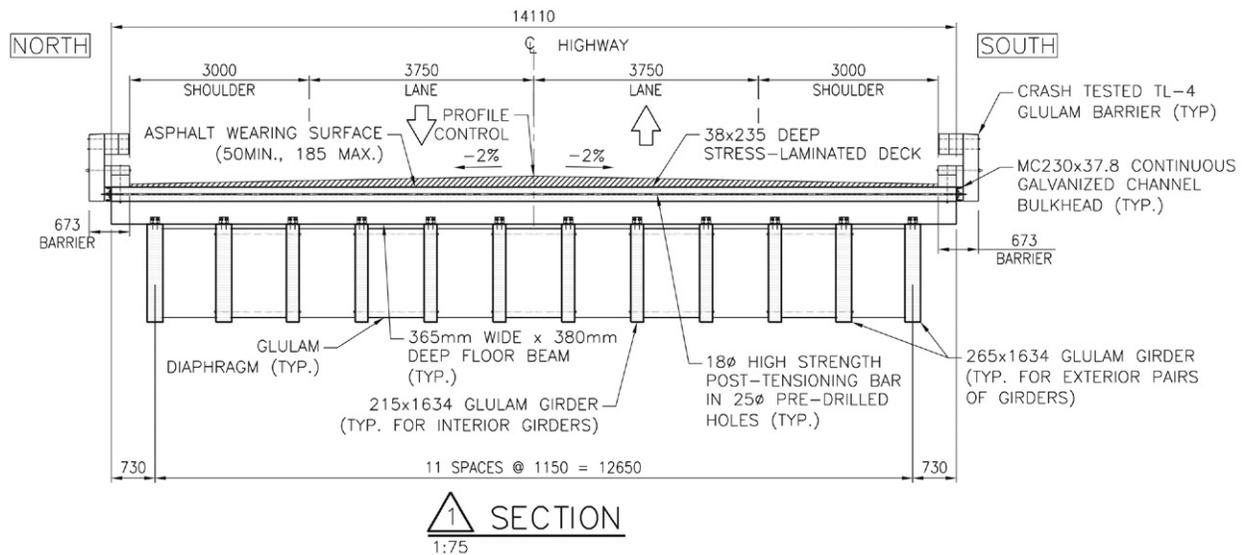
Figure 3.2.3. – Cross-section of the structure

The cross-section consists of two 3 750 mm-wide lanes, two 3 000 mm-wide shoulders, and two 305 mm-wide glued-laminated curbs. The resulting total deck width is 14 110 mm, and the total travelled deck width is 13 500 mm. Highway shoulders are sometimes reduced at a bridge to reduce the bridge width, but this approach is often not preferable from a safety standpoint. The use of the 3 000 mm-wide shoulders in this design example demonstrates that using a wood bridge does not have to result in sacrifices to highway geometry.

A crash-tested TL-4 timber railing is utilized as a bridge barrier to prevent traffic from exiting off the sides of the bridge. This railing was developed and crash-tested in the United States by the Forest Products Laboratory, specifically for transverse glued-laminated deck panels (Polivka, K.A. et al. 2002, Fuller, R.K. 2000). The railing consists of a continuous upper rail, a continuous curb rail, vertical posts spaced at no more than 2 438 mm (8'-0") on centre, a discrete length spacer block at each post, a discrete length scupper block at each post, and metal fasteners and splices. All wood components are glued-laminated members. Figure 3.2.4. illustrates a typical cross-section of the railing at a post.
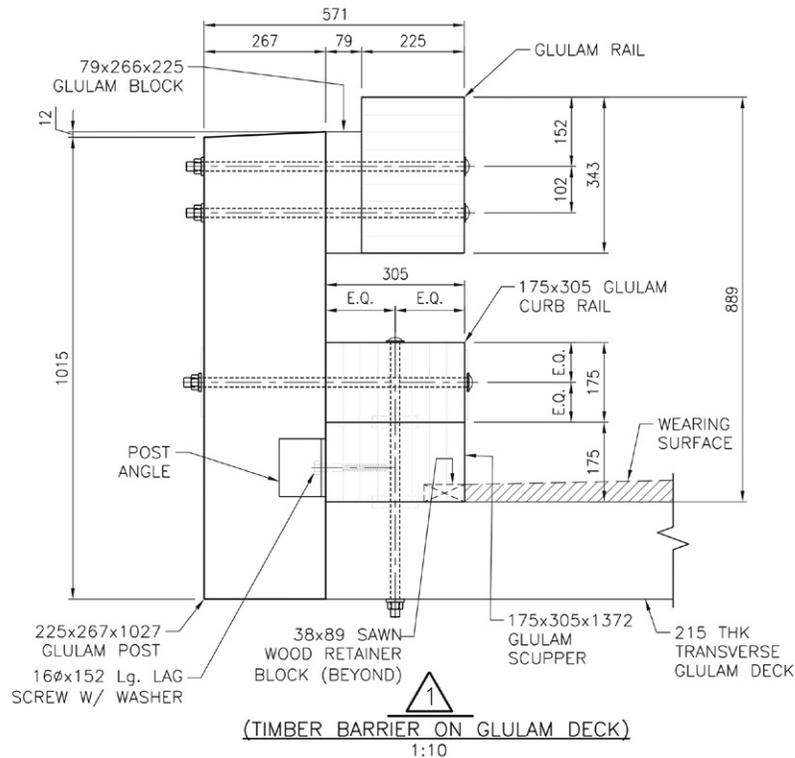
Figure 3.2.4. – Cross-section of the railing

The bridge has an asphalt wearing surface with a 2% crossfall down to the curbs from the crown. The crown is located at the centreline of the structure. The asphalt thickness varies from 185 mm at the crown to 50 mm at the inside face of the curbs.

Bridge deck waterproofing is not specified in this design example, but it should be part of a real-life wood bridge design. There are several approaches by which the deck can be waterproofed and paved. Refer to the work of Eriksson et al. (2003) and Weyers et al. (2001) for a comprehensive examination of waterproofing and paving systems for wood bridges. Note that the wood preservative used to treat the deck must be compatible with the waterproofing and pavement. There have been several instances where interaction between the two has resulted in degradation of the waterproofing and paving, as well as leaching into the surrounding environment.

The deck consists of 15, full-width, 215 mm-thick, interconnected, transverse glued-laminated deck panels. These panels are essentially glued-laminated beams turned on their sides. The panels are fastened to the girders using a combination of bolts, aluminum deck clips, and lag screws. The deck panels are detailed with a 10 mm-wide gap between them to allow for swelling of the panels due to moisture increase. The gap is to be filled with asphalt impregnated fibreboard to allow for paving overtop of the gaps. The fibreboard has relatively low compressive stiffness, thus allowing the panels to swell without developing significant force. Figures 3.2.5. illustrates the connection between the deck panels and girders. Figure 3.2.6. illustrates the joint detail between adjacent deck panels.
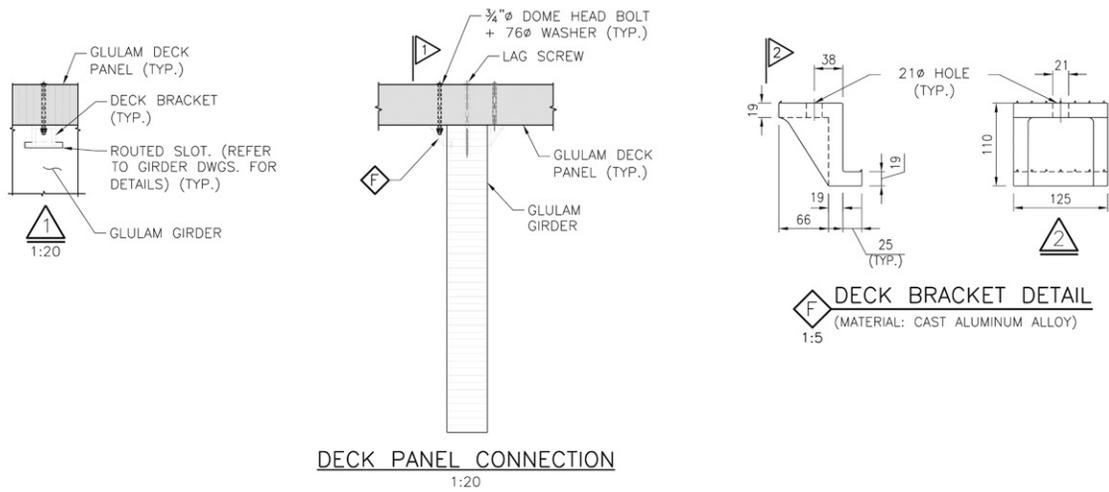
Figure 3.2.5. – Connection between deck panels and girders
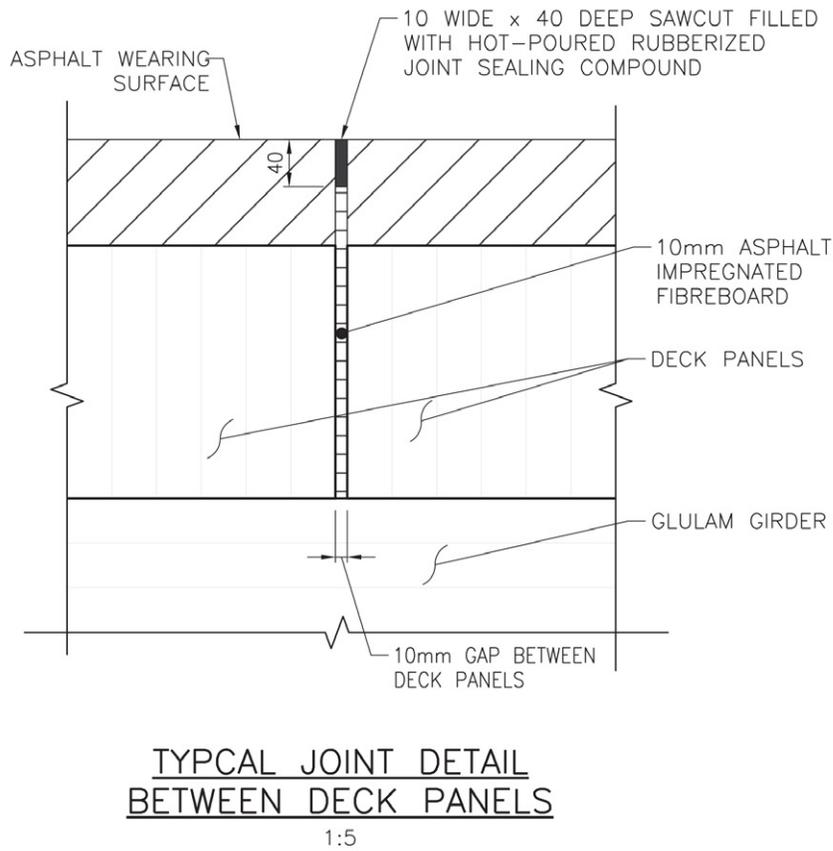
Figure 3.2.6. – Joint detail between adjacent deck panels

Longitudinal stiffener beams are fastened to the underside of the deck panels using through-bolts. The purpose of the stiffener beams is to provide a degree of longitudinal continuity between the panels, as a sort of external dowel. The use of stiffener beams has been noted to significantly reduce the likelihood of reflective asphalt cracking at the joints between panels (Eriksson et al. 2003, Witmer et al. 2002). Each stiffener beam extends from one end of the bridge to the other. This configuration, as opposed to numerous discrete stiffener beams at each joint between deck panels, allows for easy erection of the stiffener beams. A single stiffener beam can be rested on top of the diaphragms before the deck panels are installed. After the panels have been installed, the stiffener beam can then be lifted off the diaphragms and into place as a single unit. The consequence of using a full-length stiffener beam is that the bolt holes in the stiffener beams for its connection to the deck panels must be slotted in the longitudinal direction to accommodate the swelling and/or shrinkage of the deck panels perpendicular to grain. Failing to slot these holes will result in significant restraint forces in the stiffener beams and deck panels that could fail these members. Figure 3.2.7. illustrates detail of a typical deck stiffener beam.
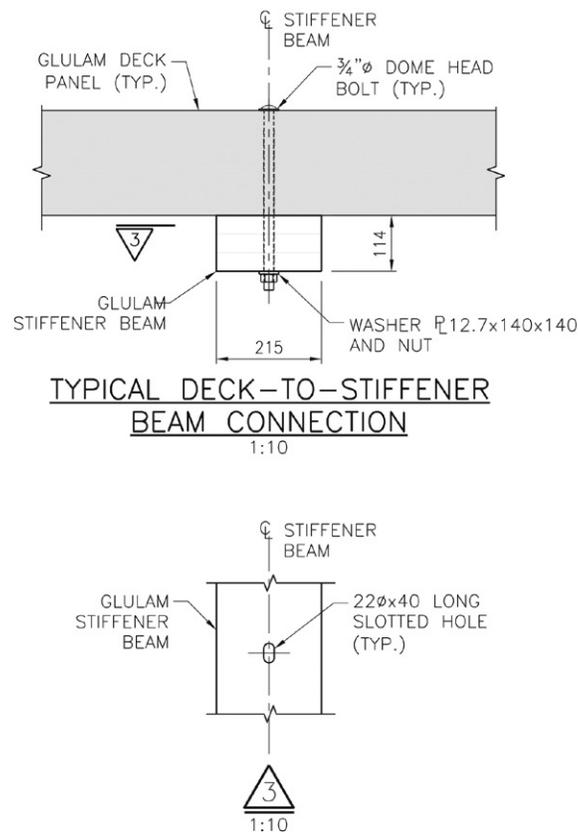
Figure 3.2.7. – Details of a typical deck stiffener beam

The deck is supported by 12, parallel, constant-depth, S-P (Spruce-Lodgepole Pine-Jack Pine) glued-laminated girders. Each girder is 215 mm wide and 1 634 mm deep. Glued-laminated diaphragms positioned at the span quarter-points are used to stabilize the girders against lateral-torsional buckling and to transmit transverse loads between girders. Figure 3.2.8. illustrates a typical cross-section at a diaphragm.
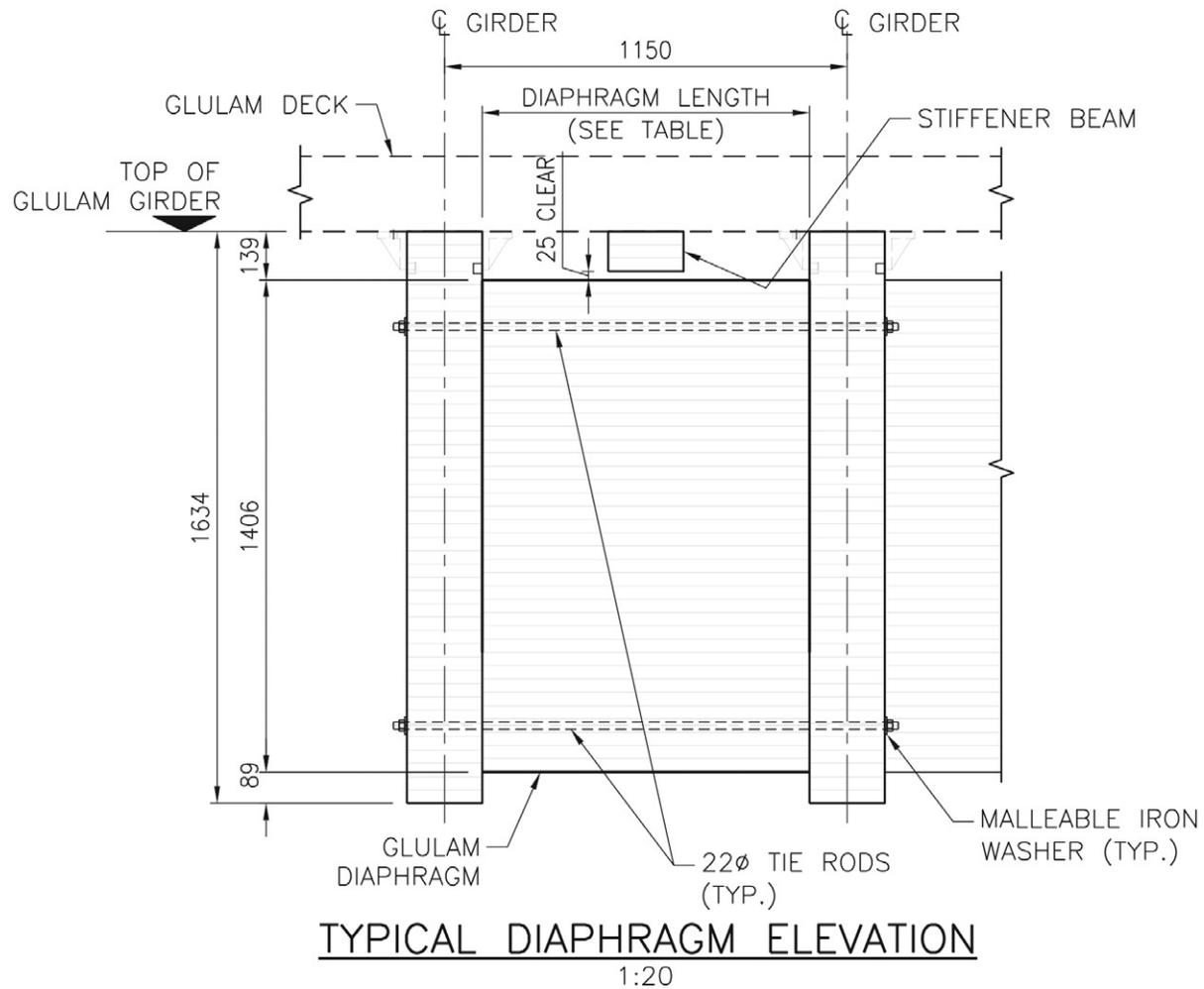
Figure 3.2.8. – Typical cross-section at diaphragm

Each girder is supported at each end by an elastomeric bearing pad. These bearing pads are part of a bearing assembly that also consists of a steel bearing plate, a bed of non-shrink grout, and the reinforced concrete bearing pedestal that projects above the abutment bearing seat. Figure 3.2.9. illustrates a typical bearing assembly.
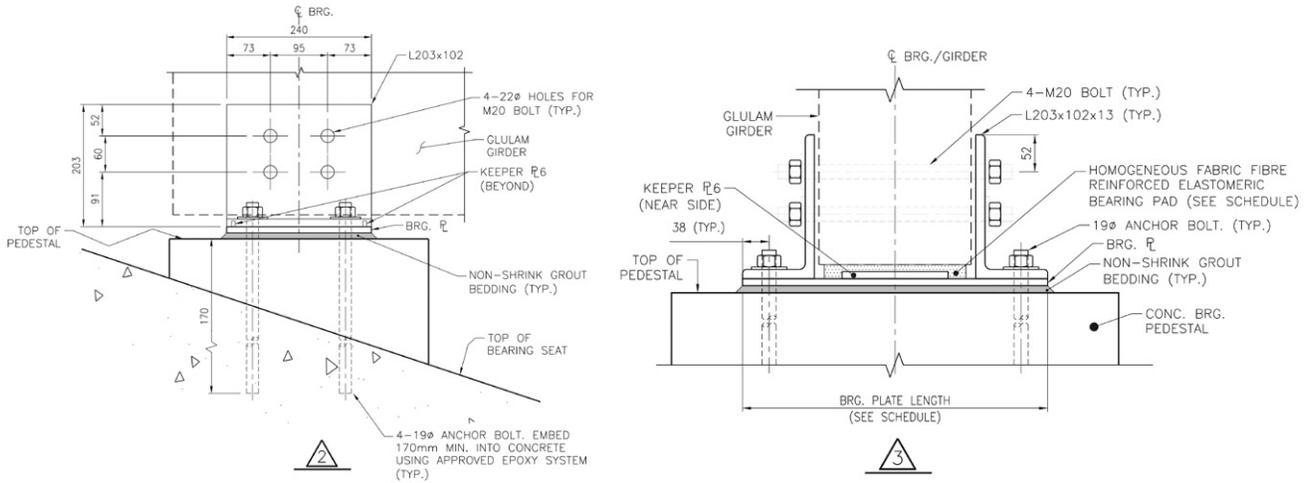
Figure 3.2.9. – Typical bearing assembly

The substructure has reinforced concrete abutments founded on spread footings. There is no limit to the number of substructure and foundation combinations that are compatible with this example bridge concept. The only requirement is that they be capable of safely receiving the loads transmitted from the superstructure and detailed in a way that drains water, debris and snow buildup away from the wood elements.

3.2.1.2. ASSUMED STRUCTURAL BEHAVIOUR

The glued-laminated deck panels resist gravity loads by spanning in the transverse direction of the bridge between girders. They serve to transmit permanent and transitory gravity loads to the girders by means of out-of-plane shearing and bending. They also serve to transmit transverse and longitudinal loads to the girders through in-plane shearing, bending, and axial forces. Furthermore, the deck panels enable the transverse distribution of discrete transitory loads amongst the girders. The deck panels also resist the loads acting on the timber railings due to vehicular impact through a combination of in-plane tension and out-of-plane shearing and bending.

While the deck panel behaviour tends strongly towards that of a one-way system spanning between girders, there is a small amount of shear and moment acting in the longitudinal direction. The longitudinal stiffener beams transmit this shear and moment between adjacent deck panels, resulting in reduced differential deflections between adjacent panels. These diminished differential deflections enhance the durability of the bridge by mitigating cracks in the wearing surface that could create a pathway for moisture ingress into the superstructure.

The girders comprise the gravity load-carrying system in the longitudinal direction. They are assumed to act non-compositely with the deck panels, although some degree of composite action likely exists due to friction and the mechanical connections between the deck panels and girders. The girders carry vertical loads by in-plane shear and bending, transverse loads by out-of-plane shear and bending, and longitudinal loads by axial tension and compression.

The diaphragms are the assumed means of preventing twisting of the girders, although the deck panels also serve that purpose due to their inherent out-of-plane flexural stiffness. The unbraced length for calculating lateral-torsional buckling of the girders in the completed bridge is assumed to be the spacing between successive rows of diaphragms.

Wind acting on the deck panels, on the railings, and on live load is transmitted into the girders using a system of lag bolts and aluminum deck clips that are side mounted to the upper portions of the girders. The transverse wind force, in addition to transverse wind acting directly on the girders, is carried across the main span to the girder supports by means of the girders bending and shearing out-of-plane. The diaphragms serve to share these out-of-plane loads amongst all the girders.

Longitudinal braking loads are resisted by the deck panels. The asphalt impregnated fibreboard joint filler is assumed to be sufficiently soft to prevent the sharing of loads between panels, i.e. each deck panel must resist the braking loads that are applied directly to it. Lag screws passing through the deck panels and into the upper region of the girders are the assumed mechanism for transferring the braking loads from the deck panels into the girders. Friction between the deck panels and girders may transmit these forces, especially if it is enhanced by the clamping action of the lag bolts and aluminum deck clips in a manner analogous to interface shear transfer in concrete structures, but this mechanism is ignored in this example. The braking force that is transmitted to the girders is then transmitted to the bearings by means of axial tension and compression in the girders.

Loads are transmitted to the substructure via the elastomeric bearing pads at the abutments. The bearing pads serve to transfer vertical, transverse, and longitudinal loads to the abutments, while allowing for rotation, expansion, and contraction of the girders.

The steel bearing plates provide a clean surface onto which to seat the bearing pads. They can be beveled in thickness to suit the roadway profile, girder camber, and bridge self-weight rotations. These plates also allow for anchorage of the bearing assemblies into the abutment pedestals by use of anchor bolts. When fitted with keeper bars or drift pins, they also serve as a restraint against bearing pad migration due to transverse and longitudinal forces acting on lightly loaded bearing pads.

The concrete pedestals transfer the loads from the bearing assemblies down into the abutment stem. No matter the workmanship, there are always imperfections on hardened concrete surfaces. Accordingly, a thin bed of non-shrink grout is specified under the steel bearing plates to establish firm contact with the concrete bearing pedestals.

3.2.2. MATERIALS

All glued-laminated timber used in this design example is assumed to be species combination S-P. S-P dimension lumber is plentiful in many parts of the country, especially Ontario. The CHBDC does not presently provide material properties for the use of S-P glued-laminated timber with its design equations; however, the 2019 version of the CHBDC is anticipated to include these material properties. Accordingly, the glued-laminated timber material properties for this design example have been assumed from CSA O86 and modified, as necessary, to be compatible with the CHBDC design equations.

3.2.2.1. GLUED-LAMINATED TIMBER GIRDERS, STIFFENER BEAMS, & DIAPHRAGMS

The following material properties are from Table 7.3 of CAN/CSA-O86-14 Engineering Design in Wood. The girders, stiffener beams, and diaphragms are a horizontally laminated 20f-E stress grade comprised of Spruce-Lodgepole Pine-Jack Pine species laminations. The nominal values have not been adjusted by service condition or treatment factors, or for the effects of impact and cyclical loading.

f_b = 25.6 MPa bending moment (positive)

f_b = 19.2 MPa bending moment (negative)

f_v = 1.75 MPa longitudinal shear

f_c = 25.2 MPa compression parallel to grain

f_{cb} = 25.2 MPa compression parallel to grain combined with bending

f_{cp} = 5.8 MPa compression perpendicular to grain

f_{tn} = 17.0 MPa tension at net section

f_{tg} = 12.7 MPa tension at gross section

f_{tp} = 0.51 MPa tension perpendicular to grain

E_{50} = 10300 MPa 50th percentile modulus of elasticity

E_{05} = 8960 MPa 5th percentile modulus of elasticity

The 5th percentile modulus of elasticity is calculated as 87% of the 50th percentile modulus of elasticity for this design example per CSA O86.

The properties from CSA O86 must be modified to reflect the service condition and preservative treatment of the girders to be compatible with the CHBDC design equations. A "semi-wet" service condition is assumed, acknowledging that the girders, stiffener beams, and diaphragms should not gain significant moisture in service due to their protection by the deck panels. The girders will be pressure-treated with a wood preservative to enhance their durability. They will be incised to increase the depth of preservative penetration.

The following are service condition factors for glued-laminated timber based on "semi-wet" conditions.

K_{Sb} = 0.90   service condition factor for bending

K_{Sv} = 0.94   service condition factor for shear

K_{Sc} = 0.88   service condition factor for compression parallel to grain

K_{Scp} = 0.84   service condition factor for compression perpendicular to grain

K_{St} = 0.88   service condition factor for tension parallel to grain

K_{Stp} = 0.93   service condition factor for tension perpendicular to grain

K_{SE} = 0.95   service condition factor for modulus of elasticity

Note that "wet" service factors may be appropriate for certain members or bridge configurations and the designer will need to make that judgement. CSA O86 provides service factors for "wet" conditions. In all cases, connections should be designed using "wet" service factors while members may be designed with either "semi-wet" or "wet" factors. Steel connection hardware tends to result in moisture condensation thereby increasing the moisture content of timbers in connections.

The following treatment factor for incised glued-laminated timber was taken from the proposed provisions for the forthcoming 2019 edition of the CHBDC.

K_T = 1.00   modification factor for treatment

The modification factor for treatment has been taken to be equal to unity, as incising is not considered to reduce the strength of glued-laminated timber (APA 2013) but should be confirmed by the designer for specific treatment applications.

The modified material properties that can be used directly with the CHBDC equations for design, accounting for service condition and preservative treatment, are as follows:

f_b	=	23.0	MPa	bending moment (positive)
f_b	=	17.3	MPa	bending moment (negative)
f_v	=	1.65	MPa	longitudinal shear
f_c	=	22.2	MPa	compression parallel to grain
f_{cb}	=	22.2	MPa	compression parallel to grain combined with bending
f_{cp}	=	4.9	MPa	compression perpendicular to grain
f_{tn}	=	15.0	MPa	tension at net section
f_{tg}	=	11.2	MPa	tension at gross section
f_{tp}	=	0.47	MPa	tension perpendicular to grain
E_{50}	=	9785	MPa	50th percentile modulus of elasticity
E_{05}	=	8512	MPa	5th percentile modulus of elasticity

3.2.2.2. GLUED-LAMINATED TIMBER DECK

The deck panels are vertically laminated glued-laminated beams; that is, a glued-laminated beam loaded perpendicular to the narrow face of the laminations. Clause 9.12.3 of the CHBDC indicates that "vertically laminated beams are to be designed as a built-up system of sawn lumber members of No. 2 grade, unless a higher grade is specified for all laminations". Accordingly, assume that the deck panels are similar to transverse nail-laminated decks for the purpose of calculating the load-sharing factor. Assume that the deck material properties are for S-P-F No. 2 grade structural joist and plank sawn wood members, with material properties from Table 9.12 of the CHBDC.

f_b	=	8.4	MPa	bending moment
f_v	=	1.1	MPa	longitudinal shear
f_c	=	6.7	MPa	compression parallel to grain
f_{cp}	=	3.0	MPa	compression perpendicular to grain
f_t	=	3.9	MPa	tension at net section
E_{50}	=	8500	MPa	50th percentile modulus of elasticity
E_{05}	=	5800	MPa	5th percentile modulus of elasticity

These deck panel material properties have been adjusted for service condition and treatment. The CHBDC assumes a wet service condition, which is appropriate for a wood bridge deck. It also assumes that the wood has undergone preservative treatment and has been incised. Both of these assumptions are appropriate, as clause 9.17.1 of the CHBDC requires all wood in permanent structures to be preservative treated in accordance with the CSA O80 Series of Standards, while clause 9.17.3 of the CHBDC requires all glued-laminated timber members to be incised before preservative treatment.

3.2.2.3. MATERIAL PROPERTIES FOR COMPUTER STRUCTURAL ANALYSIS

Material properties of the glued-laminated deck panels and girders are provided in the previous sections of this design example. Additional orthotropic material properties are required for three-dimensional computer modelling and structural analysis of wood structures. These properties were estimated from Table 5-1 of the Wood Handbook – Wood as an Engineering Material (FPL 2010) for Lodgepole Pine as follows:

E_T / E_L = 0.068   ratio of tangential elastic modulus to the longitu elastic modulus

E_R / E_L = 0.102   ratio of radial elastic modulus to the longitudina elastic modulus

G_{LR} / E_L = 0.049   ratio of shear modulus in the longitudinal-radial plane to the longitudinal elastic modulus

G_{LT} / E_L = 0.046   ratio of shear modulus in the longitudinal-tange plane to the longitudinal elastic modulus

G_{RT} / E_L = 0.005   ratio of shear modulus in the radial-tangential p to the longitudinal elastic modulus

All properties are given for wood at a moisture content of 12%.

Based on Tables 5-1 and 5-2 of the Wood Handbook, the structural analysis properties for the glued-laminated <u>girders</u> are as follows:

E_L	=	9785	MPa	50th percentile modulus of elasticity parallel to grain
E_T	=	665	MPa	50th percentile modulus of elasticity perpendicular to grain in tangential direction
E_R	=	998	MPa	50th percentile modulus of elasticity perpendicular to grain in radial direction
G_{LR}	=	479	MPa	50th percentile shear modulus in longitudinal-radial plane
G_{LT}	=	450	MPa	50th percentile shear modulus in longitudinal-tangential plane
G_{RT}	=	49	MPa	50th percentile shear modulus in radial-tangential plane
μ_{LR}	=	0.316		Poisson's ratio in radial direction due to applied stress in longitudinal direction
μ_{LT}	=	0.347		Poisson's ratio in tangential direction due to applied stress in longitudinal direction
μ_{RT}	=	0.469		Poisson's ratio in tangential direction due to applied stress in radial direction
μ_{TR}	=	0.381		Poisson's ratio in radial direction due to applied stress in tangential direction
μ_{RL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in radial direction
μ_{TL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in tangential direction

Similarly, the structural analysis properties for the glued-laminated <u>deck</u> panels are as follows:

E_L	=	8500	MPa	50th percentile modulus of elasticity parallel to grain
E_T	=	578	MPa	50th percentile modulus of elasticity perpendicular to grain in tangential direction
E_R	=	867	MPa	50th percentile modulus of elasticity perpendicular to grain in radial direction
G_{LR}	=	417	MPa	50th percentile shear modulus in longitudinal-radial plane
G_{LT}	=	391	MPa	50th percentile shear modulus in longitudinal-tangential plane
G_{RT}	=	43	MPa	50th percentile shear modulus in radial-tangential plane
μ_{LR}	=	0.316		Poisson's ratio in radial direction due to applied stress in longitudinal direction
μ_{LT}	=	0.347		Poisson's ratio in tangential direction due to applied stress in longitudinal direction
μ_{RT}	=	0.469		Poisson's ratio in tangential direction due to applied stress in radial direction
μ_{TR}	=	0.381		Poisson's ratio in radial direction due to applied stress in tangential direction
μ_{RL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in radial direction
μ_{TL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in tangential direction

3.2.3. LOADS & IMPOSED DEFORMATIONS

3.2.3.1. SELF-WEIGHT

The unit weights of the deck, stiffener beams, girders, and diaphragms are taken as 6.0 kN/m³ for softwood, per Table 3.4 of the CHBDC. The following table summarizes the linear weight of these components:

Component	Width		Depth		Cross-Sectional Area		Linear Weight	
deck panel 'A'	1444	mm	215	mm	0.310	m²	1.86	kN/m
deck panel 'B'	1178	mm	215	mm	0.253	m²	1.52	kN/m
stiffener beam	215	mm	114	mm	0.025	m²	0.15	kN/m
girder	215	mm	1634	mm	0.351	m²	2.11	kN/m
diaphragm	130	mm	1406	mm	0.183	m²	1.10	kN/m

3.2.3.2. SUPERIMPOSED DEAD LOADS

3.2.3.2.1. WEARING SURFACE

The bridge will be paved with an asphalt wearing surface having a unit weight of 23.5 kN/m³. The asphalt varies from 185 mm thickness at the crown to 50 mm thickness at the face of the timber railings, resulting in a 2% crossfall. Recalling that the girder spacing is 1 150 mm, the average asphalt thickness for either of the two interior girders nearest the crown is

$$t_{ws,int} = \frac{(185\ mm - 1150\ mm\ \times 0.02) + 185\ mm}{2} = \frac{162\ mm + 185\ mm}{2} = 174\ mm$$

The deck panel overhang is 730 mm and the timber railing curbs are 305 mm wide, thus the average asphalt thickness for either of the two exterior girders is

$$t_{ws,ext} = \frac{(50\ mm + [730\ mm - 305\ mm + 0.5 \times 1150\ mm]\ \times 0.02) + 50\ mm}{2}$$
$$\times \frac{(730\ mm + 0.5 \times 1150\ mm - 305\ mm)}{730\ mm + 0.5 \times 1150\ mm} = \frac{70\ mm + 50\ mm}{2} \times 0.77 = 46\ mm$$

These two asphalt thicknesses are used to design the interior and exterior girders, respectively.

The deck panels are designed by applying the actual thickness of asphalt as trapezoidal distributed load.

3.2.3.2.2. BARRIERS

The bridge barrier is a crash-tested TL-4 timber railing. This barrier has an unfactored linear weight of 1.3 kN/m. The frontal area for horizontal wind on the barrier is 0.635 m²/m. The centroid of the frontal area is 594 mm above the mid-depth of the glued-laminated deck. The frontal area of the barriers that overlaps with the horizontal projection of the deck has been neglected from these calculations. This frontal area is considered to belong to the deck. Calculations concerning the barrier weight and frontal area are not included in this design example.

3.2.3.3. WIND

Both vertical and horizontal wind loads are assumed to act simultaneously, per clause 3.10.2.1 of the CHBDC. For this design example, it is assumed that the reference wind pressure for a one-in-fifty return period wind load is 465 Pa.

3.2.3.3.1. VERTICAL WIND

The vertical wind load is calculated in accordance with clause 3.10.2.3 of the CHBDC. The vertical wind load acting on the superstructure is as follows:

q_{50}	=	465	Pa	Hourly mean reference wind pressure for a 50-year return period
C_e	=	1.0		wind exposure coefficient
C_g	=	2.0		wind gust effect coefficient
C_v	=	1.0		vertical wind load coefficient
F_v	=	930	Pa	vertical wind load per unit exposed plan area

The vertical wind load is considered to act both upwards and downwards. Two vertical wind load applications are considered for both upward and downward wind:

• Uniform load acting over the entire bridge plan area
• Eccentric wind load with the centroid of the total wind load acting at the windward quarter-point

The eccentric wind load results in a wind pressure that varies linearly across the deck width. By geometry, the magnitude of the wind pressures at the windward and leeward sides of the deck for downward acting vertical wind, are 2.5 x F_v and −0.5 x F_v, respectively. For downward acting vertical wind, those magnitudes become −2.5 x F_v and 0.5 x F_v, respectively, at the windward and leeward sides of the deck.

3.2.3.3.2. HORIZONTAL WIND ON SUPERSTRUCTURE

The horizontal wind load acting on the superstructure was calculated in accordance with clause 3.10.2.2 of the CHBDC. The horizontal wind load acting on the superstructure is as follows:

q_{50} = 465 Pa Hourly mean reference wind pressure for a 50-year return period

C_e = 1.0 wind exposure coefficient

C_g = 2.0 wind gust effect coefficient

C_h = 2.0 horizontal wind load coefficient

F_h = 1860 Pa horizontal wind load per unit exposed frontal area

The horizontal wind load acts on the exposed frontal area of the structure, including the railings, deck panels, and girders. The exposed frontal area of the railings is the total area of railings above the top of the deck panels. Recall that this frontal area is equal to 0.635 m² per metre. The resulting unfactored uniformly distributed load due to wind acting on the railing is

$$P_{h,railing} = \frac{0.635 \, m^2}{m} \times 1.860 \, kPa = 1.12 \, kN/m$$

The bridge width is large enough such that the shielding factors in clause C3.10.2.2 of the CHBDC commentary do not apply for horizontal wind acting on the railings. Thus, horizontal wind load is considered to act on the exposed frontal area of each timber railing.

The exposed frontal area of the deck is the product of its width and thickness. The resulting unfactored uniformly distributed load due to wind acting on the deck is

$$P_{h,deck} = 0.215 \, m \times 1.860 \, kPa = 0.40 \, kN/m$$

As noted in clause C3.10.2.2 of the CHBDC commentary, most highway bridges, including slab-on-girder bridges, behave aerodynamically as single bodies. Consequently, it is only necessary to apply horizontal wind load to the exposed frontal area of the windward exterior girder. The resulting unfactored uniformly distributed load due to wind acting on an exterior girder is

$$P_{h,girder} = 1.634 \, m \times 1.860 \, kPa = 3.04 \, kN/m$$

3.2.3.3.3. HORIZONTAL WIND ON LIVE LOAD

The horizontal wind load acting on live load is calculated in accordance with clause 3.10.2.4 of the CHBDC. The calculation is the same as for the horizontal wind load acting on the superstructure, but the magnitude of the horizontal wind load coefficient, C_h, is reduced from 2.0 to 1.2, resulting in a horizontal wind load acting on live load equal to 1116 Pa.

The horizontal wind load acting on live load is assumed to act uniformly over a height of 3.0 m above the roadway surface along the length of the structure. The frontal area within that envelope that has already been considered in the calculation of the horizontal wind load acting on the superstructure (i.e. the timber railings) is neglected from the horizontal wind load acting on live load calculation. Consequently, the total exposed area for wind on live load, excluding the exposed frontal area of a timber railing is:

$$A_{fr,LL} = 3.0 \ m \ \times 1.0 \ m + 0.050 \ m - 0.635 \ m^2 = 2.415 \ m^2/m$$

The resulting unfactored uniformly distributed load due to wind acting on live load is

$$P_{h,LL} = \frac{2.415 \ m^2}{m} \times 1.116 \ kPa = 2.70 \ kN/m$$

3.2.3.4. LIVE LOAD

The bridge is subject to vertical and longitudinal live loads due to the weight of moving vehicles and braking forces, respectively.

3.2.3.4.1. VERTICAL LIVE LOAD

Per clause 3.8 of the CHBDC, the design of the bridge considers the vertical live load effects caused by each of the CL-625-ONT truck and the CL-625-ONT lane load. The CL-625-ONT truck is a five-axle truck of 18 m length, with a total weight of 625 kN. The CL-625-ONT lane load is a uniformly distributed load of 9 kN/m superimposed with 80% of the CL-625-ONT truck load. Figure 3.2.10. illustrates these loads as they are depicted in the CHBDC.
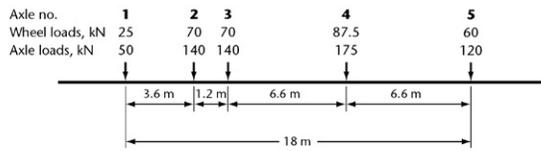
Figure A3.4.1
CL-625-ONT Truck
(See Clause A3.4.1.)
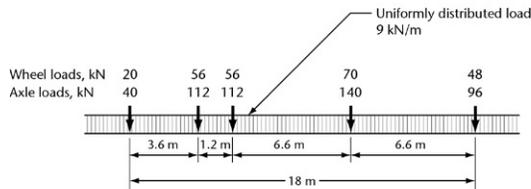
Figure A3.4.2
CL-625-ONT Lane load
(See Clause A3.4.1.)
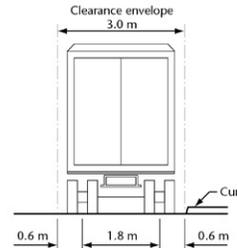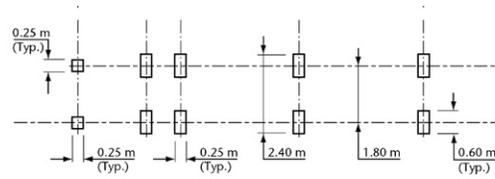
Figure 3.2
CL-W Truck
(See Clauses 3.8.3.1.2 and 3.8.4.3.)

Figure 3.2.10. – CHBDC vertical live load (Source: Clause A3.4.1, Figure A3.4.1 CL-625-ONT Truck; Clause A3.4.1, Figure A3.4.2 CL-625-ONT Lane Load; Clause 3.8.3.2., Figure 3.2 CL-W Truck – Canadian Highway Bridge Design Code. © 2017 Canadian Standards Association)

At the SLS 1 and ULS limit states, the vertical live load per design lane is the greater of the CL-625-ONT truck amplified by the dynamic load allowance or the CL-625-ONT lane load without the dynamic load allowance. At the SLS 2 and FLS limit states, the vertical live load for the entire bridge is the CL-625-ONT truck amplified by the dynamic load allowance and centred across the width of one design lane.

Truck axles and portions of the uniformly distributed lane load that reduce the load effect are neglected. The bridge is a single span in this design example, so no component of the vertical live loading will reduce the load effect.

The vertical live load is positioned both longitudinally along the length a design lane and transversely across the width of a design lane, to maximize the load effect. The transverse positioning of the truck across the width of a design lane respects the truck clearance envelope.

The vertical live load caused by the CL-625-ONT truck is increased by a factor equal to one plus the dynamic load allowance, per clause 3.8.4.5 of the CHBDC. The dynamic load allowance is a scalar factor that converts the dynamic effects of vertical live load to an equivalent static load. Dynamic effects are typically caused by the interaction of a moving vehicle and the bridge, static and vibratory deflections, and discrete and random irregularities in the riding surface (CSA 2014b).

Vertical Live Load	Dynamic Load Allowance
one axle of the CL-625-ONT truck	0.50
any two axles or axles 1-3 of the CL-625-ONT truck	0.40
any three of more axles of the CL-625-ONT truck, except for axles 1-3	0.30

The dynamic load allowance is reduced by 30% because the bridge is comprised entirely of wood components, per clause 3.8.4.5.4 of the CHBDC. Wood bridges are often short-span structures. This reduction accounts for the improved response of short-span bridges to dynamic loads. It also accounts for the inherent higher damping observed in wood relative to other conventional bridge materials (CSA 2014b).

The bridge has a total width of 14.110 m and a total travelled width of 13.500 m. The number of design lanes is as follows:

W_c = 13.500 m   total travelled bridge width

n = 2 or 3   lanes   number of design lanes per Table 3.5 of CHBDC

W_{e2} = 6.750   m   design lane width for two-lane configuration

W_{e3} = 4.500   m   design lane width for three-lane configuration

The travelled bridge width is such that both two and three design lanes must be considered. Each design lane configuration consists of one or more lanes loaded with a single CL-625-ONT truck or CL-625-ONT lane load. The modification factor for multi-lane loading, per Table 3.6 of the CHBDC, is considered for vertical live load to account for the unlikely presence of more than one design vehicle acting simultaneously on the bridge. Those factors are as follows for this design example:

Number of Loaded Design Lanes	Modification Factor
1	1.00
2	0.90
3	0.80

3.2.3.4.2. BRAKING FORCE

The braking force is calculated as the sum of 180 kN plus 10% of the uniformly distributed portion of the lane for one design lane. The bridge span is 18.000 m. The girders overhang the centreline of abutment bearings by 0.272 m, resulting in a total deck length of 18.544 m. The braking force is thus:

$$F_{br} = 180 \; kN + 0.10 \times 9 \frac{kN}{m} \times 18.544 \; m = 197 \; kN$$

For a bridge with a continuous concrete deck slab, the braking force would normally be distributed along the entire bridge length, as the concrete slab acts as a large in-plane diaphragm. This application would be incorrect for the bridge in this design example because the deck panels do not abut, and therefore cannot share longitudinal loads. As a result, a given deck panel must resist all the braking forces applied to it.

Per commentary clause C3.8.6 of the CHBDC, the 180 kN portion of the lane load is intended to represent heavy braking by a design vehicle, while the 10% of the lane load UDL is intended to capture lighter traction forces due to other traffic on the bridge. The 180 kN represents approximately 25% of the gross vehicle weight of two design trucks braking in two lanes simultaneously. This force has been modified to account for the lesser probability of simultaneous braking of design vehicles in multiple lanes. The force has also been modified to account for the difference in live load factors between the CHBDC and former Ontario Highway Bridge Design Code, the latter having originally developed the braking load formula (CSA 2014b). In light of this information, the braking force due to a wheel load is determined by multiplying the weight of a given wheel load by the ratio of 180 kN and twice the weight of a CL-625-ONT truck. The heaviest wheel load of the CL-625-ONT truck occurs at Axle 4, with a wheel load of 87.5 kN. The maximum discrete wheel braking load is thereby:

$$F_{br} = 87.5 \; kN \times \frac{180 \; kN}{2 \times 625 \; kN} = 87.5 \; kN \times 0.144 = 12.6 \; kN$$

Using this approach, the maximum discrete wheel braking load represents 14.4% of the gross wheel load. The deck panels are narrow enough that only a single axle would be effectively acting on any given panel at a time. The lane load component of the braking force can be ignored for the example bridge, as the 18 m bridge span length is only long enough to fit a single CL-625-ONT truck, without room for other traffic.

Thus, the design of the deck panels for braking considers the effect of 14.4% of the gross weight of axle 4 of the CL-625-ONT for a design truck in two lanes. The use of modification factors for multi-lane loading is not necessary, as these factors are already built into the braking force equation (CSA 2014b).

3.2.3.5. LOAD COMBINATIONS

The load factors and load combinations prescribed in clause 3.5 of the CHBDC are used for this design example, as applicable.

Exceptional loads are ignored in this design example, meaning that ULS load combinations 5 to 8 are not examined. The designer should consider these load combinations if exceptional loads are a possibility. ULS load combination 9 is also not considered because the wood bridge in this design example is very light relative to the weight of the live loads that it is designed for. The designer should consider these load combinations if the wood bridge happens to carry significant permanent load.

The fatigue limit state (FLS) is not considered in this design example because fatigue has traditionally not been considered for wood bridge design (Ritter 1992). Refer to the Wood Handbook – Wood as an Engineering Material (Forest Product Laboratory (FPL) 2010) for further information on fatigue in wood members. The FLS stresses in all metal connections should be checked in accordance with the CHBDC for real-life designs. These checks are not provided in this design example.

3.2.4. STRUCTURAL ANALYSIS

3.2.4.1. GLUED-LAMINATED TIMBER DECK PANELS

The distribution of discrete live loads acting across the width of the deck panels must be considered. It is not appropriate to simply assume that the full width of the deck panel is effective in providing resistance to live loads. Glued-laminated timber deck panels were developed at the United State Forest Products Laboratory in the 1970's. Research by McCutcheon and Tuomi (1973, 1974) yielded design equations that determine the live load bending moments and shears per metre width of deck panel. This research was substantiated by Bakht in 1988; however, Bakht observed that the equations are unconservative for live loads acting near the free edges of the panels. Bakht developed design curves to address this observation. Those curves were approximated into the equations presented in clause 5.7.3.2 of the CHBDC. Although that clause is explicitly for stress-laminated wood decks, the equations are based on the same work by Bakht (1988) and are applicable to glued-laminated timber decks. Instead of determining the live load bending moments and shears per metre panel width, Bakht's work provides the width of deck panel that is effective in resisting a transverse line of wheels.

The AASHTO LFRD Bridge Design Specifications (AASHTO) were updated in 1994 regarding glued-laminated timber deck panels. AASHTO provides equations to determine the width of deck panel that is effective in resisting a transverse line of wheels. Comparison of Bakht's (1988) and AASHTO's equations produce significantly different results, with the latter suggesting much wider effective strip widths. The AASHTO equations are based in part on work by Sexsmith *et al.* (1979), wherein it was determined that the slight flexural softening of wood members stressed beyond the proportional limit results in load sharing between adjacent members. The work by Sexsmith *et al.* forms part of the basis for the load-sharing factor that is used when determining the flexural and shear resistances of wood members in both the CHBDC and CSA O86. The AASHTO LFRD Bridge Design Specifications do not utilize a load-sharing factor when determining member resistance. Accordingly, it would seem potentially unconservative to use the AASHTO effective strip width equations and the CHBDC load-sharing factor for amplifying member resistances, as this might result in relying on the work of Sexsmith *et al.* for both the demand and capacity. Research is currently underway to investigate this possibility. As a result, this design example relies on the work of Bakht (1988) when determining the effective strip with of the deck.

The effective strip width of a deck panel, b_{eff}, is calculated using the equation from clause 5.7.3.2 of the CHBDC for decks without edge stiffening. This equation is a numerical representation of the design curve presented by Bakht for decks without edge stiffening. It is a function of the girder spacing, S. The edge stiffening refers to stiffening of the free edges that run parallel to the span of the deck; that is, the edges oriented in the transverse direction of the bridge. These edges are not stiffened because it is impractical to do so for the deck panel and girder configuration described in this design example. This edge stiffening is not to be confused with the role of the longitudinal stiffener beams, the latter of which serves to share moments and shears acting in the longitudinal direction of the bridge amongst adjacent deck panels.

The effective strip width of a deck panel, b_{eff}, is calculated as

$$b_{eff} = 0.30 + 0.14S = 0.30 + 0.14 \times 1.150 \, m = 0.461 \, m$$

Accordingly, a single strip of deck panel of 461 mm width is modelled as a continuous beam spanning across the girders. The girders are considered to be rigid vertical supports. The bending moments due to self-weight, wearing surface, barriers, wind, and live load are determined from the analysis, as is the live load deflection. Shears are ignored because the shear design of vertically laminated decks is not required, per CHDBC clause 9.7.5.

The deck has a depth of 215 mm and a unit weight of 6 kN/m³, resulting in an unfactored linear weight of

$$\omega_{deck} = 0.215 \, m \times 0.461 \, m \times 6\frac{kN}{m^3} = 0.59 \, kN/m$$

The minimum and maximum ULS load factors for the deck are 0.90 and 1.20, respectively.

The asphalt wearing surface has a unit weight of 23.5 kN/m. The minimum and maximum asphalt thicknesses are 50 mm and 185 mm, respectively, resulting in an unfactored trapezoidal distributed load that varies linearly from $\omega_{ws,min}$ to $\omega_{ws,max}$, where

$$\omega_{ws,min} = 0.050\ m\ \times 0.461\ m \times 23.5\frac{kN}{m^3} = 0.55\frac{kN}{m}$$

$$\omega_{ws,min} = 0.185\ m\ \times 0.461\ m \times 23.5\frac{kN}{m^3} = 2.00\ kN/m$$

This trapezoidal distributed load is applied to the continuous deck design strip within the limits of the wearing surface. The minimum and maximum ULS load factors for the wearing surface are 0.65 and 1.50, respectively.

The weight of the timber railings is applied as a concentrated load at the free end of each deck cantilever. Each railing results in an unfactored point load of

$$P_{railing} = 1.30\frac{kN}{m} \times 0.461\ m = 0.60\ kN$$

The minimum and maximum ULS load factors for the timber railings are 0.90 and 1.20, respectively.

The vertical wind pressure is 930 Pa. The resulting unfactored uniformly distributed loads acting along the length of the deck design strip is

$$\omega_{wind,uniform} = 930\ Pa \times \frac{kPa}{1000\ Pa} \times 0.461\ m = 0.43\frac{kN}{m}$$

A previously stated, eccentric wind load must be considered, with the resultant of the wind load acting along the windward quarter-point. By geometry, the magnitude of the wind pressures at the windward and leeward sides of the deck for downward acting vertical wind, are 2.5 x F_v and −0.5 x F_v, respectively. For downward acting vertical wind, those magnitudes become −2.5 x F_v and 0.5 x F_v, respectively, at the windward and leeward sides of the deck. The pressures result in a trapezoidal distributed load acting along the length of the deck design strip. Those corresponding pressures are

$$\pm2.5 \times F_v = \pm2.5 \times 930\ Pa \times \frac{kPa}{1000\ Pa} \times 0.461\ m = \pm1.08\frac{kN}{m}$$

$$\pm0.5 \times F_v = \pm0.5 \times 930\ Pa \times \frac{kPa}{1000\ Pa} \times 0.461\ m = \pm0.22\frac{kN}{m}$$

The vertical wind load is considered to act upward and downward. It is only applicable for the ULS 3 and ULS 4 load combinations. The ULS load factors for those combinations are 0.45 and 1.40, respectively.

Live load is to be applied to the deck design strip. The live load shall be the heaviest transverse line of wheels. For the CL-625-ONT loading, axle 4 is the heaviest transverse line of wheels. These wheel loads are positioned transversely within the design lanes to create the worst load effect. This positioning must respect the lane widths determined by clause 3.8.2 of the CHBDC. It must also respect the truck clearance envelope specified in clauses 3.8.4.1(d)(ii) and 3.8.4.3(d) of the CHBDC. Both two and three lanes are considered for this design example. For the two lane configuration, each of a single loaded lane and both loaded lanes is considered. For the three lane configuration, each of one, two, or three loaded lanes is considered for all possible combinations. Multi-lane reduction factors of 1.0, 0.90, and 0.80 are applied for one, two, and three loaded lanes, respectively. This analysis is made easy by use of computer structural analysis. The analysis is made less conservative by idealizing the wheels as discrete uniformly distributed loads instead of concentrated loads.

The wheel loads are amplified by the dynamic load allowance of 0.40 for a single truck axle, per clause 3.8.4.5.3 of the CHBDC. This value is reduced by 30% to account for the dynamic qualities of wood bridges, per CHBDC clause 3.8.4.5.4. The resulting dynamic load allowance is calculated to be 1 + 0.40 x 0.70 = 1.28.

All of the previously described load effects are applied to a continuous beam of 461 mm width that spans the full 14110 mm deck width. The girders are assumed to act as rigid vertical supports.

The total factored bending moments experienced by the deck design strip are summarized in the following table.

TOTAL FACTORED BENDING MOMENTS

Span	SLS 1	ULS 1	ULS 2	ULS 3	ULS 4	ULS 4 (Uplift)	
interior	18	34	32	28	1	1	kNm
cantilever	-16	-30	-28	-25	-1	-1	kNm

The factored SLS live load deflections experienced by the deck design strip are summarized in the following table. They have not been amplified by the dynamic load allowance.

LIVE LOAD DEFLECTIONS

Span	SLS 1	
interior	1.1	mm
cantilever	1.3	mm

3.2.4.2. GLUED-LAMINATED TIMBER GIRDERS

The girders can be analyzed using traditional simplified methods or by computer structural analysis. This design example examines both approaches.

3.2.4.2.1. SIMPLIFIED METHOD

The traditional simplified approach is based on clause 5.6 of the CHBDC. The permanent loads acting within the tributary width of a girder are modelled along the length of that single girder. Live load distribution factors are calculated to determine the percentage weight of the vertical live load that is carried by a single girder. That weight is then applied, in addition to the permanent loads, and the load effects are determined. Both and interior and exterior girder are analyzed.

The interior and exterior girders have a depth of 1634 mm, a width of 215 mm, and a unit weight of 6 kN/m³, resulting in unfactored linear weights of

$$\omega_{girder,int} = \omega_{girder,ext} = 1.634 \; m \; \times 0.215 \; m \times 6\frac{kN}{m^3} = 2.11\frac{kN}{m}$$

The minimum and maximum ULS load factors for the girders are 0.90 and 1.20, respectively.

The asphalt wearing surface has a unit weight of 23.5 kN/m. An interior girder near the bridge centreline has an average asphalt thickness of 174 mm and a tributary asphalt width of 1150 mm. An exterior girder has an average asphalt thickness of 46 mm and a tributary asphalt width of 1000 mm. This geometry results in unfactored linear weights of

$$\omega_{ws,int} = 0.174 \; m \; \times 1.150 \; m \times 23.5\frac{kN}{m^3} = 4.70 \; kN/m$$

$$\omega_{ws,ext} = 0.046 \; m \; \times 1.000 \; m \times 23.5\frac{kN}{m^3} = 1.08 \; kN/m$$

The minimum and maximum ULS load factors for the asphalt wearing surface are 0.65 and 1.50, respectively.

The weight of the timber railings is assumed to be carried exclusively by the exterior girders. Each railing has an unfactored linear weight of

$$\omega_{railing} = 1.40 \frac{kN}{m}$$

The minimum and maximum ULS load factors for the timber railings are 0.90 and 1.20, respectively.

The vertical wind pressure is 930 Pa. The resulting unfactored uniformly distributed loads acting on the interior and exterior girders, respectively, are

$$\omega_{wind,int} = 930 \; Pa \times \frac{kPa}{1000 \; Pa} \times 1.150 \; m = 1.07 \frac{kN}{m}$$

$$\omega_{wind,ext} = 930 \; Pa \times \frac{kPa}{1000 \; Pa} \times 1.305 \; m = 1.21 \frac{kN}{m}$$

The vertical wind load is considered to act upward and downward. It is only applicable for the ULS 3 and ULS 4 load combinations. The ULS load factors for those combinations are 0.45 and 1.40, respectively.

The distribution of discrete live loads acting across the width of the bridge must be considered. The simplified method of analysis for longitudinal load effects, per CHBDC clause 5.6, is an appropriate means of determining the percentage of live load carried by a single girder. Clause 5.6.7, which specifically addresses transverse live load distribution in sawn wood stringer bridges with transverse laminated wood decks, is used for this purpose in this design example. The glued-laminated timber girders are considered analogous to sawn wood stringers, given that they both have similar flexural and torsional stiffnesses. The resulting live load effects are added to the effects caused by self-weight, wearing surface, barriers, and wind load to produce the total load effects.

The use of the CHBDC simplified method of analysis relies upon satisfying the following criteria from clause 5.6.2.

Clause	Criteria	Criteria Satisfied?
(a)	the width of the bridge is constant	Yes
(b)	the deck is continuous along the entire bridge width	Yes
(c)	The span between centreline of supports or bearing units is constant throughout the width of the bridge	Yes
(d)	the support conditions are closely equivalent to line support in all cases	Yes
(f)	diaphragms and bracing systems comply with the applicable requirements of Sections 8 to 10 and 17	Yes

The bridge width is constant, satisfying clause (a). The deck is continuous along the entire bridge width, satisfying clause (b). The span is constant length and the girders are spaced closely enough to approximate a line support, satisfying clauses (c) and (d). Finally, there are diaphragms at each support, per clause 9.20.2, satisfying clause (f). Note that the diaphragms that are offset 600 mm from the centreline of abutment bearings may be considered as abutment diaphragms in assessing whether there are diaphragms at the supports. Previous designs have had success with the diaphragms offset up to 900 mm from the centreline of abutment bearings (Wacker, J.P. & Smith, M.S. 2001). The criteria from clauses 5.6.2 (e), (g), (h), (i), (j), and (k) are not applicable to wood bridges, and have thus been excluded from the above table. Therefore, it is acceptable to use the simplified method for analysis of the girders.

The deck has a travelled width of $w_c = 13.500\ m$. From Table 3.5 of the CHBDC, both two and three design lanes are to be considered for this travelled width. For two design lanes, the average lane width, w_e, is

$$w_e = \frac{w_c}{n} = \frac{13.500\ m}{2\ lanes} = 6.75\ m\ per\ design\ lane$$

Similarly, the average lane width for three design lanes is 4.500 m.

The longitudinal bending moment per girder due to CL-625-ONT live loading, M_L, is calculated as

$$M_L = F_T F_S M_T$$

where F_T is the truck fraction carried per girder, F_S is the skew factor, and M_T is the longitudinal bending moment generated by the passage of the CL-625-ONT live loading along a single design lane.

Similarly, the longitudinal shear force per girder due to CL-625-ONT live loading, V_L, is calculated as

$$V_L = F_T F_S V_T$$

where V_T is the longitudinal shear force generated by the passage of the CL-625-ONT live loading along a single design lane.

The skew factor, F_S, is taken as 1.0, per CHBDC clause 5.6.4.5 because the bridge is not skewed.

The truck fraction, F_T, is calculated as

$$F_T = \frac{S}{D_T \gamma_c (1 + \mu\lambda)} \geq 1.05 \frac{nR_L}{N} \ for \ ULS \ \& \ SLS$$

$$F_T = \frac{S}{D_T \gamma_c (1 + \mu\lambda + \gamma_e)} \geq 1.05 \frac{1}{N} \ for \ FLS$$

The girder spacing, S, is equal to 1.150 m. The number of girders, N, is equal 12.

The truck load distribution width, D_T, is taken from Table 5.11 of the CHBDC for sawn wood stringer bridges with transverse laminated wood decks. The value is a function of the number of design lanes, n, and the span length for the equivalent beam method, L_e. The latter value is equal to the span length between inflection points under the governing load case. Given that the bridge is simply-supported, value of L_e is merely equal to the span length of 18 m. CHBDC Figure 5.1.(a) may be used to determine the value of L_e for multi-span bridges.

For two design lanes or more, the truck load distribution width for bending moments at ULS, $D_{T,m,ULS}$, is calculated to be

$$D_{T,m,ULS} = 2.60 + 0.08 L_e \leq 3.4$$

$$D_{T,m,ULS} = 2.60 + 0.08 \times 18 \ m = 4.04 \ m > 3.4 \ m$$

$$\therefore D_{T,m,ULS} = 3.4 \ m$$

Similarly, the truck load distribution width for bending moment at FLS, $D_{T,m,FLS}$, is calculated to be

$$D_{T,m,FLS} = 3.6 \ m$$

Analysis at FLS is normally not required for the design of wood components, but the truck fraction for bending moment at FLS may be used as an approximate means for determining the live load deflection of a component at SLS using the simplified method, per CHBDC clause 5.6.4.7.

From CHBDC Table 5.11, the truck load distribution width for interior girder shear at ULS is equal to $D_{T,v,ULS} = 3.10 \ m.$.

The truck modification factor, γ_c, is taken from CHBDC Table 5.3 and varies for interior and exterior girders as a function of the load effect considered, number of design lanes, and girder arrangement. For consideration of bending moments in interior girders at ULS, $\gamma_{c,int,m,ULS} = 1.0$ for any number of design lanes. For consideration of bending moments in exterior girders at ULS, the truck modification factor is taken from Table 5.5. For this design example, the distance from the free end of the deck cantilever to the centreline of the exterior girder web, S_c, is equal to 730 mm, which is more than half the girder spacing. Therefore, the truck modification factor is taken as

$$\gamma_{c,ext,m,ULS} = 1.25 - \frac{0.50 S_c}{S} \leq 1.0$$

$$\gamma_{c,ext,m,ULS} = 1.25 - \frac{0.50 \times 0.730\ m}{1.150\ m} = 0.932 \leq 1.0$$

The use of the previous equation is predicated on the assumption that $S_c = 0.730\ m \leq 0.6S = 0.690\ m$. This assumption is violated, but only slightly, so the violation is ignored. If the ratio of S_c to S was equal to the limiting 0.60 ratio, then the truck modification factor would be equal to 0.950, which is insignificantly greater than 0.932. Furthermore, a lesser value of γ_c results in greater live load effect experienced by a girder, so the value of 0.932 is conservative for design.

The truck modification factor for consideration of shear at ULS in interior and exterior girders, $\gamma_{c,v,ULS}$ is taken from Table 5.6. Since the girder spacing is less than 2.0 m, the value of $\gamma_{c,v,ULS}$ is

$$\gamma_{c,v,ULS} = \left(\frac{S}{2}\right)^{0.25} = \left(\frac{1.150\ m}{2}\right)^{0.25} = 0.871$$

The truck modification factor for consideration of bending moments at FLS in interior girders, $\gamma_{c,int,m,FLS}$, is found from Table 5.4 to be equal to 1.0. The corresponding value for exterior girders, $\gamma_{c,ext,m,FLS}$, is found from Table 5.5 to be 0.932.

The lane width modification factor, μ, is equal to $\mu = \frac{w_e - 3.3}{0.6} \leq 1.0$

The lane width modification factor is calculated to be 5.75 and 2 for two and three design lanes, respectively. Since both those values exceed 1.0, the lane width modification factor is taken as equal to 1.0.

The width correction factor, λ, is taken as zero, per CHBDC clause 5.6.7.5.

The truck fraction for bending moments at FLS also relies on the truck modification factor for exterior girders, γ_e. This value accounts for the influence of vehicle edge distance (CSA 2014b). It is equal to zero for interior girders. It is calculated using the equations in CHBDC Table 5.7 for exterior girders. For the bridge in this design example, the value of γ_e for exterior girders is calculated as

$$\gamma_e = 0.28(D_{VE} - 1.0)[1 + 0.40(D_{VE} - 1.0)^2]$$

The vehicle edge distance, D_{VE}, is defined in CHBDC Figure 5.2. to be the distance from the longitudinal free edge of the deck to the centreline of exterior line of wheels for a truck in an exterior design lane. In calculating, D_{VE}, the truck is to be placed in the centre of an exterior design lane. For this design example, the magnitude of D_{VE} is calculated for two and three lane configurations as

$$D_{VE,2\ lanes} = 0.305\ m + \frac{6.750\ m}{2} - \frac{1.8\ m}{2} = 2.780\ m$$

$$D_{VE,3\ lanes} = 0.305\ m + \frac{4.500\ m}{2} - \frac{1.8\ m}{2} = 1.655\ m$$

Accordingly, the magnitude of γ_e for two and three lane configurations, respectively, is

$$\gamma_{e,2\ lanes} = 0.28(2.780 - 1.0)[1 + 0.40(2.780 - 1.0)^2] = 1.130$$

$$\gamma_{e,3\ lanes} = 0.28(1.655 - 1.0)[1 + 0.40(1.655 - 1.0)^2] = 0.215$$

Based on the preceding values, the truck fractions for bending moments at ULS in the interior and exterior girders $F_{T,v,int}$, and $F_{T,m,ext}$, respectively, are calculated to be

$$F_{T,m,int} = \frac{S}{D_{T,m,ULS}\gamma_{c,int,m,ULS}(1 + \mu\lambda)} = \frac{1.150\ m}{3.4\ m \times 1.000 \times (1 + 1.0 \times 0)} = 0.340$$

$$F_{T,m,int} = 0.340 > 1.05\frac{nR_L}{N} = 1.05\frac{3 \times 0.90}{12} = 0.236$$

$$\therefore F_{T,m,int} = 0.340$$

$$F_{T,m,ext} = \frac{S}{D_{T,m,ULS}\gamma_{c,ext,m,ULS}(1 + \mu\lambda)} = \frac{1.150\ m}{3.4\ m \times 0.932 \times (1 + 1.0 \times 0)} = 0.363$$

$$F_{T,m,ext} = 0.363 > 1.05\frac{nR_L}{N} = 1.05\frac{3 \times 0.90}{12} = 0.236$$

$$\therefore F_{T,m,ext} = 0.363$$

The truck fraction for shear at ULS in the interior girder, $F_{T,v,int}$ is calculated to be

$$F_{T,v,int} = \frac{S}{D_{T,v,ULS}\gamma_{c,v,ULS}(1 + \mu\lambda)} = \frac{1.150\ m}{3.10\ m \times 0.871 \times (1 + 1.0 \times 0)} = 0.426$$

$$F_{T,v,int} = 0.426 > 1.05\frac{nR_L}{N} = 1.05\frac{3 \times 0.90}{12} = 0.236$$

$$\therefore F_{T,v,int} = 0.426$$

Clause 5.6.7.1 of the CHBDC specifies that the shear in an exterior girder is to be calculated as the reaction due to the wheel loads acting on the exterior girder, if the deck is simply-supported between girders (i.e. the lever rule), but that the calculated shear shall not be less than that determined using the truck fraction for shear in an interior girder. The centreline of the exterior wheels of a design truck cannot come closer than 600 mm to the inside face of the curb for global analysis, per CHBDC clause 3.8.4.1(d)(ii). The distance from the first interior girder to the centreline of the exterior line of wheels, x_{ext}, is thus equal to

$$x_{ext} = 1.150 \, m + 0.730 \, m - 0.305 \, m - 0.600 \, m = 0.975 \, m$$

The truck wheels are spaced 1.800 m apart, resulting in the distance from the first interior girder to the centreline of the interior line of wheels, x_{int}, being equal to

$$x_{int} = x_{ext} - 1.800 \, m = 0.975 \, m - 1.800 \, m = -0.825 \, m$$

The negative value of x_{int} means that the interior line of wheels will not contribute to the shear force in the exterior girder, as that wheel line is positioned between the first and second interior girder and is thus not within the tributary width of the exterior girder. The truck fraction for shear in the exterior girder at ULS, $F_{T,v,ext}$, is therefore equal to

$$F_{T,v,ext} = \frac{x_{ext}}{2S} = \frac{0.975 \, m}{2 \times 1.150 \, m} = 0.424$$

$$F_{T,v,ext} = 0.424 < F_{T,v,int} = 0.426$$

$$\therefore F_{T,v,ext} = F_{T,v,int} = 0.426$$

The interior line of wheels may not always be positioned outside the tributary width of the exterior girder for the ULS shear truck fraction calculation. Refer to Timber Construction Manual by the American Institute for Timber Construction (2012) for an examination of the effect of alternate truck positioning on the calculation of the truck fraction for shear in exterior girders.

Recall that the truck fraction for bending moment at FLS can be used to approximate the SLS live load deflection in a girder. Based on the previously calculated values, the truck fractions for the interior girder, $F_{T,m,int,FLS}$, and the exterior girder, $F_{T,m,ext,FLS}$, are equal to

$$F_{T,m,int,FLS} = \frac{S}{D_{T,m,FLS}\gamma_{c,m,FLS}(1 + \mu\lambda + \gamma_e)} = \frac{1.150\ m}{3.6 \times 1.0 \times (1 + 1.0 \times 0 + 0)} = 0.320$$

$$F_{T,m,int,FLS} = 0.320 > 1.05\frac{1}{N} = \frac{1.05}{12} = 0.088$$

$$\therefore F_{T,m,int,FLS} = 0.320$$

$$F_{T,m,ext,FLS} = \frac{S}{D_{T,m,FLS}\gamma_{c,m,FLS}(1 + \mu\lambda + \gamma_e)} = \frac{1.150\ m}{3.6 \times 0.932 \times (1 + 1.0 \times 0 + 0.215)} = 0.282$$

$$F_{T,m,ext,FLS} = 0.282 > 1.05\frac{1}{N} = \frac{1.05}{12} = 0.088$$

$$\therefore F_{T,m,ext,FLS} = 0.282$$

The longitudinal bending moment, shear force, and deflection, generated by the passage of the CL-625-ONT live loading along a single design lane is most easily determined using the moving load analysis function of a structural analysis program. The maximum bending moment, shear, and deflections are found to be as follows:

Live Load Effect	Truck Load	Lane Load	Truck Axles on Bridge
bending moment	1505 kNm	1568 kNm	1 to 4
shear force	394 kN	397 kN	2 to 5
deflection	72 mm	76 mm	1 to 4

The load effects due to the lane load are greater than those due to the truck load; however, it is ultimately the truck load that will govern in this design example because the truck load is subject to amplification by the dynamic load allowance, while the lane load is not subject to this amplification. The dynamic load allowance taken as 0.25 because four axles cause the critical load effects. The dynamic load allowance is reduced by 30% to account for the dynamic qualities of wood bridges, per CHBDC clause 3.8.4.5.4. The resulting M_T, V_T, and Δ_T are calculated to be

$$M_T = 1505\ kNm \times (1 + 0.25 \times 0.70) = 1505\ kNm \times 1.175 = 1769\ kNm$$

$$V_T = 394\ kN \times (1 + 0.25 \times 0.70) = 394\ kN \times 1.175 = 463\ kN$$

$$\Delta_T = 72\ mm \times (1 + 0.25 \times 0.70) = 72\ mm \times 1.175 = 85\ mm$$

The resulting maximum bending moment experienced by an interior girder and an exterior girder due to CL-625-ONT live loading, $M_{L,int}$ and $M_{L,int}$, respectively, are calculated to be

$$M_{L,int} = 0.340 \times 1.0 \times 1769\ kNm = 602\ kNm$$

$$M_{L,ext} = 0.363 \times 1.0 \times 1769\ kNm = 642\ kNm$$

The resulting maximum shear force experienced by an interior girder and an exterior girder due to CL-625-ONT live loading, $V_{L,int}$ and $V_{L,ext}$, respectively, are calculated to be

$$V_{L,int} = 0.426 \times 1.0 \times 463 \; kN = 198 \; kN$$

$$V_{L,ext} = 0.426 \times 1.0 \times 463 \; kN = 198 \; kN$$

The resulting maximum live load deflection experienced by an interior girder and an exterior girder due to CL-625-ONT live loading, $\Delta_{L,int}$ and $\Delta_{L,ext}$, respectively, are calculated to be

$$\Delta_{L,int} = 0.320 \times 1.0 \times 72 \; mm = 23 \; mm$$

$$\Delta_{L,ext} = 0.282 \times 1.0 \times 72 \; mm = 21 \; mm$$

The calculated live load effects are combined with the permanent load effects using the appropriate CHBDC load combinations. The resulting factored bending moments, factored shear forces, SLS live load deflections, and unfactored permanent load deflections are shown below for the interior and exterior girders. The SLS live load deflections exclude the dynamic load allowance.

Load Effect	Exterior Girder	Interior Girder
Mf	1453 kNm	1489 kNm
Vf	445 kN	440 kN
Δ_{LL}	16 mm	18 mm
$\Delta_{PERMANENT}$	11 mm	17 mm

3.2.4.2.2. COMPUTER STRUCTURAL ANALYSIS

Computer structural analysis is in the form of a three-dimensional grillage model. Frame elements are used to represent the girders, stiffener beams, deck panels, with one line of frame elements used for each deck panel, stiffener beam, each girder. Link elements are used to model the connections and bearings. Shell elements are used to model the girder diaphragms.

The factored bending moments, factored shear forces, SLS live load deflections, and unfactored permanent load deflections are determined by computer structural analysis are shown below for the interior and exterior girders. The SLS live load deflections exclude the dynamic load allowance

Load Effect	Exterior Girder	Interior Girder
M_f	1474 kNm	1308 kNm
V_f	374 kN	363 kN
Δ_{LL}	13 mm	14 mm
$\Delta_{PERMANENT}$	13 mm	17 mm

3.2.4.2.3. STRUCTURAL ANALYSIS RESULTS FOR DESIGN

The girders were analyzed using the CHBDC simplified method and by computer structural analysis. This following table compares the results.

Load Effect	Exterior Girder		Interior Girder	
	Simplified Analysis	Computer Analysis	Simplified Analysis	Computer Analysis
M_f	1453 kNm	1474 kNm	1489 kNm	1308 kNm
V_f	445 kN	374 kN	440 kN	363 kN
Δ_{LL}	16 mm	13 mm	18 mm	14 mm
$\Delta_{PERMANENT}$	11 mm	13 mm	17 mm	17 mm

Comparison of the structural analysis results shows that the simplified method overestimates the bending moments, shears, and live load deflections for both interior and exterior girders, relative to the use of computer structural analysis. This observation follows the general trend in bridge engineering, wherein a refined analysis using a computer yields lesser magnitude load effects than a simplified analysis. With that said, the simplified method of analysis is sometimes more preferential, particularly for preliminary design and/or situations where structural analysis software is unavailable or basic in capability. The design of the girders is based on the results of the computer structural analysis for this design example, as those results lead to a more economical design.

As noted by clause 9.7.3 of the CHBDC, the factored shear resistance is to exceed the factored shear load. The factored shear load represents the maximum horizontal shear force experienced by a glued-laminated member. It is not the same as the factored shear force experienced at a given cross-section. Glued-laminated timber is known to be weaker in horizontal shear than transverse vertical shear, so the shear design of glued-laminated timber members is predicated on designing for horizontal shear. Clause 9.7.3 presents an equation to determine the factored shear load. It determines the maximum horizontal shear force experienced by the member as a function of the member volume and the vertical shear force distribution along the member. The equation is cumbersome to use in the presence of live load, as the critical live load position for maximum horizontal shear is generally not the same as the critical live load position for maximum vertical shear. Consequently, a trial-and-error approach is often taken, wherein a designer moves the live load along the bridge in discrete increments and calculates the shear load for each increment. Furthermore, the calculation of shear load does not allow for superposition of load cases. Therefore, the designer must calculate the shear load with all applicable loads applied simultaneously. The ULS 1 load combination governs in this design example, and will likely always govern for wood bridge shear design.

The calculation of the shear load is best performed on an isolated girder. It is often beneficial to apply permanent loads at discrete locations, say the span tenth-points, as the shear load integral reduces to a simple summation for step-wise vertical shear force diagrams. A fraction of the weight of the CL-625-ONT loading is applied to the girder. That fraction is either the truck fraction determined by use of the CHBDC simplified method of live load analysis or the truck fraction implied from the global computer structural analysis model. This latter term is calculated as the ratio of the maximum shear force experienced by a girder in the computer structural analysis model to the maximum shear force experienced by single isolated girder under the passage of one lane of CL-625-ONT loading. The shear loads in this design example are based on the implied truck fraction determined from the computer structural analysis model. The resulting live load is factored and moved along the girder until the factored shear load is maximized.

There is no intuitive positioning of the live load that maximizes the shear load. The live load was moved along the girder at one metre intervals to determine the shear load for this design example, resulting in a total of 36 distinct shear load calculations. Multi-step live load analysis and filtered spreadsheet results are useful tools for this procedure. The shear load calculations are not shown in this design example due to their size. Refer to the CHBDC commentary clause C9.7 for an example calculation of the shear load.

The calculated shear load for the exterior and interior girders are summarized in the following table.

Load Effect	Exterior Girder	Interior Girder
factored vertical shear	374 kN	363 kN
factored shear load	168 kN	168 kN
ratio	0.45	0.47

The use of the factored shear load for shear design is obviously quite advantageous relative to the use of the factored vertical shear. The factored demand is reduced by over 50% in this case. Despite its cumbersome calculation process, there is significant benefit to be gained by calculating the factored shear load versus using the much easier to obtain factored vertical shear.

The factored shear load is a design concept distinct to the CHBDC since the 2000 edition. Previous wood design practice was to design for the maximum vertical shear force at the lesser of the span quarter-point and three beam depths from the support. Since many wood bridge girders have a span-to-depth ratio of approximately 10 to 12, these two locations often roughly coincide. The AASHTO LFRD Bridge Design Specifications still make use of this design approach. A comparison of this method versus the shear load is made in the following table for the interior and exterior girders.

Location	Exterior Girder	Interior Girder
at support	374 kN	363 kN
span quarterpoint	214 kN	200 kN
three beam depths from support	213 kN	198 kN
shear load	168 kN	168 kN

The shear load is less than the maximum shear force at the span quarter-point and at three beam depths from the support. Knowing this, a designer may wish to use the shear force at span quarter-point for preliminary design, and then perform the rigorous shear load calculation for detailed design.

3.2.4.3. GLUED-LAMINATED STIFFENER BEAMS

The longitudinal stiffener beams are specified as continuous beams spanning the entire length of the bridge in order to ease constructability. They can be idealized as discrete length beams with two fasteners per deck panel for the purposes of analysis and design. The longitudinal bending moment and shear that must be transferred by a stiffener beam between adjacent deck panels is found using the analysis equations presented by McCutcheon and Tuomi (1973, 1974). The factored moment, $M_{f,stiffener}$, and the factored shear, $V_{f,stiffener}$, that are to be transmitted by the stiffener beam are found to be

$$M_{f,stiffener} = 4.8 \; kNm$$

$$V_{f,stiffener} = 51.7 \; kN$$

This factored moment and factored shear are used to determine the forces in the stiffener beams fasteners using the work of Witmer et al. (2002). The maximum fastener axial force, T_f, is found to be

$$T_f = 55 \; kN$$

The shear force diagram of a stiffener beam can be determined using the calculated fastener forces. In doing so, the shear load in the stiffener beam is calculated as

$$V_f = 33 \; kN$$

3.2.5. MEMBER DESIGN

3.2.5.1. GLUED-LAMINATED TIMBER DECK

The deck panels are designed for flexure in accordance with clause 9.6 of the CHBDC. The panels are considered to be analogous to transverse nail-laminated decks of No. 2 grade for material properties, size effect factor, and load-sharing factor. Shear is not considered in the design of the deck panels in accordance with clause 9.7.5 of the CHBDC.

Differential live load deflection of the deck panels is considered in accordance with clause 9.4.2 of the CHBDC. An additional maximum deflection criterion of 0.05 in (1.27 mm) is imposed based on the findings of limiting the potential for asphalt pavement cracking (Eriksson et al. 2003).

3.2.5.1.1. FLEXURAL DESIGN

The flexural resistance of the deck, $M_{r,deck}$, is calculated as for a sawn wood member, per CHBDC clause 9.6.1. The resistance is equal to

$$M_{r,deck} = \phi k_d k_{ls} k_m k_{sb} f_{bu} S$$

The material resistance factor, $\emptyset$, is determined from CHBDC Table 9.1 to be equal to 0.9 for sawn wood in flexure.

The load-duration factor, k_d, is determined using CHBDC clause 9.5.3. This factor considers the influence of cumulative load towards the creep rupture of a member (FPL 2010). The critical design bending moments and shears are the result of the ULS 1 load combination. Accordingly, the load-duration factor is equal to 1.0. The load-duration factor is equal to 1.15 when calculating the resistance against loads resulting from the ULS 3 and ULS 4 load combinations, as these combinations include wind load.

The lateral stability factor, k_{ls}, is determined using CHBDC clause 9.6.3. This factor accounts for the possibility of lateral-torsional buckling instability occurring before cross-sectional rupture occurs. It is equal to 1.0 for laminated wood decks.

The load-sharing factor, k_m, is determined using CHBDC clause 9.5.6. This factor accounts for the reduced probability of material defects within adjacent members of given cross-section. It also accounts for the statical indeterminacy afforded by neighbouring members (CSA 2014b). Wood is traditionally designed using the assumption of linear-elastic material behaviour; however, Sexsmith *et al*. (1979) demonstrated that the slight flexural softening experienced by wood members before failure is sufficient to shed load to adjacent stiffer members in systems of closely spaced members. The effective width over which this phenomenon occurs D_e, is equal to 0.400 m for a transverse nail-laminated deck, per CHBDC Table 9.3. The deck laminations have a width of 0.038 m, resulting in $n = 0.400\ m \div 0.038\ m = 10.5$ laminations being effective in sharing load. Using the value of $n = 10.5$ laminations, the load-sharing factor is linearly interpolated from CHBDC Table 9.2 to be equal to 1.36. Refer to Bakht and Jaeger (1991) for further information concerning the load-sharing in timber design.

The size-effect factor, k_{sb}, is linearly interpolated from CHBDC Table 9.4 to be equal to 1.14 for 38 mm x 215 mm deck laminations.

The loss of stiffness due to the presence of butt joints is considered in the design of design of nail-laminated decks, per CHBD clause 9.5.8. This consideration is unnecessary for the design of glued-laminated timber deck panels because the individual laminations in a glued-laminated timber member are finger-jointed and glued together to form a continuous lamination. These joints undergo strength testing to ensure their performance as continuous joints, per CSA O122, during the manufacture of glued-laminated timber. The section modulus is therefore equal to

$$S = \frac{bd^2}{6} = \frac{461\ mm \times (215\ mm)^2}{6} = 3552 \times 10^3\ mm^3$$

The resulting factored flexural resistance is equal to

$$M_r = \phi k_d k_{ls} k_m k_{sb} f_{bu} S$$

$$M_{r,deck} = 0.9 \times 1.0 \times 1.0 \times 1.36 \times 1.14 \times 8.4\ MPa \times 3552 \times 10^3\ mm^3 = 41\frac{kNm}{m}$$

The factored flexural demand was calculated to be equal to

$$M_{f,deck} = 34\frac{kNm}{m} < M_{r,deck} = 41\frac{kNm}{m}$$

Therefore, the deck has sufficient flexural capacity.

3.2.5.1.2. SLS DESIGN

The SLS deflection due to live load under the SLS 1 load combination is limited to a maximum of 1/400th the span, per clause 9.4.2 of the CHBDC. These limiting values for the exterior and interior spans are

$$\Delta_{max,ext} = \frac{L_{ext}}{400} = \frac{730}{400} = 1.8\ mm$$

$$\Delta_{max,int} = \frac{L_{int}}{400} = \frac{1150}{400} = 2.9\ mm$$

The SLS 1 live load deflections were determined to be

$$\Delta_{SLS,ext} = 1.3\ mm < \Delta_{max,ext} = 1.8\ mm$$

$$\Delta_{SLS,int} = 1.1\ mm < \Delta_{max,int} = 2.9\ mm$$

Therefore, SLS deflections are within the permissible limit prescribed the CHBDC.

To minimize the potential for cracking of the asphalt wearing surface, Eriksson et al. (2003) suggested limiting the differential deflection of deck panels to 0.05 inches (1.3 mm). The differential deck deflections do not exceed this limit, so the potential for asphalt cracking is minimized.

3.2.5.2 GLUED-LAMINATED TIMBER GIRDERS

The bridge girders are designed for flexure and shear at the ultimate limit state, in accordance with CHBDC clauses 9.6 and 9.7, respectively. They are designed for deflection and vibrations at the serviceability limit state in accordance with clauses 9.4.2 and 3.4.4, respectively.

3.2.5.2.1. SLS DESIGN

Clause 3.4.4 of the CHDBC mandates that the deflection due to live load, including dynamic load allowance, is to be less than the deflections limits prescribed by the curves in CHBDC Figure 3.1. The deflection limits are a function of the first flexural frequency of the bridge and consider whether the bridge has sidewalks. This criterion is an implicit means of limiting accelerations for user comfort (CSA 2014b). The SLS 2 load combination is used to calculate the deflection. The live load is to be positioned as specified in clause 3.8.4.1, and the deflection is to be measured at the inside face of the bridge barriers for a bridge without sidewalks. The first flexural frequency of the bridge in this design example was determined to be 4.5 Hz using the computer structural analysis model. From CHBDC Figure 3.1., the maximum permissible deflection corresponding to this first flexural frequency for a bridge without sidewalks is 17 mm. The live load deflection at the face of the railings was determined to be 14.8 mm using the computer structural analysis model, which is less than the deflection limit. Therefore, superstructure vibrations have been addressed.

The Ontario Ministry of Transportation (MTO) Structural Manual includes guidelines for the design of bridges on low volume roads. A low volume road is defined as a road having an average annual daily traffic of 400 or less in both directions. These guidelines relax the superstructure vibration criterion defined in Clause 3.4.4 of the CHBDC to be a deflection limit of 1/360th the span under the SLS 2 load combination, irrespective of first flexural frequency. A designer may wish to make use of this relaxed criterion if superstructure vibration is governing the design of a wood bridge on a low volume road.

Clause 9.4.2 of the CHBDC requires that the deflection due to live load, excluding dynamic load allowance, be less than 1/400th the span. The SLS 1 load combination is to be used for this purpose. The live load is to be positioned as specified in clause 3.8.4.1. The 50th percentile modulus of elasticity is to be used to determine the deflection. The corresponding maximum live load deflections for the exterior and interior girders of this design example were determined to be 12.7 mm and 13.3 mm, respectively. Both of these deflections are less than the deflection limit of $18000 \ mm \div 400 \ = 45 \ mm$. Therefore, superstructure deflections under live load have been addressed.

3.2.5.2.2. FLEXURAL DESIGN

The flexural resistance of a girder, $M_{r,girder}$, is calculated as for a glued-laminated timber member, per CHBDC clause 9.6.1. The resistance is equal to the lesser of

$$M_{r,girder} = \phi k_d k_{ls} k_m f_{bu} S \text{ and } M_{r,girder} = \phi k_d k_m k_{sb} f_{bu} S$$

The variables in these two equations have the same definitions as those used to calculate the flexural resistance of the glued-laminated deck. The follow calculations demonstrate the flexural resistance of a 265 mm x 1 634 mm exterior girder.

The material resistance factor, Ø, is determined from CHBDC Table 9.1 to be equal to 0.9 for glued-laminated timber in flexure.

The load-duration factor is taken as $k_d = 1.0$.

The lateral stability factor, k_{ls}, requires explicit calculation because the depth-to-width ratio of the girders exceeds 1.0. It is calculated as a function of the slenderness factor, C_s, which is in turn a function of the laterally unsupported length, L_u. The latter is equal to the diaphragm spacing, as the diaphragms are designed to be the points of lateral support against lateral-torsional buckling for the purpose of calculating the lateral stability factor. The slenderness factor is therefore equal to

$$C_s = \sqrt{\frac{L_u d}{b^2}} = \sqrt{\frac{6000 \ mm \times 1634 \ mm}{(265 \ mm)^2}} = 11.82 > 10$$

The slenderness factor is greater than ten, so the lateral stability factor must be calculated as a function of both the slenderness ratio, C_s, and the intermediate slenderness ratio, C_k. The latter is calculated as

$$C_k = \sqrt{\frac{E_{05}}{f_{bu}}} = \sqrt{\frac{8512}{23.0}} = 19.24$$

Accordingly, the lateral stability factor is determined from CHBDC Table 9.5 to be

$$k_{ls} = 1 - 0.3 \left(\frac{C_s}{C_k}\right)^4 = 1 - 0.3 \left(\frac{11.82}{19.24}\right)^4 = 0.96$$

The load-sharing factor, k_m, is equal to 1.0, per CHBDC clause 9.5.6, because glued-laminated timber girders are not covered in CHBDC Table 9.3. Some designers have been known to treat glued-laminated timber girders as sawn wood stringers for calculating the load-sharing factor. It is possible that this approach is somewhat unconservative. Recall that the load-sharing factor accounts for the reduced probability of material defects within adjacent members of given cross-section. Since glued-laminated timber inherently contains less defects than sawn wood, it is logical that the load-sharing benefit in glued-laminated timber members is less than that in sawn wood members. Research is underway currently to investigate this possibility.

The size-effect factor for glued-laminated timber, k_{sb}, is determined from CHBDC clause 9.6.2 to be equal to

$$k_{sb} = \left(\frac{130}{b}\right)^{\frac{1}{10}} \left(\frac{610}{d}\right)^{\frac{1}{10}} \left(\frac{9100}{L}\right)^{\frac{1}{10}} \leq 1.3$$

$$k_{sb} = \left(\frac{130}{265}\right)^{\frac{1}{10}} \left(\frac{610}{1634}\right)^{\frac{1}{10}} \left(\frac{9100}{18000}\right)^{\frac{1}{10}} = 0.79 \leq 1.3$$

$$\therefore k_{sb} = 0.79$$

The section modulus of the girder is

$$S = \frac{bd^2}{6} = \frac{265\ mm \times (1634\ mm)^2}{6} = 117923 \times 10^3\ mm^3$$

The resulting factored flexural resistance of an exterior girder is equal to the lesser of

$$M_{r,girder,ext} = \phi k_a k_{ls} k_m f_{bu} S$$
$$M_{r,girder,ext} = 0.9 \times 1.0 \times 0.96 \times 1.0 \times 23.0\ MPa \times 117923 \times 10^3\ mm^3 = 2344\ kNm$$

and

$$M_{r,girder,ext} = \phi k_a k_m k_{sb} f_{bu} S$$
$$M_{r,girder,ext} = 0.9 \times 1.0 \times 1.0 \times 0.79 \times 23.0\ MPa \times 117923 \times 10^3\ mm^3 = 1928\ kNm$$

$$\therefore M_{r,girder,ext} = 1928\ kNm$$

Similar calculations for the interior girders yields $M_{r,girder,int} = 1592\ kNm$

The factored flexural demand was calculated to be equal to

$$M_{f,girder,ext} = 1474\ kNm < M_{r,girder,ext} = 1928\ kNm$$

$$M_{f,girder,int} = 1308\ kNm < M_{r,girder,int} = 1592\ kNm$$

Therefore, the girders have sufficient flexural capacity.

3.2.5.2.3. SHEAR DESIGN

The shear resistance of a girder, $V_{r,girder}$, is calculated per CHBDC clause 9.6.1 as

$$V_{r,girder} = \phi k_a k_m k_{sv} f_{vu} A / 1.5$$

The following calculations demonstrate the shear resistance of a 265 mm x 1 634 mm exterior girder.

The material resistance factor, $\emptyset$ is determined from CHBDC Table 9.1 to be equal to 0.9 for glued-laminated timber in shear.

The load-duration factor and load-sharing factor are the same as for flexure, with $k_d = 1.0$ and $k_m = 1.0$.

The size-effect factor for glued-laminated timber in shear is determined using CHBDC clause 9.7.2 to be equal to

$$k_{sv} = V^{-0.18} = (0.265\ m \times 1.634\ m \times 18.000\ m)^{-0.18} = 0.69$$

Like the section modulus, the cross-sectional area of a glued-laminated timber member does not need to be reduced in stiffness because there are no butt joints. The cross-sectional area is thus equal to

$$A = bd = 265\ mm \times 1634\ mm = 433010\ mm^2$$

The resulting factored shear resistance is equal to

$$V_{r,girder,ext} = \frac{\phi k_d k_m k_{sv} f_{vu} A}{1.5} = \frac{0.9 \times 1.0 \times 1.0 \times 0.69 \times 1.18\ MPa \times 433010\ mm^2}{1.5} = 211\ kN$$

Similar calculations for the interior girders yields $V_{r,girder,int} = 178\ kN$

The factored shear load was calculated to be equal to

$$V_{f,girder,ext} = 168\ kNm < V_{r,girder,ext} = 211\ kN$$

$$V_{f,girder,int} = 168\ kNm < V_{r,girder,int} = 178\ kN$$

Therefore, the girders have sufficient shear capacity.

3.2.5.3. GLUED-LAMINATED TIMBER DIAPHRAGMS

The glued-laminated timber diaphragms serve to brace the girders against lateral-torsional buckling, maintain the relative spacing of the girders, and transmit lateral load between girders (Ritter 1992). They are required at all supports, per clause 9.20.2 of the CHBDC. They are also required at midspan for spans less than 12 m and at the span third-points for spans of 12 m or greater. A designer may wish to add more diaphragms than required by the CHBDC to increase the lateral stability factor for girder flexure design. Ritter (1992) recommends that the diaphragm spacing not exceed 7.62 m (25 feet), regardless of the span. The diaphragms are provided at the abutments and span third-points in this design example because the bridge span exceeds 12 m. The abutment diaphragms are offset 600 mm forward from the bearings to not interfere with the bearing assemblies. CHBDC commentary clause C9.20.2 states that abutment diaphragms remain effective when offset up to one girder depth from the bearings.

The diaphragms should be as deep as possible, per CHBDC clause 9.20.2. Ritter (1992) advises leaving a gap of 50 mm to 125 mm between the top of the diaphragms and underside of the deck, to allow for air circulation and to preclude interference with deck attachment hardware. The diaphragms in this example have been detailed to provide a 25 mm gap between the diaphragms and the stiffener beams for air circulation. The resulting gap between the diaphragms and the deck is 139 mm.

Ritter (1992) also advises locating the tie rods outside of the outer tension zone of the girders, which is usually considered to be the outer ten percent of the girder depth. The tie rods in this design example are located at 222 mm from the underside of the girder, which is outside the $1634 \, mm \div 10 = 163 \, mm$ deep outer tension zone. It is sometimes convenient to position the tie rods at the interface between adjacent laminations. In doing so, the tie rod holes can be routed into the outside faces of these laminations before they are glued together. The tie rods have been positioned 133 mm from the top and bottom faces of the diaphragms for this design example, which is exactly equal to 3.5 laminations.

The primary purpose of the diaphragms is to brace the girders against lateral-torsional buckling. Analogous to steel design, 2% of the total compression force acting within the compression zone of a girder at maximum factored bending moment is taken as the brace force. The maximum factored moment in a girder is 1474 kNm. Assuming a linear-elastic stress distribution, the lever between the resultant tensile and compression forces is

$$e = 2 \times \left(\frac{1.634 \, m}{2} \times \frac{2}{3} \right) = 1.090 \, m$$

The total compression force within the compression zone of the girder is

$$C_f = \frac{M_f}{e} = \frac{1474 \ kNm}{1.090 \ m} = 1353 \ kN$$

The lateral brace force to be resisted by the diaphragm, in either tension or compression, is

$$F_{brace} = \pm 0.02 \times C_f = \pm 0.02 \times 1353 \ kN = \pm 27 \ kN$$

Brace forces acting in compression are transmitted to the diaphragms through direct bearing between the side faces of the girders and the end faces of the diaphragms. Brace forces acting in tension are transmitted to the diaphragms by the tie rods.

The diaphragms also experience forces arising from external loads acting on the bridge. The tie rods were modelled directly in the computer structural analysis model to capture these forces. The maximum factored force experienced by a tie rod due to external loads is 46 kN in tension. Thus, the maximum factored tie rod force, considering both forces arising from external loads and bracing of the girders, is equal to

$$T_f = 46 \ kN + 27 \ kN = 73 \ kN$$

It is typical practice to perform one tie rod design based on the maximum factored tie rod force. The use of different diameter tie rods can be confusing on site, leading to construction errors. This practice is not recommended. A single tie rod design is presented in this design example in section 7.3.

The diaphragms in this design example are 130 mm wide, 1406 mm deep, and 885 mm to 935 mm long, depending on which girders they are positioned between. From the computer structure analysis model, the maximum axial stress in a diaphragm is 0.83 MPa. Using the tie rod spacing of 1140 mm, the additional factored axial stress in the diaphragm due to bracing the girders at ULS is

$$\frac{27 \ kN}{130 \ mm \times 1406 \ mm} \times \frac{1000 \ N}{kN} + \frac{27 \ kN \times 1.140 \ m}{130 \times 1406^2 \div 6} \times \frac{1000 \ N}{kN} \times \frac{1000 \ mm}{m}$$
$$= 0.15 \ MPa + 0.72 \ MPa = 0.87 \ MPa$$

Therefore, the total factored axial stress in the diaphragms is

$$\sigma_{f,diaphragm} = 0.83 \ MPa + 0.87 \ MPa = 1.70 \ MPa$$

Analogous to the flexural resistance presented for the girder design, the flexural resistance of the diaphragms is taken as the lesser of

$$\sigma_{r,diaphragm} = M_{r,diaphragm} \div S = \phi k_d k_{ls} k_m f_{bu}$$

and

$$\sigma_{r,diaphragm} = M_{r,diaphragm} \div S = \phi k_d k_m k_{sb} f_{bu}$$

where

$$\emptyset = 0.9$$

$$k_d = 1.0$$

$$k_m = 1.0$$

$$\frac{d}{b} = \frac{1406\ mm}{130\ mm} = 10.82 < 10$$

$$C_s = \sqrt{\frac{L_u d}{b^2}} = \sqrt{\frac{6000\ mm \times 1634\ mm}{(265\ mm)^2}} = 11.82 > 10\ and\ < C_k$$

$$C_k = \sqrt{\frac{E_{05}}{f_{bu}}} = \sqrt{\frac{8512}{23.0}} = 19.24$$

$$\therefore k_{ls} = 1 - 0.3\left(\frac{C_s}{C_k}\right)^4 = 1 - 0.3\left(\frac{10.82}{19.24}\right)^4 = 0.97$$

$$k_{sb} = \left(\frac{130}{130}\right)^{\frac{1}{10}}\left(\frac{610}{1406}\right)^{\frac{1}{10}}\left(\frac{9100}{935}\right)^{\frac{1}{10}} = 1.15 \leq 1.3$$

$$\therefore k_{sb} = 1.15$$

$$M_{r,diaphragm} \div S = \phi k_d k_{ls} k_m f_{bu}$$
$$\sigma_{r,diaphragm} = 0.9 \times 1.0 \times 0.97 \times 1.0 \times 23.0\ MPa = 20.1\ MPa$$

and

$$M_{r,diaphragm} \div S = \phi k_d k_m k_{sb} f_{bu}$$
$$\sigma_{r,diaphragm} = 0.9 \times 1.0 \times 1.0 \times 1.15 \times 23.0\ MPa = 23.8\ MPa$$

$$\therefore \sigma_{r,diaphragm} = 20.1\ MPa > \sigma_{f,diaphragm} = 1.70\ MPa$$

Therefore, the diaphragms have sufficient capacity.

3.2.5.4. GLUED-LAMINATED TIMBER STIFFENER BEAMS

The stiffener beams are designed for the longitudinal shears and bending moments experienced by the deck panels in accordance with the work of McCutcheon and Tuomi (1973, 1974).

The longitudinal glued-laminated timber stiffener beams are designed for flexure and shear using the same procedure as for the girders. The stiffener beams may be idealized as discrete length beams to calculate the size effect factors in flexure and shear. The length may be taken as the largest distance between the fasteners required to transmit the longitudinal bending moment and shear force from one deck to another. The factored moment resistance, and factored shear resistance of a stiffener beam are calculated to be

$$M_{r,stiffener} = 9.6 \; kNm > M_{f,stiffener} = 4.8 \; kNm$$

$$V_{r,stiffener} = 51 \; kN > V_f = 33 \; kN$$

Therefore, the stiffener beams have adequate capacity.

It is imperative that the fastener holes in the stiffener beams be slotted in the longitudinal direction if a continuous stiffener beam is specified. Failure to slot these holes will lead to the development of large restraint forces due to shrinking and swelling of the decks panels perpendicular to grain. These forces could lead to failure of the stiffener beams and/or deck panels.

3.2.6. CONNECTION DESIGN

The bridge design features four major connections:

- The deck-to-girder connection;
- The deck-to-stiffener beam connection;
- The diaphragm connection; and,
- The girder bearing connection.

The design of these four major connections is explained in this section of the bridge design example.

3.2.6.1. DECK-TO-GIRDER CONNECTION

The deck-to-stiffener beam connection must resist vertical forces acting upwards and downwards, transverse forces, and longitudinal forces. The connection consists of a series of through-bolts and aluminum deck clips. Each through-bolt bears against the top of the deck and the underside of the upper leg of a Z-shaped aluminum deck clip. The lower leg of the deck clip features teeth that are seated in a shallow longitudinal slot that is routed into the side face of the girder. The deck-to-girder connection also features lag screws that are fastened through the deck panels and down into the tops of the girders.

Downward forces are resisted by direct bearing between the deck panels and tops of girders. Upward forces are resisted by tension in the through-bolts and the aluminum deck clips pulling up on the routed slots in the girders. Traverse forces are resisted by shear in the through-bolts and direct bearing of aluminum deck clips against the side faces of the girders. Longitudinal loads are resisted by shear in the lag screws.

As an aside, it is likely that friction between the deck panels and the girders is sufficient on its own to resist the longitudinal forces. The only longitudinal force acting on the deck panels is the braking force. This consists of 14.4% of the unfactored vertical load acting on the panels. With a 1.7 times ULS 1 load factor for braking, this corresponds to approximately 25% of the vertical load acting on the panels. The coefficient of static friction for smooth, dry wood acting on smooth, dry wood is approximately 0.30 to 0.50 (FPL 2010). The friction between the deck panels and girders would therefore be able resist at least 30% of the vertical load acting as a longitudinal force. Furthermore, the through-bolts and aluminum deck clips would offer additional normal force due to clamping action, thereby increasing the frictional resistance of longitudinal loads. The effect of friction between the deck panels and girders is ignored in this design example.

The design of the deck-to-girder connections is as follows:

F$_z$
Use 19Ø Lags with 215 mm embedment

Total length of lag screw:

$$Length = 215 + 150 = 365 \ mm$$

SPF, WET SERVICE

$$P'_{rw} = 84 \ x \ 141 \ x \ 5 \ lags \ x \ (1.0x0.67x1.0x1.0) \ x \ 10^{-3} = 39.6kN > 34kN \ (F_z \ 86\%)$$

$$L_t = 152 - 11.1 = 141mm$$

FX (MAIN PERP, SIDE PARALLEL) Q$_r$^{3}
Use 8-19Ø Lag screws @150 OC

SPF

$$P_r = 7.63 \; x \; 7.0 \; x \; 0.67 = 24.8 kN$$

$$Q_r = 6.21 \; x \; 7.0 \; x \; 0.67 = 29.1 kN > F_x = 29 kN \; (99\%)$$

$$A_m = 1200 \; x \; 215 = 258{,}00 mm^2$$

$$A_s = 1558 \; x \; 215 = 334{,}970 mm^2$$

$$\frac{A_m}{A_s} = 0.77$$

$$n_{Fe} = \frac{1}{2}(6.8 + 7.28) = 7.0$$

Note: this ignores friction between deck and girder

F_Y (MAIN PARALLEL, SIDE PERPENDICULAR) Q_r[4]
SPF

$$P_r = 7.63 \; x \; 4.85 \; x \; 1 \; x \; (1.0 x 0.67 x 1.0) = 24.8 kN$$

$$Q_r = 4.73 \; x \; 4.85 \; x \; 1 \; x \; (1.0 x 0.67 x 1.0) = 15.4 kN > F_y = 11 kN \; (71\%)$$

Assume aluminum clips @ 600 OC (2 per panel per girder)

4-19Ø RODS A307

$$T_r = 4 \; x \; 59.3 = 237 kN > F_z^{UP} = 34 kN \; (15\%)$$

75Ø WASHERS

$$Q_r = 0.8 \; x \; 5.8 \; x \; (0.67) \; x \; (4 \; x \; 4200) \; x \; 10^{-3} = 52 kN > F_z^{UP} = 34 kN \; (65\%)$$

CLIP IN BOLTED SLOT

$$Q_r = 0.8 \; x \; 5.8 \; x \; (0.67) \; x \; (4 \; x \; 25 \; x \; 125) \; x \; 10^{-3} = 39 kN > F_z^{UP} = 34 kN \; (88\%)$$

SLOT SHEAR

$$V_r = 0.8 \; x \; 1.75 \; x \; 0.67 \; x \; 0.87 \; x \; 100 \; x \; 125 \; x \; 4 \; x \; 10^{-3} = 40.6 kN > F_z^{UP} = 34 kN \; (84\%)$$

CLIPS – SINGLE SHEAR PLATE/WOOD

$$P_r = 13.4 \; x \; 0.67 = 9.0 kN/bolt \; x \; 4 = 35.9 kN > F_z^{UP} = 34 kN \; (95\%)$$

Note: check aluminum clip for bending & bolt bearing (not shown)

F$_X$^{DOWN}

$$F_z^{DOWN} = 8kN$$

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8 \ x \ 5.8 \ x \ (1.0 \ x \ 0.67 \ x \ 1.0)x(215 \ x \ 1200)x1.0x1.0x10^{-3} = 732kN > T_f$$
$$= 8 \ kN \ (1\%)$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

check loading on wheel load:

$$Q_r = 0.8 \ x \ 5.3 \ x \ (0.67) \ x \ (215x220) \ x \ 10^{-3} = 134kN > T_f = 8 \ kN \ (6\%)$$

3.2.6.2. DECK-TO-STIFFENER BEAM CONNECTION

The deck-to-stiffener beam connection is based on the work of Witmer et al. The design is as follows:

a) Stiffener beam bolt forces determined per "Reinforcing Transverse Glued-laminated Deck Panels with Through-bolted Glued-laminated Stiffener Beams" ASCE Journal of Bridge Engineering paper
b) Stiffener beam moments and shears determined per "Simplified Design Procedure for Glued-Laminated Bridge Decks" by McCatcheon and Taomi
c) Strength design per the CHBDC

1) LONGITUDINAL MOMENT AND SHEAR TO BE TRANSFERRED BY STIFFENER BEAMS

$P = 87.5kN = 19646lb \; (AXLE \; 4 \; WHEEL \; LOAD)$

$s = 1150mm = 45.3in \; (GIRDER \; SPACING)$

$R_y = \dfrac{6Ps}{1000} = 5337lb$
$\qquad = 23.8kN \; (LIVE \; LOAD \; LONGITUDINAL \; SHEAR \; TO \; BE \; TRANSFERED)$

$M_y = \dfrac{Ps}{1600}(s - 10) = 19611lb.in$
$= 2.2kNm \; (LIVE \; LOAD \; LONGITUDINAL \; MOMENT \; TO \; BE \; TRANSFERED)$

2) STIFFENER BEAM BOLT FORCES
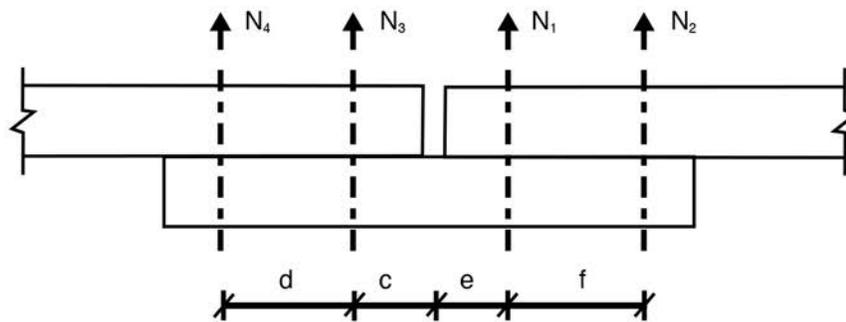
$\alpha_{LL} = 1.7 \; (ULS \; LIVE \; LOAD \; FACTOR)$

$M_f = \alpha_{LL}M_y = 3.8kNm \; (ULS \; MOMENT)$

$V_f = \alpha_{LL}R_y = 40.4kN \; (ULS \; SHEAR)$

$d = 0.317m$

$c = 0.152m$

$e = 0.152m$

$f = 0.317m$

$$R_{1M} = -\left(\frac{1}{f}\right)M_f = -11.9kNm$$

$$R_{1V} = \left(1+\frac{e}{f}\right)V_f = 59.8kN$$

$$\left.\begin{array}{l}\end{array}\right\} \quad N_1 = R_{1M} + R_{1V} = 47.9kN$$

$$R_{2M} = \left(\frac{1}{f}\right)M_f = 11.9kNm$$

$$R_{2V} = -\left(\frac{e}{f}\right)V_f = -19.4kN$$

$$\left.\begin{array}{l}\end{array}\right\} \quad N_2 = R_{2M} + R_{2V} = -7.5kN$$

$$R_{3M} = -\left(\frac{1}{d}\right)M_f = -11.9kNm$$

$$R_{3V} = -\left(1+\frac{c}{d}\right)V_f = -59.8kN$$

$$\left.\begin{array}{l}\end{array}\right\} \quad N_3 = R_{3M} + R_{3V} = -71.7kN$$

$$R_{4M} = \left(\frac{1}{d}\right)M_f = 11.9kNm$$

$$R_{4V} = \left(\frac{c}{d}\right)V_f = 19.4kN$$

$$\left.\begin{array}{l}\end{array}\right\} \quad N_4 = R_{4M} + R_{4V} = 31.1kN$$

3) SHEAR IN STIFFENER BEAM

Assume that the continuous stiffener beam can be idealized as several discrete beams of length:

$$d + c + e + f = 0.938m$$
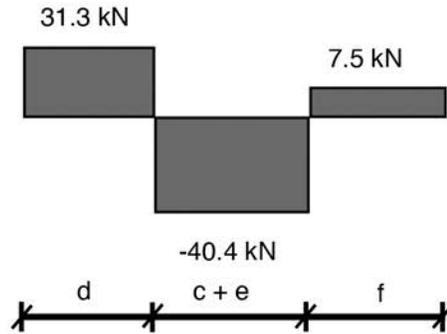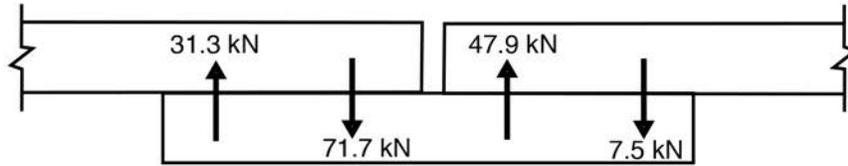
$$V_f = 0.82 \left[\frac{1}{0.938m} x\{|31.3|^5 x0.317 + |-40.4|^5 x0.304 + |7.5kN|^5 x0.317\}\right]^{0.2} = 28kN$$

$$V_r = \phi k_d k_m k_{sv} f_{vu} A/1.5$$

$$V_r = 51.3kN > V_f = 28kN \therefore OK$$

Where,

$$\phi = 0.90$$

$$k_d = 1.0$$

$$k_m = 1.0$$

$$k_{sv} = V^{-0.18} = (0.215 x 0.114 x 0.938)^{-0.18} = 1.97$$

$$f_{vu} = 1.18 \, MPa$$

$$A = 215 x 114 = 24510 mm^2$$

3.2.6.3. DIAPHRAGM-TO-GIRDER CONNECTION

The diaphragm-to-girder connection consists of a pair of threaded rods through-bolted through a routed slot along the length of the diaphragms and through the side faces of adjacent girders. The design is as follows:

RODS

$$T_f = 73kN$$

Use 22Ø A307 rod with threaded ends

$$T_r = 80.7kN > T_f = 73kN \ OK \ (91\%)$$

WASHER: 170X170X12.7

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(170x170)x1.0x1.0x10^{-3}$$

$$= 90kN > T_f = 73kN \ OK \ (82\%)$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.15 \ (RATIO = \frac{215}{38} = 5.65 > 2)$$

NOTE: Designer to also check washer plate thickness.

BEARING ON SIDE OF GIRDER

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8 x 5.8 x (1.0 x 0.67 x 1.0) x (130 x 200) x 1.0 x 1.0 x 10^{-3} = 81 kN > T_f$$

$$= 73 kN \; OK \; (90\%)$$

Where,

$\phi = 0.8$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$f_{cp} = 5.8 MPa$

$K_D = 1.0$

$K_{S_{cp}} = 0.67 \; (WET \; SERVICE \; CONDITION \; AT \; CONNECTION)$

$K_T = 1.0$

$K_B = 1.0$

$K_{Z_{cp}} = 1.0$

3.2.6.4. GIRDER-TO-BEARING CONNECTION

The girder-to-bearing connection design is as follows.

BEARING (6.5.9.2 O86-01)

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8 x 5.8 x (1.0 x 0.67 x 1.0) x (215 x 500) x 1.0 x 1.15 x 10^{-3} = 385 kN \geq F = 371 kN$$

Where,

$\phi = 0.8$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$f_{cp} = 5.8 MPa$

$K_D = 1.0$

$K_{S_{cp}} = 0.67 \; (WET \; SERVICE \; CONDITION \; AT \; CONNECTION)$

$K_T = 1.0$

$K_B = 1.0$

$K_{Z_{cp}} = 1.15 \; (RATIO = \frac{215}{38} = 5.65 > 2)$

BOLTS IN GLUED-LAMINATED TIMBER (10 O86-01)

$$P_f = 87kN$$

$$Q_f = 9kN$$

$$N_f^{3.4°} = \sqrt{87^2 + 9^2} = 87.5kN$$

$$K_{SF} = 0.67 \ (WET \ SERVICE)$$

$$K_{SV} = 0.87$$

$$K_{st} = 0.75$$

$$\theta = \text{atan}\left(\frac{9}{87}\right) = 5.9°$$

S-W-S (STEEL SIDE PLATES)

TRY 2 ROWS OF 2 BOLTS, 19Ø, A307:

$$P_r = 104kN > F_x = 87kN \ (BRITTLE \ RESISTANCE) \ OK$$

$$Q_{RS} = 17kN > F_z^{UP} = 9kN \ (SPLITTING \ RESISTANCE) \ OK$$

$$N_r = 114kN > N_f^{5.9°} = 87.5kN \ (YIELDING \ RESISTANCE) \ OK$$

$$N_r = 103kN > N_f = 87.5kN \ (BRITTLE \ RESISTANCE) \ OK$$

ANCHOR BOLTS (10 O86-01)
Use 4-13 Ø galvanized anchor bolts using epoxy system per manufacturer's requirements.

$$T_r = 33kN/BOLT \ x0.9x0.78x4 \ BOLTS = 93kN \ > F_z^{UP} = 9kN$$

$$V_r = 83kN/BOLT \ x0.9x0.42x4 \ BOLTS = 125kN \ > F_x = 87kN$$

Unity check:

$$\left(\frac{9}{93}\right)^{\frac{5}{3}} + \left(\frac{87}{125}\right)^{\frac{5}{3}} = 0.57 < 1.0 \ OK$$

ECCENTRICITY OF F_X ON ANCHORS:

$$M_f = 87 \times 0.14m = 12.2 kNm$$

$$T_f = C_f = \frac{M_f}{e} = \frac{12.2}{0.204} = 60kN \div 2\ BOLTS = 30kN/BOLT$$

$$T_r = \frac{93}{4} = 23.3 kN/BOLT$$

∴ USE LARGER BOLT: TRY 19 Ø

$$T_r = 55 \times 0.91 \times 0.88 = 44kN/BOLT > T_f = \frac{9}{4 BOLTS} + 30 = 32kN/BOLT$$

$$V_r = 107 \times 0.91 \times 0.56 = 54.5kN/BOLT > V_f = \frac{87}{4 BOLTS} = 22kN/BOLT$$

UNITY CHECK:

$$\left(\frac{32}{44}\right)^{\frac{5}{3}} + \left(\frac{22}{54.5}\right)^{\frac{5}{3}} = 0.81 < 1.0\ OK$$

Anchor bolt strength

$$T_r = 66kN > T_f = 32kN$$

$$V_r = 34kN > V_f = 22kN$$

∴ USE 4-19 Ø galvanized anchor bolt with 170 MIN. EMBED

NOTE: Designer to check bearing plate thickness for gravity and uplift forces

UPPER CLIPS (10 O86-01)

ESTIMATE SIDE FORCE

$$M_r = 1380kNm$$

$$T_f = C_f = \frac{1380}{1.55} = 886kN \ (AT \ MID - SPAN)$$

say 2% is torsional components for restraint:

$$C_f^b = 0.02x886 = 17.7kN$$

assume 200x100 bearing area on steel angle (6.5.9.2 O86-01):

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1x0.67x1.0)x(200x100)x1.0x1.0x10^{-3} = 62kN > 17.7kN \ OK$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

Note: Diaphragms near support will also restrain. Designer to check angles for bending & prying on anchor bolts.

SIDE PLATES AT BOLTS (6.5.9.2 O86-01)

Bearing on wood

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8 x 5.8 x (1.0 x 0.67 x 1.0) x (356 x 25) x 1.0 x 1.0 x 10^{-3} = 28kN > F_y = 12kN$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$$f_{cp} = 5.8 MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

Note: Designer to check plate bending

THIS IS THE END OF THE FIRST DESIGN EXAMPLE.

3.3. STRESS-LAMINATED TIMBER DECK ON GLUED-LAMINATED TIMBER GIRDERS

3.3.1. THE BRIDGE CONCEPT

3.3.1.1. DESCRIPTION

For the second design example, the bridge is a single-span, wood highway bridge carrying a two-lane highway 18 m across a stream. The bridge follows a tangent horizontal alignment and a 1.50% tangent vertical profile. The centreline of the highway is collinear with the centreline of the structure. Figures 3.3.1. and 3.3.2. illustrate an elevation view and the profile of the structure..
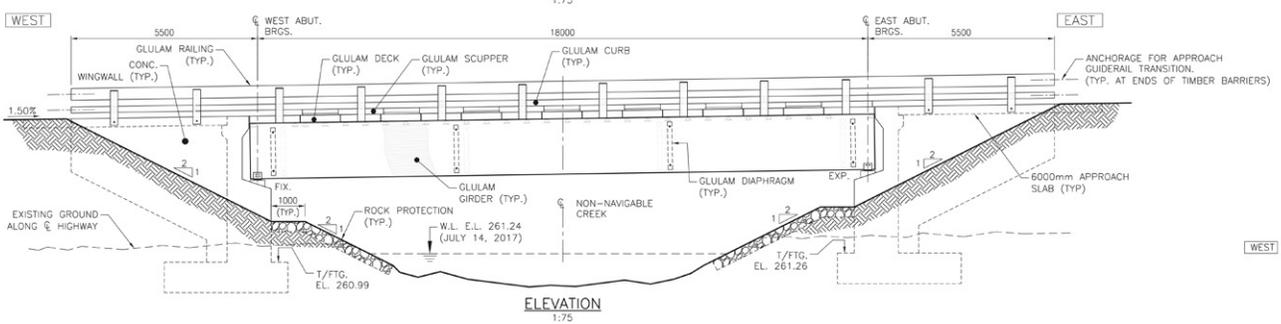
Figure 3.3.1. – Elevation of the structure
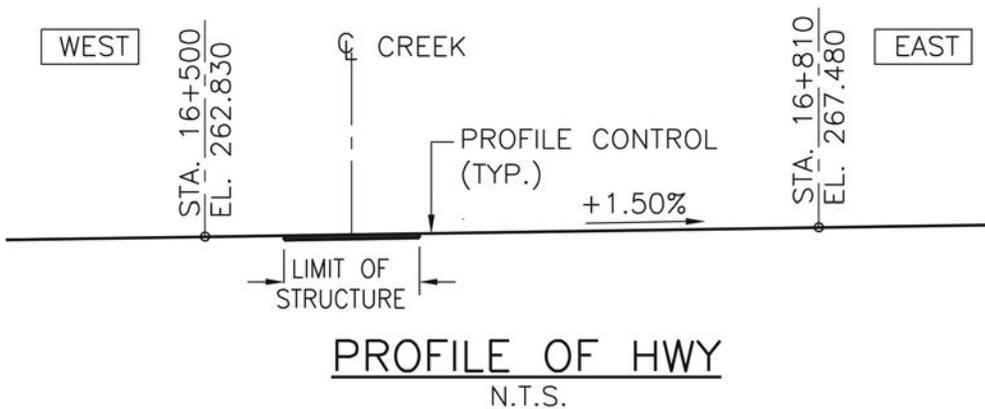
Figure 3.3.2. – Profile of the structure

The roadway cross-section was developed using the "Geometric Design Standard for Ontario Highways" document with 2002 revisions (Ontario Ministry of Transportation (MTO) 2002). It is valid for an undivided arterial road with a design speed limit of 110 km/h or less. This criterion describes the Trans-Canada Highway in many parts of the country. Figure 3.3.3. illustrates a typical cross-section of the roadway and structure.
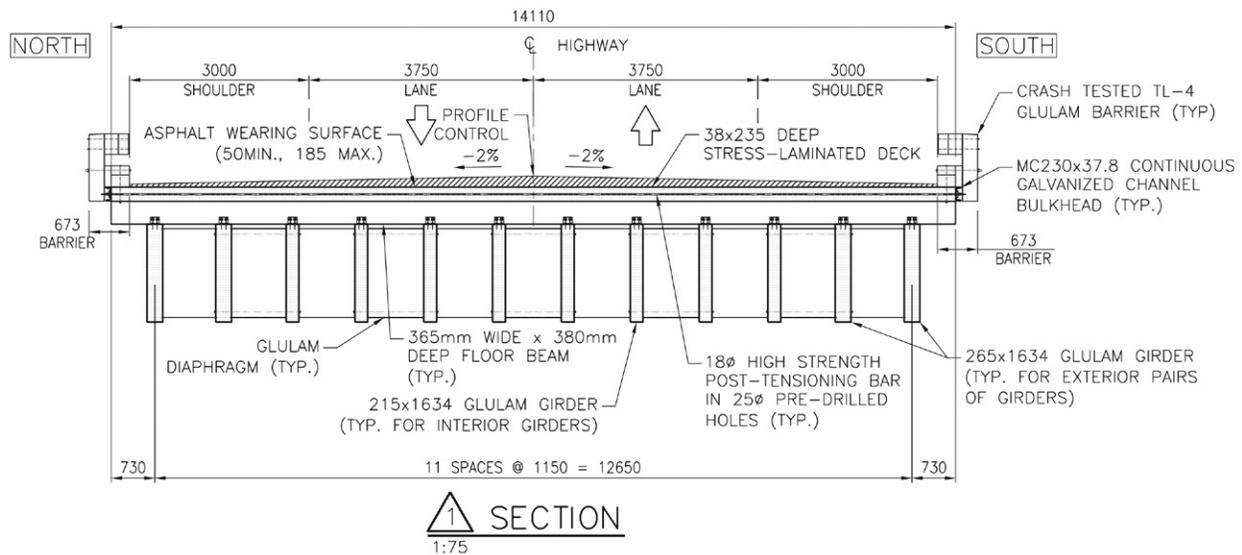
Figure 3.3.3. – Cross-section of the structure

The cross-section consists of two 3 750 mm-wide lanes, two 3 000 mm-wide shoulders, and two 305 mm-wide glued-laminated curbs. The resulting total deck width is 14 110 mm, and the total travelled deck width is 13 500 mm. Highway shoulders are sometimes reduced at a bridge to reduce the bridge width, but this approach is often not preferable from a safety standpoint. The use of the 3 000 mm-wide shoulders in this design example demonstrates that using a wood bridge does not have to result in sacrifices to the highway geometry.

A crash-tested TL-4 timber railing is utilized as a bridge barrier to prevent traffic from exiting off the sides of the bridge. This railing was developed and crash-tested in the United States by the Forest Products Laboratory, specifically for longitudinally spanning wood decks (Ritter et al. 1995). The railing consists of a continuous upper rail, a continuous curb rail, vertical posts spaced at no more than 1905mm (6'-3") on centre, a discrete length spacer block at each post, a discrete length scupper block at each post, and metal fasteners and splices. All wood components are glued-laminated members. The railing has been adapted to fit the geometry of the stress-laminated deck by thickening the spacer blocks and by adding a spacer block to offset the post from the steel channel distribution bulkhead. The post spacing has been chosen so as not to interfere with the post-tensioning anchorages of the stress-laminated deck. Figure 3.3.4. illustrates a typical cross-section of the railing at a post.

Figure 3.3.4. – Cross-section of the railing

The bridge features an asphalt wearing surface with a 2% crossfall down to the curbs from the crown. The crown is located at the centreline of the structure. The asphalt thickness varies from 185 mm at the crown to 50 mm at the inside face of the curbs.

Bridge deck waterproofing is not specified in this design example, but it should be part of a real-life wood bridge design. There are several approaches by which the deck can be waterproofed and paved. Refer to the work of Eriksson et al. (2003) and Weyers et al. (2001) for a comprehensive examination of waterproofing and paving systems for wood bridges. Local jurisdictions may also provide guidance on the subject. Ultimately, the wood preservative used to treat the deck must be compatible with the waterproofing and pavement. There have been several instances where interaction between these three has resulted in degradation of the waterproofing and paving, as well as leaching of the preservative and asphalt into the surrounding environment.

The deck is a longitudinal stress-laminated wood deck that extends across the full width and length of the bridge. It is discretely supported every 3.0 m on wood floor beams and behaves as a six-span continuous longitudinal flexural member. The deck is comprised of 38 x 235 sawn wood laminations oriented to form a vertically laminated deck. The laminations are transversely post-tensioned using internal, unbonded, galvanized post-tensioning bars. Full-width circular holes are drilled through the laminations to facilitate passage of the post-tensioning bars. The bars produce sufficient compressive and frictional forces between adjacent laminations such that the deck behaves as an orthotropic plate, as opposed to individual, narrow beams. The post-tensioning bars are anchored along the sides of the deck. Each anchorage consists of a steel anchorage plate and nut. The anchorage plates bear against a continuous steel bulkhead comprised of a MC230x37.8 channel. The bulkhead is utilized to distribute the post-tensioning forces between the discretely located post-tensioning bars. Multiple stressing operations are undertaken to ensure that the long-term prestressing force, after losses, is enough for orthotropic plate behaviour. The post-tensioning bar extensions and anchorage nuts are covered by a protective plastic cap after all stressing operations are completed. The cap, which serves to protect against corrosion and tampering, is packed with anti-corrosion paste. The post-tensioning bars are also coated with this anti-corrosion paste. Figure 3.3.5. illustrates the typical deck post-tensioning anchorage.
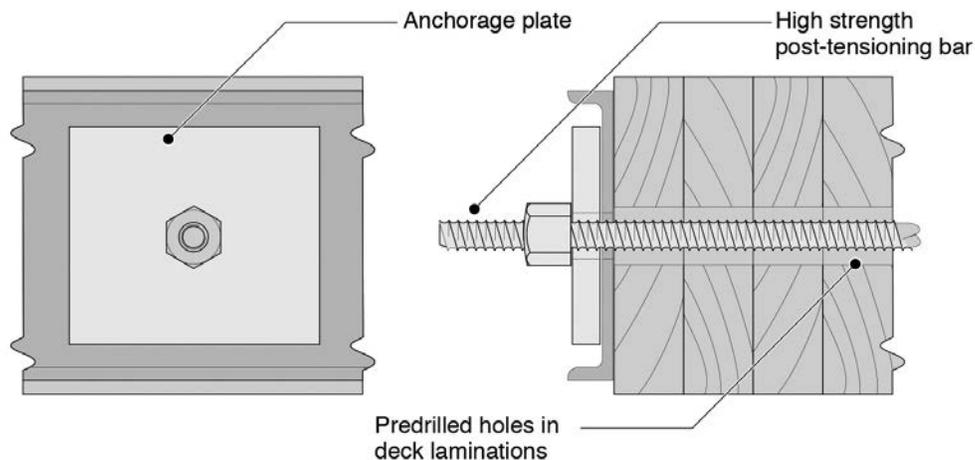
Figure 3.3.5. – Typical deck post-tensioning anchorage
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

The deck is supported every 3.0 m on a glued-laminated floor beam. Each floor beam is 365 mm wide and 380 mm deep. The floors beams are supported directly on top of the girders and span the full width of the deck as a continuous flexural member. The deck is through-bolted to the floor beams in the spaces between the girders. The floor beams are connected to the girders using steel angles. A pair of angles are through-bolted across the width of each floor beam at each girder. The angles are fastened to the girders using lag screws. Figure 3.3.6. illustrates the typical connection between the deck and floor beams. Figure 3.3.7. illustrates the typical connection between the floor beam and girders.
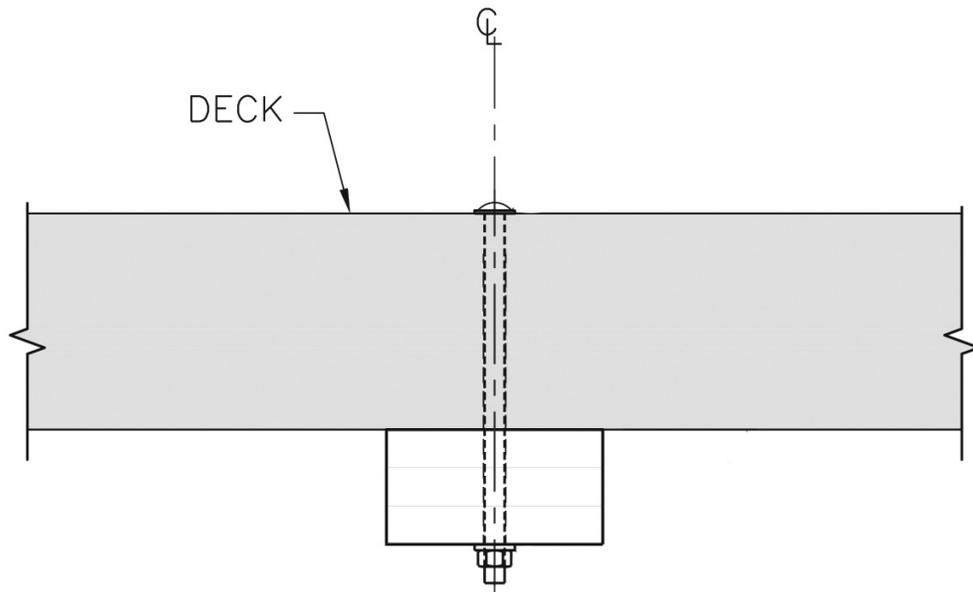
Figure 3.3.6. – Typical connection between deck and floor beams
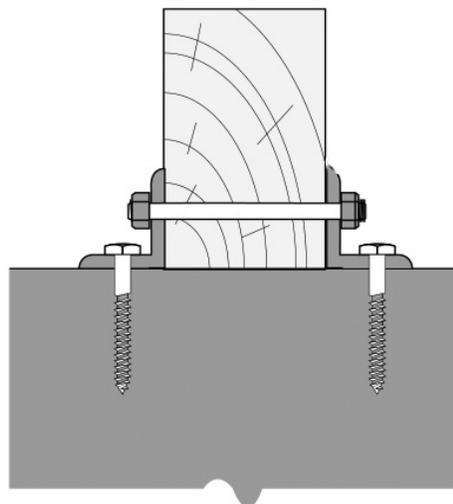
Figure 3.3.7. – Typical connection between floor beams and girders
(Photo credit courtesy of the Canadian Wood Council's Wood Highway Bridges (1994))

Some owners might not prefer the proposed connection between the floor beams and girders because the field installation of the lag screws into the tops of the girders provides a direct path for moisture to penetrate the cores of the girders. The stress-laminated deck should be watertight if it designed properly, thus precluding the moisture ingress, but some owners have a strict policy against fasteners penetrating the top surface of girders. As a result, the designer may desire to reorient the connection by positioning the steel angles along the side faces of the girders, through-bolting across the width of the girders, and fastening the lag screws upward into the underside of the floor beams.

The floor beams are supported by 12, parallel, constant-depth, S-P (Spruce-Lodgepole Pine-Jack Pine) glued-laminated girders. The two exterior-most girders on each side of the bridge is 265 mm and 1634 mm deep. Each interior girder is 215 mm wide and 1634 mm deep. Glued-laminated diaphragms positioned at the span quarter-points are used to stabilize the girders against lateral-torsional buckling and to transmit transverse loads between girders.
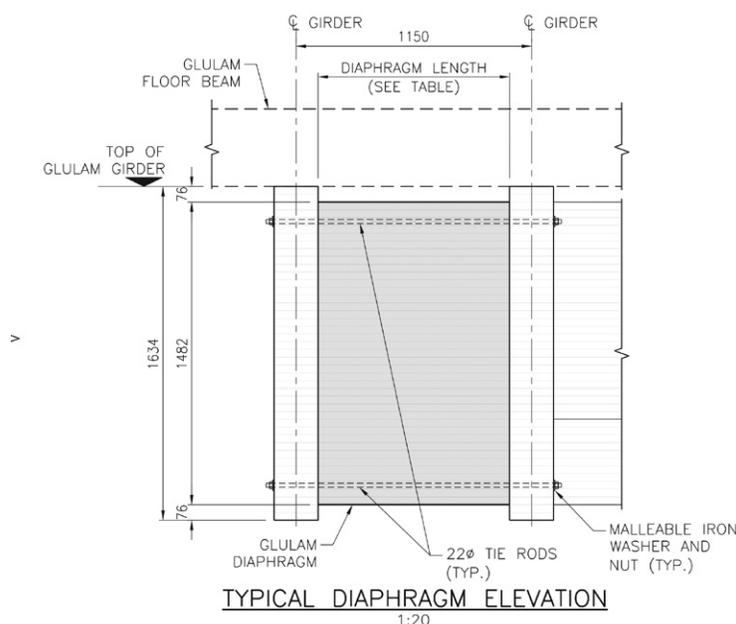
Figure 3.3.8. – Typical cross-section at diaphragm

Each girder is supported at each end by an elastomeric bearing pad. These bearing pads are part of a bearing assembly that also consists of a steel bearing plate, a bed of non-shrink grout, and the reinforced concrete bearing pedestal that projects above the abutment bearing seat. Figure 3.3.9. illustrates a typical bearing assembly.
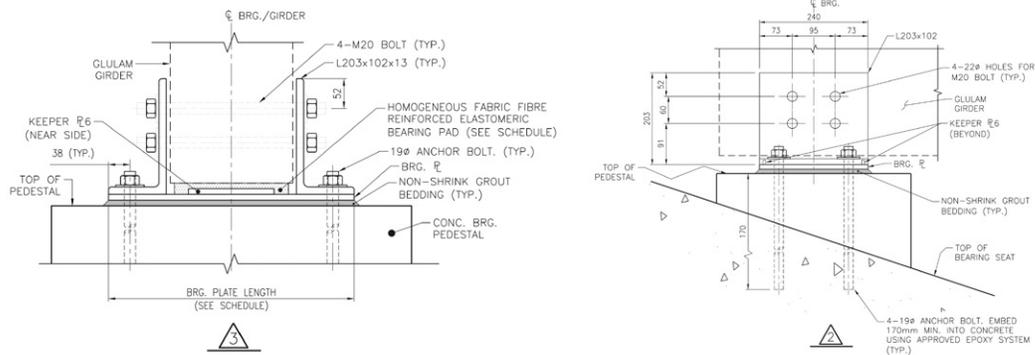
Figure 3.3.9. – Typical bearing assembly

The substructure has reinforced concrete abutments founded on spread footings. There is no limit to the number of substructure and foundation combinations that are compatible with this example bridge concept. The only requirement is that they be capable of safely receiving the loads transmitted from the superstructure and detailed in a way that drains water, debris and snow buildup away from the wood elements.

3.3.1.2. ASSUMED STRUCTURAL BEHAVIOUR

The stress-laminated deck serves to transmit permanent and transitory gravity loads to the floor beams by means of out-of-plane shearing and bending. It also serves to transmit transverse and longitudinal loads to the floor beams through in-plane shearing, bending, and axial forces. Furthermore, the deck, in conjunction with the floor beams, enables the transverse distribution of discrete transitory loads amongst the girders. The deck also resists the loads acting on the timber railings due to vehicular impact through a combination of in-plane tension and out-of-plane shearing and bending.

The floor beams serve to transmit all loads from the deck to the girders. They also provide most of the transverse stiffness of the bridge, as the floor beams are much stiffer in their longitudinal direction than the stress-laminated deck is in its transverse direction. The floor beams in this design example have been oversized for strength to increase the transverse stiffness to the bridge. This approach was taken to improve live load sharing between girders. A designer may wish to take this approach when there is a significant difference between the structural demand of the exterior and interior girders.

The floor beams also enable the deck to span in the longitudinal direction. Placing a stress-laminated deck directly on top of the girders would result in primarily one-way bending in the deck in the transverse direction. This behaviour would necessitate the use of a transverse stress-laminated deck, as opposed to a longitudinal stress-laminated deck. The former features post-tensioning anchorages running across the width of the deck at each end. This approach is not desirable, as it means that traffic will operate directly above the anchorages, which greatly increases their risk of corrosion, in addition to hindering the necessary post-tensioning restressing operations. Longitudinal stress-laminated decks are generally preferred for these reasons, and there exist very few transverse stress-laminated decks.

The girders comprise the gravity load-carrying system in the longitudinal direction. They are assumed to act non-compositely with the floor beams and deck, although some degree of composite action likely exists due to friction and the mechanical connections between all components. The girders carry vertical loads by in-plane shear and bending, transverse loads by out-of-plane shear and bending, and longitudinal loads by axial tension and compression.

The diaphragms are the assumed means of preventing twisting of the girders, although the floor beams and deck also serve that purpose due to their inherent out-of-plane flexural stiffness. The unbraced length for checking lateral-torsional buckling of the girders in the completed bridge is assumed to be the spacing between successive rows of diaphragms.

Wind acting on the deck, on the railings, and on live load is transmitted into the girders via the deck and floor beams. The transverse wind force, in addition to transverse wind acting directly on the girders, is carried across the main span to the girder supports by means of the girders bending and shearing out-of-plane. The diaphragms serve to share these out-of-plane loads amongst all the girders.

Longitudinal braking loads are resisted by the deck as in-plane forces. The use of a full-length deck, in conjunction with the post-tensioning, enables the braking forces to be resisted by the entire deck. Shearing and bending of the floor beams enables transmission of the braking force to the girders. These forces are then transmitted to the bearings by means of axial tension and compression in the girders.

Loads are transmitted to the substructure via the elastomeric bearing pads at the abutments. The bearing pads serve to transfer vertical, transverse, and longitudinal loads to the abutments, while allowing for rotation, expansion, and contraction of the girders.

The steel bearing plates provide a clean surface onto which to seat the bearing pads. They can be beveled in thickness to suit the roadway profile, girder camber, and bridge self-weight rotations. These plates also allow for anchorage of the bearing assemblies into the abutment pedestals by use of anchor bolts. When fitted with keeper bars or drift pins, they also serve as a restraint against bearing pad migration due to transverse and longitudinal forces acting on lightly loaded bearing pads.

The concrete pedestals transfer the loads from the bearing assemblies down into the abutment stem. No matter the workmanship, there are always imperfections on hardened concrete surfaces. Accordingly, a thin bed of non-shrink grout is specified under the steel bearing plates to establish firm contact with the concrete bearing pedestals.

3.3.2. MATERIALS

All glued-laminated timber used in this design example is assumed to be species combination S-P. S-P dimension lumber is plentiful in many parts of the country, especially Ontario. The CHBDC does not presently provide material properties for the use of S-P glued-laminated timber with its design equations; however, the 2019 version of the CHBDC is anticipated to include these material properties. Accordingly, the glued-laminated timber material properties for this design example have been assumed from CSA O86 and modified, as necessary, to be compatible with the CHBDC design equations.

3.3.2.1. GLUED-LAMINATED TIMBER GIRDERS, FLOOR BEAMS, & DIAPHRAGMS

The following material properties are from Table 7.3 of CAN/CSA-O86-14 Engineering Design in Wood. The girders, floor beams, and diaphragms are a horizontally laminated 20f-E stress grade comprised of Spruce-Lodgepole Pine-Jack Pine species laminations. The nominal values have not been adjusted by service condition or treatment factors, or for the effects of impact and cyclical loading.

f_b = 25.6 MPa bending moment (positive)

f_b = 19.2 MPa bending moment (negative)

f_v = 1.75 MPa longitudinal shear

f_c = 25.2 MPa compression parallel to grain

f_{cb} = 25.2 MPa compression parallel to grain combined with bending

f_{cp} = 5.8 MPa compression perpendicular to grain

f_{tn} = 17.0 MPa tension at net section

f_{tg} = 12.7 MPa tension at gross section

f_{tp} = 0.51 MPa tension perpendicular to grain

E_{50} = 10300 MPa 50th percentile modulus of elasticity

E_{05} = 8960 MPa 5th percentile modulus of elasticity

The 5th percentile modulus of elasticity is calculated as 87% of the 50th percentile modulus of elasticity for this design example per CSA O86.

The properties from CSA O86 must be modified to reflect the service condition and preservative treatment of the girders, floor beams, and diaphragms to be compatible with the CHBDC design equations. A "semi-wet" service condition is assumed, acknowledging that the girders should not gain significant moisture in service due to their protection by the deck. The girders, floor beams, and diaphragms will be pressure-treated with a wood preservative to enhance their durability. They will be incised to increase the depth of preservative penetration.

The following are service condition factors for glued-laminated timber based on "semi-wet" conditions.

K_{Sb} = 0.90   service condition factor for bending

K_{Sv} = 0.94   service condition factor for shear

K_{Sc} = 0.88   service condition factor for compression parallel to grain

K_{Scp} = 0.84   service condition factor for compression perpendicular to grain

K_{St} = 0.88   service condition factor for tension parallel to grain

K_{Stp} = 0.93   service condition factor for tension perpendicular to grain

K_{SE} = 0.95   service condition factor for modulus of elasticity

Note that "wet" service factors may be appropriate for certain members or bridge configurations and the designer will need to make that judgement. CSA O86 provides service factors for "wet" conditions. In all cases, connections should be designed using "wet" service factors while members may be designed with either "semi-wet" or "wet" factors. Steel connection hardware tends to result in moisture condensation, thereby increasing the moisture content of timbers in connections.

The following treatment factor for incised glued-laminated timber was taken from the proposed provisions for the forthcoming 2019 edition of the CHBDC.

K_T = 1.00   modification factor for treatment

The modification factor for treatment has been taken to be equal to unity, as incising is not considered to reduce the strength of glued-laminated timber (APA 2013).

The modified material properties that can be used directly with the CHBDC equations for design, accounting for service condition and preservative treatment, are as follows:

f_b = 23.0 MPa bending moment (positive)

f_b = 17.3 MPa bending moment (negative)

f_v = 1.65 MPa longitudinal shear

f_c = 22.2 MPa compression parallel to grain

f_{cb} = 22.2 MPa compression parallel to grain combined with bending

f_{cp} = 4.9 MPa compression perpendicular to grain

f_{tn} = 15.0 MPa tension at net section

f_{tg} = 11.2 MPa tension at gross section

f_{tp} = 0.47 MPa tension perpendicular to grain

E_{50} = 9785 MPa 50th percentile modulus of elasticity

E_{05} = 8512 MPa 5th percentile modulus of elasticity

3.3.2.2. STRESS-LAMINATED TIMBER DECK

The deck is comprised of dimension lumber post-tensioned together to form a stress-laminated deck. The dimension lumber will be lodgepole pine select structural grade. Jack pine, red pine, and white pine would also be acceptable species for the deck laminations. The following material properties are from Table 9.12 of the CHBDC for Spruce-Pine-Fir structural joists and planks of select structural grade.

f_b = 11.8 MPa bending moment

f_v = 1.2 MPa longitudinal shear

f_c = 8.5 MPa compression parallel to grain

f_{cp} = 3.0 MPa compression perpendicular to grain

f_t = 6.1 MPa tension at net section

E_{50} = 9400 MPa 50th percentile modulus of elasticity

E_{05} = 6700 MPa 5th percentile modulus of elasticity

These deck material properties have been adjusted for service condition and treatment. The CHBDC assumes a wet service condition, which is appropriate for a wood bridge deck. It also assumes that the wood has undergone preservative treatment and has been incised. Both assumptions are appropriate, as clause 9.17.1 of the CHBDC requires all wood in permanent structures to be preservative treated in accordance with the CSA O80 Series of Standards.

This design example makes use of select structural dimension lumber. The designer should consider the cost premium associated with the use of select structural versus No. 1/No. 2 grade dimension lumber when designing the deck.

3.3.2.3. MATERIAL PROPERTIES FOR COMPUTER STRUCTURAL ANALYSIS

Material properties of the glued-laminated members and stress-laminated deck are provided in Sections 3.1 and 3.2 of this design example. Additional orthotropic material properties are required for three-dimensional computer modelling and structural analysis of wood structures. These properties were estimated for the glued-laminated members and stress-laminated deck using Table 5-1 of the Wood Handbook – Wood as an Engineering Material (FPL 2010) for Lodgepole Pine as follows:

E_T / E_L = 0.068   ratio of tangential elastic modulus to the longitudinal elastic modulus

E_R / E_L = 0.102   ratio of radial elastic modulus to the longitudinal elastic modulus

G_{LR} / E_L = 0.049   ratio of shear modulus in the longitudinal-radial plane to the longitudinal elastic modulus

G_{LT} / E_L = 0.046   ratio of shear modulus in the longitudinal-tangential plane to the longitudinal elastic modulus

G_{RT} / E_L = 0.005   ratio of shear modulus in the radial-tangential plane to the longitudinal elastic modulus

All properties are given for wood at a moisture content of 12%.

Based on Tables 5-1 and 5-2 of the Wood Handbook, the structural analysis properties for the glued-laminated <u>girders, floor beams, and diaphragms</u> are as follows:

E_L	=	9785	MPa	50th percentile modulus of elasticity parallel to grain
E_T	=	665	MPa	50th percentile modulus of elasticity perpendicular to grain in tangential direction
E_R	=	998	MPa	50th percentile modulus of elasticity perpendicular to grain in radial direction
G_{LR}	=	479	MPa	50th percentile shear modulus in longitudinal-radial plane
G_{LT}	=	450	MPa	50th percentile shear modulus in longitudinal-tangential plane
G_{RT}	=	49	MPa	50th percentile shear modulus in radial-tangential plane
μ_{LR}	=	0.316		Poisson's ratio in radial direction due to applied stress in longitudinal direction
μ_{LT}	=	0.347		Poisson's ratio in tangential direction due to applied stress in longitudinal direction
μ_{RT}	=	0.469		Poisson's ratio in tangential direction due to applied stress in radial direction
μ_{TR}	=	0.381		Poisson's ratio in radial direction due to applied stress in tangential direction
μ_{RL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in radial direction
μ_{TL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in tangential direction

Similarly, the structural analysis properties for the stress-laminated <u>deck</u> are as follows:

E_L	=	9400	MPa	50th percentile modulus of elasticity parallel to grain
E_T	=	639	MPa	50th percentile modulus of elasticity perpendicular to grain in tangential direction
E_R	=	959	MPa	50th percentile modulus of elasticity perpendicular to grain in radial direction
G_{LR}	=	461	MPa	50th percentile shear modulus in longitudinal-radial plane
G_{LT}	=	432	MPa	50th percentile shear modulus in longitudinal-tangential plane
G_{RT}	=	47	MPa	50th percentile shear modulus in radial-tangential plane
μ_{LR}	=	0.316		Poisson's ratio in radial direction due to applied stress in longitudinal direction
μ_{LT}	=	0.347		Poisson's ratio in tangential direction due to applied stress in longitudinal direction
μ_{RT}	=	0.469		Poisson's ratio in tangential direction due to applied stress in radial direction
μ_{TR}	=	0.381		Poisson's ratio in radial direction due to applied stress in tangential direction
μ_{RL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in radial direction
μ_{TL}	=	0		Poisson's ratio in longitudinal direction due to applied stress in tangential direction

3.3.3. LOADS & IMPOSED DEFORMATIONS

3.3.3.1. SELF-WEIGHT

The unit weights of the deck, floor beams, girders, and diaphragms are taken as 6.0 kN/m³ for softwood, per Table 3.4 of the CHBDC. The following table summarizes the linear weight of these components:

Component	Width		Depth		Cross-Sectional Area		Linear Weight	
deck (per metre width)	1000	mm	235	mm	0.235	m²	1.41	kN/m
floor beam	365	mm	380	mm	0.139	m²	0.83	kN/m
exterior girder	265	mm	1634	mm	0.433	m²	2.60	kN/m
interior girder	215	mm	1634	mm	0.351	m²	2.11	kN/m
diaphragm	130	mm	1482	mm	0.193	m²	1.16	kN/m

The design of the exterior portion of the deck also considers the weight of the continuous steel channel bulkhead running along the sides of the deck. These members are MC230x37.8 sections with a linear weight of 0.372 kN/m.

3.3.3.2. SUPERIMPOSED DEAD LOADS

3.3.3.2.1. WEARING SURFACE

The bridge will be paved with an asphalt wearing surface having a unit weight of 23.5 kN/m³. The asphalt varies from 185 mm thickness at the crown to 50 mm thickness at the face of the timber railings, resulting in a 2% crossfall. Recalling that the girder spacing is 1150 mm, the average asphalt thickness for either of the two interior girders nearest the crown is

$$t_{ws,int} = \frac{(185\ mm - 1150\ mm \times 0.02) + 185\ mm}{2} = \frac{162\ mm + 185\ mm}{2} = 174\ mm$$

The deck overhang is 730 mm and the timber railing curbs are 305 mm wide, thus the average asphalt thickness for either of the two exterior girders is

$$t_{ws,ext} = \frac{(50\ mm + [730\ mm - 305\ mm + 0.5 \times 1150\ mm] \times 0.02) + 50\ mm}{2}$$
$$\times \frac{(730\ mm + 0.5 \times 1150\ mm - 305\ mm)}{730\ mm + 0.5 \times 1150\ mm} = \frac{70\ mm + 50\ mm}{2} \times 0.77 = 46\ mm$$

These two asphalt thicknesses are used to design the interior and exterior girders, respectively.

Similarly, the interior and exterior strips of deck are designed for an average asphalt thickness of 180 mm and 40 mm, respectively.

3.3.3.2.2. BARRIERS

The bridge barrier is a crash-tested TL-4 timber railing. This barrier has an unfactored linear weight of 1.4 kN/m. The frontal area for horizontal wind on the barrier is 0.665 m²/m. The centroid of the frontal area is 589 mm above the mid-depth of the glued-laminated deck. The frontal area of the barriers that overlaps with the horizontal projection of the deck has been neglected from these calculations. This frontal area is considered to belong to the deck. Calculations concerning the barrier weight and frontal area are not included in this design example.

3.3.3.3. WIND

Both vertical and horizontal wind loads are assumed to act simultaneously, per clause 3.10.2.1 of the CHBDC. For this design example, it is assumed that the reference wind pressure for a one-in-fifty return period wind load is 465 Pa.

3.3.3.3.1. VERTICAL WIND

The vertical wind load is calculated in accordance with clause 3.10.2.3 of the CHBDC. The vertical wind load acting on the superstructure is as follows:

q_{50} = 465 Pa   Hourly mean reference wind pressure for a 50-year return period

C_e = 1.0   wind exposure coefficient

C_g = 2.0   wind gust effect coefficient

C_v = 1.0   vertical wind load coefficient

F_v = 930 Pa   vertical wind load per unit exposed plan area

The vertical wind load is considered to act both upwards and downwards. Two vertical wind load applications are considered for both upward and downward wind:

- Uniform load acting over the entire bridge plan area
- Eccentric wind load with the centroid of the total wind load
  acting at the windward quarter-point

The eccentric wind load results in a wind pressure that varies linearly across the deck width. By geometry, the magnitude of the wind pressures at the windward and leeward sides of the deck for downward acting vertical wind, are $2.5 \times F_v$ and -0.5, respectively. For downward acting vertical wind, those magnitudes become $-2.5 \times F_v$ and $0.5 \times F_v$, respectively, at the windward and leeward sides of the deck.

3.3.3.3.2. HORIZONTAL WIND ON SUPERSTRUCTURE

The horizontal wind load acting on the superstructure was calculated in accordance with clause 3.10.2.2 of the CHBDC. The horizontal wind load acting on the superstructure is as follows:

q_{50}	=	465	Pa	Hourly mean reference wind pressure for a 50-year return period
C_e	=	1.0		wind exposure coefficient
C_g	=	2.0		wind gust effect coefficient
C_h	=	2.0		horizontal wind load coefficient
F_h	=	1860	Pa	horizontal wind load per unit exposed frontal area

The horizontal wind load acts on the exposed frontal area of the structure, including the railings, deck, floor beams, and girders. The exposed frontal area of the railings is the total area of railings above the top of the deck panels. Recall that this frontal area is equal to 0.665 m² per metre. The resulting unfactored uniformly distributed load due to wind acting on the railing is

$$P_{h,railing} = \frac{0.665 \; m^2}{m} \times 1.860 \; kPa = 1.24 \; kN/m$$

The bridge width is large enough such that the shielding factors in clause C3.10.2.2 of the CHBDC commentary do not apply for horizontal wind acting on the railings. Thus, horizontal wind load is considered to act on the exposed frontal area of each timber railing.

The exposed frontal area of the deck and floor beams is the product of their width and thickness. The resulting unfactored uniformly distributed loads due to wind acting on the deck and floor beams, respectively, is

$$P_{h,deck} = 0.235 \; m \times 1.860 \; kPa = 0.44 \; kN/m$$

$$P_{h,fb} = 0.365 \; m \times 0.380 \; m \times 1.860 \; kPa = 0.26 \; kN \; per \; floor \; beam$$

As noted in clause C3.10.2.2 of the CHBDC commentary, most highway bridges, including slab-on-girder bridges, behave aerodynamically as single bodies. Consequently, it is only necessary to apply horizontal wind load to the exposed frontal area of the windward exterior girder. The resulting unfactored uniformly distributed load due to wind acting on an exterior girder is

$$P_{h,girder} = 1.634\ m \times 1.860\ kPa = 3.04\ kN/m$$

3.3.3.3.3. HORIZONTAL WIND ON LIVE LOAD

The horizontal wind load acting on live load is calculated in accordance with clause 3.10.2.4 of the CHBDC. The calculation is the same as for the horizontal wind load acting on the superstructure, but the magnitude of the horizontal wind load coefficient, C_h, is reduced from 2.0 to 1.2, resulting in a horizontal wind load acting on live load equal to 1116 Pa.

The horizontal wind load acting on live load is assumed to act uniformly over a height of 3.0 m above the roadway surface along the length of the structure. The frontal area within that envelope that has already been considered in the calculation of the horizontal wind load acting on the superstructure (i.e. the timber railings) is neglected from the horizontal wind load acting on live load calculation. Consequently, the total exposed area for wind on live load, excluding the exposed frontal area of a timber railing is:

$$A_{fr,LL} = 3.0\ m\ \times 1.0\ m + 0.050\ m - 0.665\ m^2 = 2.385\ m^2/m$$

The resulting unfactored uniformly distributed load due to wind acting on live load is

$$P_{h,LL} = \frac{2.385\ m^2}{m} \times 1.116\ kPa = 2.66\ kN/m$$

3.3.3.4. LIVE LOAD

The bridge is subject to vertical and longitudinal live loads due to the weight of moving vehicles and braking forces, respectively.

3.3.3.4.1. VERTICAL LIVE LOAD

Per clause 3.8 of the CHBDC, the design of the bridge considers the vertical live load effects caused by each of the CL-625-ONT truck and the CL-625-ONT lane load. The CL-625-ONT truck is a five-axle truck of 18 m length, with a total weight of 625 kN. The CL-625-ONT lane load is a uniformly distributed load of 9 kN/m superimposed with 80% of the CL-625-ONT truck load. Figure 3.3.10. illustrates these loads as they are depicted in the CHBDC.
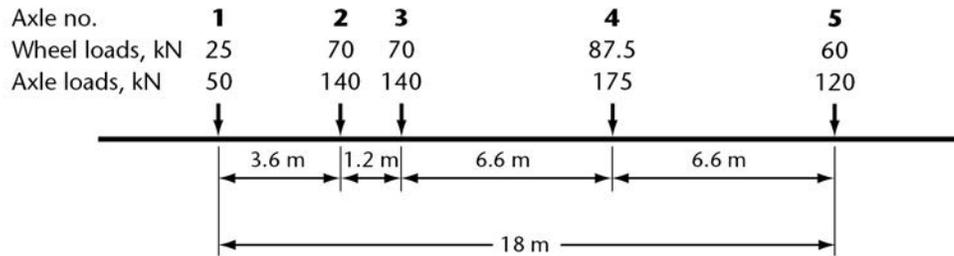
Figure A3.4.1
CL-625-ONT Truck
(See Clause A3.4.1.)
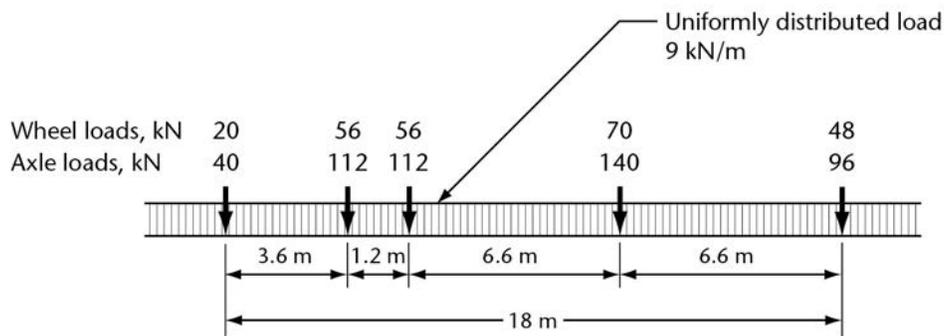
Figure A3.4.2
CL-625-ONT Lane load
(See Clause A3.4.1.)

Figure 3.3.10. – CHBDC vertical live load (Source: Clause A3.4.1, Figure A3.4.1 CL-625-ONT Truck; Clause A3.4.1, Figure A3.4.2 CL-625-ONT Lane Load; Clause 3.8.3.2., Figure 3.2 CL-W Truck – Canadian Highway Bridge Design Code. © 2017 Canadian Standards Association)

At the SLS 1 and ULS limit states, the vertical live load per design lane is the greater of the CL-625-ONT truck amplified by the dynamic load allowance or the CL-625-ONT lane load without dynamic load allowance. At the SLS 2 and FLS limit states, the vertical live load for the entire bridge is the CL-625-ONT truck amplified by the dynamic load allowance and centred across the width of one design lane.

Truck axles and portions of the uniformly distributed lane load that reduce the load effect are neglected. The bridge is a single span in this design example, so no component of the vertical live loading will reduce the load effects in the girders; however, it is necessary to neglect truck axles and portions of the uniformly distributed lane load to maximize load effects in the six-span continuous deck.

The vertical live load is positioned both longitudinally along the length a design lane and transversely across the width of a design lane, to maximize the load effect. The transverse positioning of the truck across the width of a design lane respects the truck clearance envelope.

The vertical live load caused by the CL-625-ONT truck is increased by a factor equal to one plus the dynamic load allowance, per clause 3.8.4.5 of the CHBDC. The dynamic load allowance is a scalar factor that converts the dynamic effects of vertical live load to an equivalent static load. Dynamic effects are typically caused by the interaction of a moving vehicle and the bridge, static and vibratory deflections, and discrete and random irregularities in the riding surface (CSA 2014b).

Vertical Live Load	Dynamic Load Allowance
one axle of the CL-625-ONT truck	0.50
any two axles or axles 1-3 of the CL-625-ONT truck	0.40
any three of more axles of the CL-625-ONT truck, except for axles 1-3	0.30

The dynamic load allowance is reduced by 30% because the bridge is comprised entirely of wood components, per clause 3.8.4.5.4 of the CHBDC. Wood bridges are often short-span structures. This reduction accounts for the improved response of short-span bridges to dynamic loads. It also accounts for the inherent higher damping observed in wood relative to other conventional bridge materials (CSA 2014b).

The bridge has a total width of 14.110 m and a total travelled width of 13.500 m. The number of design lanes is as follows:

W_c = 13.500 m   total travelled bridge width

n = 2 or 3   lanes   number of design lanes per Table 3.5 of CHBDC

W_{e2} = 6.750 m   design lane width for two-lane configuration

W_{e3} = 4.500 m   design lane width for three-lane configuration

The travelled bridge width is such that both two and three design lanes must be considered. Each design lane configuration consists of one or more lanes loaded with a single CL-625-ONT truck or CL-625-ONT lane load. The modification factor for multi-lane loading, per Table 3.6 of the CHBDC, is considered for vertical live load to account for the unlikely presence of more than one design vehicle acting simultaneously on the bridge. Those factors are as follows for this design example:

Number of Loaded Design Lanes	Modification Factor
1	1.00
2	0.90
3	0.80

3.3.3.4.2. BRAKING FORCE

The braking force is calculated as the sum of 180 kN plus 10% of the uniformly distributed portion of the lane for one design lane. The bridge span is 18.000 m. The girders overhang the centreline of abutment bearings by 0.272 m, resulting in a total deck length of 18.544 m. The braking force is thus:

$$F_{br} = 180 \ kN + 0.10 \times 9 \frac{kN}{m} \times 18.544 \ m = 197 \ kN$$

The behaviour of a stress-laminated deck as an orthotropic plate allows for the braking force to be distributed along the length of the bridge. This distribution will allow for multiple floor beams and connections to resist the braking force.

3.3.3.5. LOAD COMBINATIONS

The load factors and load combinations prescribed in clause 3.5 of the CHBDC are used for this design example, as applicable.

Exceptional loads are ignored in this design example, meaning that ULS load combinations 5 to 8 are not examined. The designer should consider these load combinations if exceptional loads are a possibility. ULS load combination 9 is also not considered because the wood bridge in this design example is very light relative to the weight of the live loads that it is designed for. The designer should consider these load combinations if the wood bridge happens to carry significant permanent load.

The fatigue limit state (FLS) is not considered in this design example because fatigue has traditionally not been considered for wood bridge design (Ritter 1992). Refer to the Wood Handbook – Wood as an Engineering Material (Forest Product Laboratory (FPL) 2010) for further information on fatigue in wood members. The FLS stresses in all metal connections should be checked in accordance with the CHBDC for real-life designs. These checks are not provided in this design example.

3.3.4 STRUCTURAL ANALYSIS

3.3.4.1 STRESS-LAMINATED DECK

A one metre wide interior strip and exterior strip of deck are considered for the design of the stress-laminated deck. Both strips are analyzed as six-span continuous beams, per the span arrangement of the deck. The effects of self-weight, wearing surface, barriers, wind load, and live load are considered. The self-weight is taken as that of the design strip. The exterior strip self-weight also includes the weight of the steel channel bulkhead. The wearing surface and wind loads are assigned based on the tributary strip widths. The weight of a barrier is assumed to be carried exclusively by the exterior strip. Live load is apportioned based on the calculated transverse distribution of live load for a stress-laminated deck.

The interior deck strip deck has a depth of 235 mm and a unit weight of 6 kN/m³, resulting in an unfactored linear weight of

$$\omega_{deck,int} = 0.235 \ m \ \times 1.000 \ m \times 6 \frac{kN}{m^3} = \frac{1.41 kN}{m}$$

The exterior deck strip has the same geometry but also carries the weight of the MC230x37.8 steel channel bulkhead, resulting in an unfactored linear weight of

$$\omega_{deck,ext} = 1.41 \frac{kN}{m} + 0.372 \frac{kN}{m} = 1.78 \frac{kN}{m}$$

The minimum and maximum ULS load factors for the deck are 0.90 and 1.20, respectively.

The asphalt wearing surface has a unit weight of 23.5 kN/m. The interior and exterior deck strips have an average asphalt thickness of 174 mm and 57 mm, respectively, resulting in unfactored linear weights of

$$\omega_{ws,int} = 0.174 \ m \ \times 1.000 \ m \times 23.5 \frac{kN}{m^3} = 4.09 \ kN/m$$

$$\omega_{ws,ext} = 0.057 \ m \ \times 1.000 \ m \times 23.5 \frac{kN}{m^3} = 1.34 \ kN/m$$

The minimum and maximum ULS load factors for the asphalt wearing surface are 0.65 and 1.50, respectively.

The weight of a timber railing is assumed to be carried exclusively by the exterior deck strip. Each railing has an unfactored linear weight of

$$\omega_{railing} = 1.40 \frac{kN}{m}$$

The minimum and maximum ULS load factors for the timber railings are 0.90 and 1.20, respectively.

The resulting linear weights and bending moments due to permanent loads acting on the interior and exterior deck strips are as follows. The bending moments have been calculated for a simply-supported condition between floor beams and reduced by 20% to account for continuity. Alternatively, the deck strips can be modelled as continuous beams with the floor beams acting as rigid vertical supports. This analysis will yield very similar results to the 80% simply-supported moments.

PERMANENT LOAD – INTERIOR DECK STRIP

	Linear Weights				Bending Moments			
Load	SLS	ULS Min.	ULS Max.		SLS	ULS Min.	ULS Max.	
self-weight	1.41	1.27	1.69	kN/m	1.27	1.14	1.52	kNm/m
wearing surface	4.09	2.66	6.14	kN/m	3.68	2.39	5.53	kNm/m
railing	0.00	0.00	0.00	kN/m	0.00	0.00	0.00	kNm/m
Σ	5.50	3.93	7.83	kN/m	4.95	3.53	7.05	kNm/m

PERMANENT LOAD – EXTERIOR DECK STRIP

	Linear Weights				Bending Moments			
Load	SLS	ULS Min.	ULS Max.		SLS	ULS Min.	ULS Max.	
self-weight	1.78	1.60	2.14	kN/m	1.60	1.44	1.93	kNm/m
wearing surface	1.34	0.87	2.01	kN/m	1.21	0.78	1.81	kNm/m
railing	1.40	1.26	1.68	kN/m	1.26	1.13	1.51	kNm/m
Σ	4.52	3.73	5.83	kN/m	4.07	3.35	5.25	kNm/m

The vertical wind pressure is 930 Pa. The resulting unfactored uniformly distributed loads acting on the interior and exterior deck strips, respectively, are

$$\omega_{wind,int} = 930\ Pa \times \frac{kPa}{1000\ Pa} \times 1.000\ m = 0.93\ \frac{kN}{m}$$

$$\omega_{wind,ext} = 930\ Pa \times \frac{kPa}{1000\ Pa} \times 1.000\ m = 0.93\ \frac{kN}{m}$$

The vertical wind load is considered to act upward and downward. It is only applicable for the ULS 3 and ULS 4 load combinations. The ULS load factors for those combinations are 0.45 and 1.40, respectively.

The resulting linear weights and bending moments due to wind loads acting on the interior and exterior deck strips are as follows. As for permanent loads, the bending moments have been calculated for a simply-supported condition between floor beams and reduced by 20% to account for continuity.

WIND LOADS

	Linear Weights				Bending Moments			
Design Strip	SLS	ULS 3	ULS 4		SLS	ULS 3	ULS 4	
interior	0.00	0.42	1.30	kN/m	0.00	0.38	1.17	kN/m
exterior	0.00	0.42	1.30	kN/m	0.00	0.38	1.17	kN/m

The distribution of discrete live loads acting across the width of the deck must be considered. It is not appropriate to simply assume that the full width of the deck is effective in providing resistance to live loads. The simplified method of analysis for longitudinal load effects, per CHBDC clause 5.6, is an appropriate means of determining the percentage of live load carried by longitudinal deck strip of one metre width. Clause 5.6.7, which specifically addresses transverse live load distribution in longitudinal stress-laminated decks, is used for this purpose in this design example. The resulting live load effects are added to the effects caused by self-weight, wearing surface, barriers, and wind load to produce the total load effects.

The use of the CHBDC simplified method of analysis relies upon satisfying the following criteria from clause 5.6.2.

Clause	Criteria	Criteria Satisfied?
(a)	the width of the bridge is constant	Yes
(b)	the deck is continuous along the entire bridge width	Yes
(c)	The span between centreline of supports or bearing units is constant throughout the width of the bridge	Yes
(d)	the support conditions are closely equivalent to line support in all cases	Yes
(f)	diaphragms and bracing systems comply with the applicable requirements of Sections 8 to 10 and 17	Yes

The bridge width is constant, satisfying clause (a). The deck is continuous along the entire bridge width, satisfying clause (b). The spans are constant length and the deck is continuously supported by the floor beams, satisfying clauses (c) and (d). Finally, there are diaphragms at each support, per clause 9.20.2, satisfying clause (f). Note that the diaphragms that are offset 600 mm from the centreline of abutment bearings may be considered as abutment diaphragms in assessing whether there are diaphragms at the supports. Previous designs have had success with the diaphragms offset up to 900 mm from the centreline of abutment bearings (Wacker, J.P. & Smith, M.S. 2001). The criteria from clauses 5.6.2 (e), (g), (h), (i), (j), and (k) are not applicable to wood bridges, and have thus been excluded from the above table. Therefore, it is acceptable to use the simplified method for analysis of the stress-laminated deck.

The following calculations demonstrate the CHBDC simplified method of live load analysis for the stress-laminated deck. Analysis for shear is not provided, as the shear design of vertically laminated decks is not required, per CHDBC clause 9.7.5.

The deck has a travelled width of $w_c = 13.500\ m$. From Table 3.5 of the CHBDC, both two and three design lanes are to be considered for this travelled width. For two design lanes, the average lane width, w_e, is

$$w_e = \frac{w_c}{n} = \frac{13.500\ m}{2\ lanes} = 6.75\ m\ per\ design\ lane$$

Similarly, the average lane width for three design lanes is 4.500 m.

The longitudinal bending moment per metre deck width due to CL-625-ONT live loading, M_L, is calculated as

$$M_L = F_T F_S M_T$$

where F_T is the truck fraction carried per metre width, F_S is the skew factor, and M_T is the longitudinal bending moment generated by the passage of the CL-625-ONT live loading along a single design lane.

The skew factor, F_S, is taken as 1.0, per CHBDC clause 5.6.4.5 because the bridge is not skewed.

The truck fraction, F_T, is calculated as

$$F_T = \frac{B}{B_e D_T (1 + \mu\lambda)} \geq 1.05 \frac{nR_L}{B_e}\ for\ ULS\ \&\ SLS\ and\ \geq 1.05 \frac{1}{B_e}\ for\ FLS$$

The deck width, B, is equal to 14.110 m. The reduced deck width, B_e, is taken as equal to the deck width, per CHBDC clause 5.6.7.5.

The truck load distribution width, D_T, is taken from Table 5.12 of the CHBDC for stress-laminated decks. The value is a function of the number of design lanes, n, and the span length for the equivalent beam method, L_e. The latter value is equal to the span length between inflection points under the governing load case. It may be approximated using clause 5.6.4.6 and Figure 5.1. of the CHBDC. Using Figure 5.1(a) for a multi-span bridge, the span length for the equivalent beam method is taken as 50% of the actual span length, resulting in

$$L_e = 0.50 \times 3.000\ m = 1.500\ m$$

For two design lanes or more, the truck load distribution width, D_T, is calculated at ULS and SLS to be

$$D_T = 2.60 + 0.07 L_e \leq 4.0$$

$$D_T = 2.60 + 0.07 \times 1.500\ m = 2.705\ m \leq 4.0\ m$$

Similarly, the truck load distribution width, D_T, is calculated at FLS to be

$$D_T = 3.10 + 0.07L_e \leq 4.0$$

$$D_T = 3.10 + 0.07 \times 1.500 \, m = 3.205 \, m \leq 4.0 \, m$$

Analysis at FLS is normally not required for the design of wood components, but the truck fraction for bending moment at FLS may be used as an approximate means for determining the live load deflection of a component at SLS using the simplified method, per CHBDC clause 5.6.4.7.

The lane width modification factor, μ, is equal to

$$\mu = \frac{w_e - 3.3}{0.6} \leq 1.0$$

The lane width modification factor is calculated to be 5.75 and 2 for two and three design lanes, respectively. Since both those values are more than 1.0, the lane width modification factor is taken as equal to 1.0.

The width correction factor, λ, is taken as zero, per CHBDC clause 5.6.7.5.

Based on the preceding values, the truck fraction, F_T, is calculated at ULS and SLS to be

$$F_{T,ULS/SLS} = \frac{B}{B_e D_T (1 + \mu\lambda)} = \frac{14.110}{14.110 \, m \times 2.705 \times (1 + 1.0 \times 0)} = \frac{0.370}{m}$$

This value is greater than

$$1.05 \frac{nR_L}{B_e} = 1.05 \times \frac{2 \times 0.9}{14.110 \, m} = \frac{0.134}{m}$$

Therefore, the truck fraction at ULS and SLS is $0.370/m$. Similarly, the truck fraction at FLS is equal to $0.312/m$.

The longitudinal bending moment generated by the passage of the CL-625-ONT live loading along a single design lane, M_T, is determined using the moving load analysis function of a structural analysis program. The deck is modeled as a three-span continuous beam of one metre width, with the floor beams acting as rigid vertical supports. Recall that truck axles and portions of the uniformly distributed lane load that reduce the load effect are neglected. For short spans, like that of the stress-laminated deck in this design example, it is usually only necessary to examine effects caused by axle 4, axles 2 and 3 in combination, and axles 1-3 in combination. For this design example, it is found that the passage of axle 4 by itself is the most critical live load case, resulting in a bending moment of 105 kNm.

The dynamic load allowance is applied to the critical moment to calculate M_T. The dynamic load allowance for a single truck axle is 0.40, per CHBDC clause 3.8.4.5.3. This value is reduced by 30% to account for the dynamic qualities of wood bridges, per CHBDC clause 3.8.4.5.4. The resulting M_T value is calculated to be

$$M_T = 105 \ kNm \times (1 + 0.40 \times 0.70) = 105 \ kNm \times 1.28 = 135 \ kNm$$

The resulting longitudinal bending moment per metre deck width due to CL-625-ONT live loading, M_L, is calculated as

$$M_L = \frac{0.370}{m} \times 1.0 \times 135 \ kNm = 50 \ kNm/m$$

As previously stated, the live load deflection of a component at SLS may be calculated using the simplified method, per CHBDC clause 5.6.4.7. The live load deflection generated by the passage of the CL-625-ONT live loading along a single design lane, Δ_T, is determined in the same manner as is M_T. The maximum live deflection is determined to be 11 mm. It is caused by the passage of axle 4. It is not necessary to amplify this value by the dynamic load allowance, per CHBDC clause 9.4.2. Therefore, the longitudinal deflection per metre deck width due to CL-625-ONT live loading, Δ_L, is calculated as

$$\Delta_L = \frac{0.312}{m} \times 1.0 \times 11 \ mm = 3.4 \ mm/m$$

The total factored bending moments experienced by the stress-laminated deck are as follows:

TOTAL FACTORED BENDING MOMENTS

Design Strip	SLS	ULS 1	ULS 2	ULS 3	ULS 4	ULS 4 (Uplift)	
interior	50	93	88	78	9	2	kNm/m
exterior	50	93	88	78	9	2	kNm/m

The factored SLS live load deflections experienced by the stress-laminated deck are as follows:

LIVE LOAD DEFLECTIONS

Design Strip	SLS	
interior	3.0	mm
exterior	3.0	mm

3.3.4.2. GLUED-LAMINATED TIMBER GIRDERS & FLOOR BEAMS

The CHBDC simplified method does not explicitly cover the type of bridge presented in this design example. The simplified method presents live load distribution factors for bridges incorporating wood decks. While the bridge in this design example has a wood deck, it is oriented parallel to the girders and contributes very little flexural stiffness in the transverse direction of the bridge. Instead, it is the floor beams that provide most of the transverse stiffness. The bridges incorporating wood decks that are covered by the simplified method have decks that are oriented perpendicular to the girders, and therefore provide significant flexural stiffness in the transverse direction of the bridge. In recognition of this structural behaviour, it is recommended that the simplified method of analysis not be used for the type of bridge presented in this design example. Accordingly, the girders and floor beams are analyzed using computer structural analysis.

Computer structural analysis is in the form of a three-dimensional grillage model. Frame elements are used to represent the floor beams and girders, with one line of frame elements used for each floor beam and each girder. Shell elements are used to model the stress-laminated deck and girder diaphragms. Link elements are used to model the connections and bearings. The stiffness of the shells representing the deck is reduced to account for the presence of butt joints, per CHBDC clause 9.5.8.

The factored bending moments, factored shears, SLS live load deflections, and unfactored permanent load deflections determined by computer structural analysis are shown below for the interior and exterior girders. The SLS live load deflections exclude the dynamic load allowance.

Load Effect	Exterior Girder	Interior Girder	Floor Beam
Mf	1400 kNm	778 kNm	100 kNm
Vf	285 kN	165 kN	159 kN
ΔLL	8.5 mm	12.2 mm	1.1 mm
ΔPERMANENT	14 mm	12 mm	N/A

As noted by clause 9.7.3 of the CHBDC, the factored shear resistance is to exceed the factored shear load. The factored shear load represents the maximum horizontal shear force experienced by a glued-laminated member. It is not the same as the factored shear force experienced at a given cross-section. Glued-laminated timber is known to be weaker in horizontal shear than transverse vertical shear, so the shear design of glued-laminated timber members is predicated on designing for horizontal shear. Clause 9.7.3 presents an equation to determine the factored shear load. It determines the maximum horizontal shear force experienced by the member as a function of the member volume and the vertical shear force distribution along the member. The equation is cumbersome to use in the presence of live load, as the critical live load position for maximum horizontal shear is generally not the same as the critical live load position for maximum vertical shear. Consequently, a trial-and-error approach is often taken, wherein a designer moves the live load along the bridge in discrete increments and calculates the shear load for each increment. Furthermore, the calculation of shear load does not allow for superposition of load cases. Therefore, the designer must calculate the shear load with all applicable loads applied simultaneously. The ULS 1 load combination governs in this design example, and will likely always govern for wood bridge shear design.

The calculation of the shear load is best performed on an isolated girder. It is often beneficial to apply permanent loads at discrete locations, say the span tenth-points, as the shear load integral reduces to a simple summation for step-wise vertical shear force diagrams. A fraction of the weight of the CL-625-ONT loading is applied to the girder. That fraction is either the truck fraction determined by use of the CHBDC simplified method of live load analysis or the truck fraction implied from the global computer structural analysis model. This latter term is calculated as the ratio of the maximum shear force experienced by a girder in the computer structural analysis model to the maximum shear force experienced by single isolated girder under the passage of one lane of CL-625-ONT loading. The shear loads in this design example are based on the implied truck fraction determined from the computer structural analysis model. The resulting live load is factored and moved along the girder until the factored shear load is maximized.

There is no intuitive positioning of the live load that maximizes the shear load. The live load was moved along the girder at one metre intervals to determine the shear load for this design example, resulting in a total of 36 distinct shear load calculations. Multi-step live load analysis and filtered spreadsheet results are useful tools for this procedure. The shear load calculations are not shown in this design example due to their size. Refer to the CHBDC commentary clause C9.7 for an example calculation of the shear load.

The calculated shear load for the exterior girders, interior girders, and floor beams are summarized in the following table.

Load Effect	Exterior Girder	Interior Girder	Floor Beams
factored vertical shear	285 kN	165 kN	100 kN
factored shear load	184 kN	106 kN	80 kN
ratio	0.65	0.65	0.50

The use of the factored shear load for shear design is obviously quite advantageous relative the use of the factored vertical shear. The factored demand is reduced by 35%. Despite its cumbersome calculation process, there is significant benefit to be gained by calculating the factored shear load versus using the much easier to obtain factored vertical shear.

The factored shear load is a design concept distinct to the CHBDC since the 2000 edition. Previous wood design practice was to design for the maximum vertical shear force at the lesser of the span quarter-point and three beam depths from the support. Since many wood bridge girders have a span-to-depth ratio of approximately 10 to 12, these two locations often roughly coincide. The AASHTO LFRD Bridge Design Specifications still make use of this design approach. A comparison of this method versus the shear load is made in the following table for the interior and exterior girders.

Location	Exterior Girder	Interior Girder
at support $(x = 0\ mm)$	285 kN	165 kN
span quarterpoint $(x = 4500\ mm)$	192 kN	106 kN
three beam depths from support $(x = 4902\ mm)$	192 kN	106 kN
shear load	184 kN	106 kN

As it turns out, the maximum shear force at each of the span quarter-point and three beam depths from the supports is nearly the same as or identical to the shear load. While this design approach is not the one prescribed by the CHBDC, it very clearly provides an accurate approximation of the shear load. A designer may wish to use this approach for preliminary design, and then perform the rigorous shear load calculation for detailed design.

3.3.5. MEMBER DESIGN

3.3.5.1. STRESS-LAMINATED DECK

The deck is designed for flexure as a longitudinal stress-laminated deck in accordance with clause 9.6 of the CHBDC. Shear is not considered in the design of the deck in accordance with clause 9.7.5 of the CHBDC.

Live load deflection of the deck is considered in accordance with clause 9.4.2 of the CHBDC. Per that clause, the live load deflection caused by the design live loading, excluding dynamic load allowance, is not to exceed 1/400th the span. The 50th percentile modulus of elasticity is used for this calculation.

3.3.5.1.1. FLEXURAL DESIGN

The flexural resistance of the deck, $M_{r,deck}$, is calculated as for a sawn wood member, per CHBDC clause 9.6.1. The resistance is equal to

$$M_{r,deck} = \phi k_d k_{ls} k_m k_{sb} f_{bu} S$$

The material resistance factor, $\emptyset$, is determined from CHBDC Table 9.1 to be equal to 0.9 for sawn wood in flexure.

The load-duration factor, k_d, is determined using CHBDC clause 9.5.3. This factor considers the influence of cumulative load towards the creep rupture of a member (FPL 2010). The critical design bending moments and shears are the result of the ULS 1 load combination. Accordingly, the load-duration factor is equal to 1.0. The load-duration factor is equal to 1.15 when calculating the resistance against loads resulting from the ULS 3 and ULS 4 load combinations, as these combinations include wind load.

The lateral stability factor, k_{ls}, is determined using CHBDC clause 9.6.3. This factor accounts for the possibility of lateral-torsional buckling instability occurring before cross-sectional rupture occurs. It is equal to 1.0 for laminated wood decks.

The load-sharing factor, k_m, is determined using CHBDC clause 9.5.6. This factor accounts for the reduced probability of material defects within adjacent members of given cross-section. It also accounts for the statical indeterminacy afforded by neighbouring members (CSA 2014b). Wood is traditionally designed using the assumption of linear-elastic material behaviour; however, Sexsmith *et al.* (1979) demonstrated that the slight flexural softening experienced by wood members before failure is sufficient to shed load to adjacent stiffer members in systems of closely spaced members. The effective width over which this phenomenon occurs, D_e, is equal to 1.75 m for a longitudinal stress-laminated deck, per CHBDC Table 9.3. The deck laminations have a width of 0.038 m, resulting in $n = 1.75\ m \div 0.038\ m = 46$ laminations being effective in sharing load. Using the value of $n = 46$ laminations, the load-sharing factor is calculated from CHBDC Table 9.2 to be equal to 1.40. Refer to Bakht and Jaeger (1991) for further information concerning the load-sharing in timber design.

The size-effect factor, k_{sb}, is determined from CHBDC Table 9.4 to be equal to 1.10 for 38 mm x 235 mm deck laminations.

The section modulus, S, must account for the presence of butt joints, per CHBDC clause 9.5.8. Deck laminations are of finite length, and are typically not longer than six metres. The joint where a lamination ends and another lamination begins is referred to as a butt joint. The butt joint is a discontinuity in the stiffness of the section and must be accounted for in the design. The CHBDC accounts for butt joints using the butt joint stiffness factor, k_b, which is based on the work of Jaeger and Bakht (1990). Butt joints are specified at a frequency of not less than one in four laminations. Accordingly, the butt joint stiffness factor is equal to

$$k_b = \frac{N_b - 1}{N_b} = \frac{4 - 1}{4} = 0.75$$

The resulting section modulus for a one metre wide strip of deck is equal to

$$S = k_b \frac{bd^2}{6} = 0.75 \times \frac{100 \, mm \times (235 \, mm)^2}{6} = 6903 \times 10^3 \, mm^3$$

The resulting factored flexural resistance is equal to

$$M_r = \phi k_d k_{ls} k_m k_{sb} f_{bu} S$$
$$M_{r,deck} = 0.9 \times 1.0 \times 1.0 \times 1.40 \times 1.1 \times 11.8 \, MPa \times 6903 \times 10^3 \, mm^3 = 113 \frac{kNm}{m}$$

The factored flexural demand was calculated to be equal to

$$M_{f,deck} = 93 \frac{kNm}{m} < M_{r,deck} = 113 \frac{kNm}{m}$$

Therefore, the deck has sufficient flexural capacity.

3.3.5.1.2. SLS DESIGN

The SLS deflection due to live load was determined to be 3.0 mm, which is less than the permissible SLS deflection of $3000 \, mm \div 400 = 7.5 \, mm$. Therefore, SLS deflections are within the permissible limit.

3.3.5.1.3. POST-TENSIONING DESIGN

The post-tensioning system in this design example is the internal systems shown in Figure 9.6. of the CHBDC. It is designed in accordance with clause 9.23 of the CHBDC. It consists of 18 mm diameter post-tensioning bars spaced at 840 mm on centre. The post-tensioning anchorages consist of steel bearing plates that bear against a MC230x37.8 steel channel. This channel is used as a distribution bulkhead for the post-tensioning force. The post-tensioning bars pass through 25 mm diameter drilled holes that extend the entire width of the deck.

The gross cross-sectional area of a post-tensioning bar, A_{ps} and the ultimate tensile strength of a post-tensioning bar, f_{pu}, are

$$A_{ps} = 241 \; mm^2$$

$$f_{pu} = 1050 \; MPa$$

The post-tensioning bars will be galvanized to enhance their durability. It is assumed that the galvanizing process will result in a loss of 5% of the ultimate tensile strength of the bars. Any loss of strength due to coating or galvanizing of the post-tensioning bars is to be reflected in the design, per clause 9.23.2.4 of the CHBDC. The effective ultimate tensile strength is therefore equal to

$$f_{pu,eff} = (1 - 0.05) \times f_{pu} = 0.95 \times 1050 = 998 \; MPa$$

The maximum stress in the post-tensioning bars is not to exceed $0.85 f_{pu,eff}$ and $0.80 f_{pu,eff}$ at jacking and at transfer, respectively, per clause 9.23.2.4 of the CHBDC. The post-tensioning bars are to be stressed to $0.70 f_{pu,eff}$ for this design example. It is assumed that that there will be a 5% loss of post-tensioning force at transfer, resulting in a bar stress of $0.70 f_{pu,eff}$ at transfer. Therefore, the post-tensioning bar stress limits are satisfied at jacking and at transfer.

The spacing of the post-tensioning anchorages must comply with clause 9.23.4.4.4 of the CHBDC. The spacing is not to be less than 2.5 times the depth of the wood deck and 15 times the diameter of the hole for the post-tensioning bars, D_h . It is not to exceed six times the depth of the wood deck and 1.50 m. The minimum and maximum spacing s_{min}, and s_{max} , respectively, are

$$s_{min} = MAX\{2.5d, 15D_h\} = MAX\{2.5 \times 235 \; mm, 15 \times 25 \; mm\} = MAX\{587mm, 375 \; mm\}$$
$$= 587 \; mm$$

$$s_{max} = MIN\{6d, 1.50 \; m\} = MIN\{6 \times 235 \; mm, 1.50 \; m\} = MIN\{1410 \; mm, 1500 \; mm\} = 1410 \; mm$$

Therefore, the post-tensioning anchorage spacing of $s = 840 \; mm$ satisfies the minimum and maximum spacing requirements.

The behaviour of a stress-laminated deck acting as an orthotropic plate is predicated on the provision of sufficient friction between adjacent laminations to preclude relative slip between those laminations. This friction is established by the post-tensioning force acting perpendicular to the laminations. If this force is too low, then slip will occur between laminations, resulting in the laminations acting independently of one another, ultimately reducing the inherent and beneficial live load distribution characteristic of a stress-laminated deck. If this force is too large, then the laminations will crush. Clause 9.23.3.3 of the CHBDC specifies minimum and maximum normal stress limits to ensure adequate performance of a stress-laminate deck.

The maximum normal pressure between laminations occurs at jacking. The maximum normal pressure, N_j, is limited to 25% of the limiting pressure perpendicular to grain, $f_{q\ell}$, specified in Table 9.18 of the CHBDC. The limiting pressure perpendicular to grain for Lodgepole pine is $f_{q\ell} = 4.4\ MPa$. The normal pressure between laminations is defined as the post-tensioning anchorage force acting over an area equal to the product of the post-tensioning anchorage spacing, s, and the width of the lamination, b. The CHBDC defines the width of the lamination to be direction that many engineers would define as the depth of the lamination. Refer to CHBDC Figure 9.6. for clarity. Based on these parameters, the normal pressure acting on the laminations at jacking, N_j, is equal to

$$N_j = \frac{0.75 f_{pu,eff} \times A_{ps}}{s \times b} = \frac{0.75 \times 998\ MPa \times 241\ mm^2}{840\ mm \times 235\ mm} = 0.914\ MPa$$

$$N_j = 0.914\ MPa < N_{j,max} = 0.25 f_{q\ell} = 0.25 \times 4.400\ MPa = 1.100\ MPa$$

The minimum normal pressure between laminations occurs after all losses. The CHBDC assumes that there will be a combined 60% loss of prestressing force at and after transfer, resulting in a long-term effective pressure between laminations equal to $0.4N_i$. This long-term pressure must be equal to at least 0.35 MPa, per clause 9.23.3.3 of the CHBDC. The long-term effective pressure for this design example is equal to

$$0.4N_j = 0.4 \times 0.914\ MPa = 0.366\ MPa > 0.35\ MPa$$

Therefore, the specified post-tensioning system provides sufficient normal pressure between laminations to maintain orthotropic plate behaviour without crushing the laminations. Note, however, that the assumption of 60% loss of prestressing force is approximate. The designer and owner should always make provision for periodically checking the prestressing forces in the bars and re-stressing the bars if necessary (CSA 2014b).

The construction of a stress-laminated deck involves three important stressing stages: the initial stressing, the first restressing, and the second restressing. These stages are explained in clause 9.23.3.4 of the CHBDC. The initial stressing consists of two stressing operations conducted not less than 12 hours apart. The first restressing is to occur not less than two weeks after the initial stressing. The second restressing is to occur not less than four weeks after the first restressing. Restressing in advance of these times will result in greater loss of prestressing force. Increasing the time interval between stressing operations will reduce the loss of prestressing force, but limited returns are observed when waiting more than six weeks and six months, respectively, between the initial stressing and the first restressing, and the first and second restressings (CSA 2014b).

An important aspect of stress-laminated deck design is to ensure that that the post-tensioning bars are flexible enough to not lose most of their prestressing force due to creep of the deck laminations (CSA 2014b). This aspect is considered through limiting the steel/wood ratio, A_r. Clause 9.23.3.2 of the CHBDC limits this ratio to a maximum of 0.0016. The ratio is defined as the ratio of the cross-sectional area of post-tensioning steel at a single anchorage, A_s, to the product of the post-tensioning anchorage spacing and the depth of the deck, A_w. The steel/wood ratio for this design example is equal to

$$A_r = \frac{A_s}{A_w} = \frac{A_{ps}}{s \times d} = \frac{241 \; mm^2}{840 \; mm \times 235 \; mm} = 0.0012 < A_{r,max} = 0.0016$$

Therefore, the steel/wood ratio is within acceptable limits.

The post-tensioning system makes use of a distribution bulkhead to disperse the high magnitude concentrated forces from the post-tensioning anchorages to the deck laminations. The design of the distribution bulkhead is in accordance with clause 9.23.3.4 of the CHBDC.

A steel channel bulkhead extending the entire length of each deck fascia, along with steel anchorage plates, is considered a suitable distribution bulkhead, per clause 9.23.4.4.1 of the CHBDC. The depth of the steel channel bulkhead, d_c , is to be at least 85% of the width of a lamination but not exceed the width of the lamination. The minimum weak-axis moment of inertia, $I_{y,min}$, and web thickness $w_{c,min}$, of the steel channel is specified in Table 9.19 of the CHBDC as a function of the deck depth. The steel channel in this design example is a MC230x37.8 section. The properties of the channel are as follows.

$$d_{c,min} = 0.85b = 0.85 \times 235 \ mm = 200 \ mm$$

$$d_{c,max} = b = 235 \ mm$$

$$d_{c,min} = 200 \ mm < d_c = 229 \ mm < d_{c,max} = 235 \ mm$$

$$I_{y,min} = 1 \times 10^6 \ mm^4 < I_y = 3.16 \times 10^6 \ mm^4$$

$$w_{c,min} = 11.0 \ mm < w_c = 11.4 \ mm$$

Therefore, the steel channel distribution bulkhead has sufficient stiffness to distribute the post-tensioning forces.

Each post-tensioning bar transfers its force to an anchorage but that bears against a steel anchorage plate. The anchorage plate bears against the steel channel bulkhead and aids in distributing the post-tensioning force to the laminations. The proportioning of the anchorage plate is covered by clause 9.23.4.4.3 of the CHBDC. The length of the anchorage plate, L_p, is measured parallel to the span of the deck. The width of the anchorage plate, w , is measured parallel to the depth of the deck. The length-to-width ratio of the anchorage plate is to exceed 1.0, but is not to exceed 2.0. The length and width of the anchorage plates specified in this design example are

$$L_p = 220 \ mm$$

$$w = 110 \ mm$$

$$1.0 < \frac{L_p}{w} = \frac{220 \ mm}{110 \ mm} = 2.0$$

The specified anchorage plate dimensions are within the permissible limits specified by the CHBDC. The anchorage plate width should also be limited to the depth of the flat portion of web of the steel channel, T. The value of T is equal to 167 mm for a MC230x37.8 channel, so the anchorage plate will bear uniformly on the steel channel.

The CHBDC mandates the minimum thickness of the anchorage plate, $t_{pl,min}$, not be less than one-twelfth the length of the anchorage plate. The thickness of the anchorage plate, t_{pl}, is calculated assuming a uniform pressure distribution under the plate. It can be easily calculated conservatively by assuming one-way plate bending about an axis passing vertically through the centre of the plate. Assuming a load factor equal that is used for secondary prestress effects, α_p, the bearing stress under the anchorage plate at jacking is equal to

$$\sigma_{f,pl} = \frac{\alpha_p \times 0.75 f_{pu,eff} \times A_{ps}}{L_p \times w} = \frac{1.05 \times 0.75 \times 998 \, MPa \times 241 mm^2}{220 \, mm \times 110 \, mm} = 7.83 \, MPa$$

The resulting factored bending moment in the plate, M_f, is equal to

$$M_f = \sigma_{f,pl} \times \left(\frac{L_p}{2} \times w\right) \times \frac{L_p}{4}$$

$$M_f = 7.83 \, MPa \times \left(\frac{220 \, mm}{2} \times 110 \, mm\right) \times \frac{220 \, mm}{4} \times \frac{10^{-6} \, kNm}{Nmm} = 5.21 \times 10^{-6} \, Nmm$$

Assuming a plate yield stress of $F_{y,pl} = 300 \, MPa$ the plate thickness for this design example is calculated as

$$t_{pl} = \sqrt{\frac{4M_f}{\emptyset b F_y}} = \sqrt{\frac{4 \times 5.21 \times 10^{-6} \, Nmm}{0.95 \times 110 \, mm \times 300 \, MPa}} = 25.8 \, mm$$

$$\therefore use \ t_{pl} = 28 \, mm > t_{pl,min} = \frac{L_p}{12} = \frac{220 \, mm}{12} = 18.3 \, mm$$

The edge laminations must be checked against a localized bearing failure in the post-tensioning anchorage zone. The factored bearing resistance, R_r, is a function of the limiting pressure perpendicular to grain, $f_{q\ell}$, specified in Table 9.18 of the CHBDC. It is also a function of the effective bearing area, A_b. It is calculated in accordance with clause 9.23.4.2 of the CHBDC as

$$R_r = \emptyset f_{q\ell} A_b$$

The material resistance factor, $\emptyset$, is determined from CHBDC Table 9.1 to be equal to 0.8 for sawn wood in compression perpendicular to grain. The limiting pressure perpendicular to grain, $f_{q\ell}$, is equal to 4.4 MPa, as previously stated for Lodgepole pine. The effective bearing area, A_b, is equal to

$$A_b = b_b \times L_b$$

The width of the effective bearing area, b_b is equal to the depth of the steel channel bulkhead. The width of the effective bearing area, L_b, is equal to the length of the anchorage plate plus twice the web thickness of the steel channel bulkhead. The resulting factored bearing resistance is equal to

$$b_b = 229 \, mm$$

$$L_b = L_p + 2w_c = 220 \, mm + 2 \times 11.4 \, mm = 243 \, mm$$

$$A_b = b_b \times L_b = 229 \, mm \times 243 \, mm = 55647 \, mm^2$$

$$R_r = \emptyset f_{q\ell} A_b = 0.8 \times 4.4 \, MPa \times 55647 \, mm^2 \times \frac{kN}{1000 \, N} = 196 \, kN$$

The factored bearing force acting within the anchorage zone is equal to the factored force in the post-tensioning bars at jacking multiplied by the load factor equal that used for secondary prestress effects, α_p. It is equal to

$$R_f = \alpha_p \times 0.75 f_{pu,eff} \times A_{ps} = 1.05 \times 0.75 \times 998 \, MPa \times 241 mm^2 = 190 \, kN$$

$$R_f = 190 \, kN < R_r = 196 \, kN$$

Therefore, the post-tensioning system has sufficient proportions to preclude crushing of the edge laminations in the anchorage zones.

The design of a stress-laminated deck shall also meet the detailing requirements specified in clause 9.23.5 of the CHBDC. The requirements concern lamination dimensions, hole diameters for the post-tensioning bars, nailing of the laminations before stressing, and connection of the deck to its supporting elements. Of importance, a stress-laminated deck shall not be attached to its supporting elements until after the first restressing operation. Connection in advance of this restressing operation could result in a lesser normal pressure between laminations than anticipated. The only exception to this rule is when the deck dimension measured perpendicular to the length of the laminations exceeds 40 times the depth of the laminations, in which case restraint against buckling is to be provided, per clause 9.23.5.7 of the CHBDC. While this clause is explicitly for transverse stress-laminated decks, it would also apply to longitudinal stress-laminated decks, as it is the unbraced length in compression that influences the susceptibility to buckling, not the geometric orientation of the deck. The deck in this design example meets this criterion, as its unbraced length in compression during stressing is 14 110 mm, which exceeds $40 \times 235 \, mm = 9 \, 400 \, mm$. Therefore, restraint against buckling would need to be applied to the deck along the centerline of the bridge. This restraint is not detailed in this design example, but the force that it would have to restrain would be analogous to that resisted by steel column bracing. A simple solution would be to ballast the deck along the centerline of the structure. The ballast would need to be minimized though, as it would contribute to friction between the deck and its supporting elements, which would interfere with the prestressing operation.

3.3.5.2. GLUED-LAMINATED TIMBER FLOOR BEAMS

The floor beams are designed for flexure and shear at the ultimate limit state, in accordance with CHBDC clauses 9.6 and 9.7. They are designed for deflection at the serviceability limit state in accordance with clauses 9.4.2.

The floor beams span a relatively short distance, but are subject to large shears from the deck. Short span beams like these are often governed by shear resistance. As previously mentioned, the floor beams in this design example are governed by stiffness, as their flexural stiffness affords the bridge most of its ability to share live loads between girders. The floor beams were initially sized for strength, but the live load distribution between girders was poor. It was decided to increase the stiffness of the floor beams to save material in the girders. The designer should consider such impacts in real-life design. Even with the increase in floor beam size for stiffness, the floor beams are still 90% utilized in shear, which is not unreasonable from a cost standpoint.

3.3.5.2.1. FLEXURAL DESIGN

The flexural resistance of a floor beam, $M_{r,floor}$, is calculated as for a glued-laminated timber member, per CHBDC clause 9.6.1. The resistance is equal to the lesser of

$$M_{r,floor} = \phi k_d k_{ls} k_m f_{bu} S \text{ and } M_{r,floor} = \phi k_d k_m k_{sb} f_{bu} S$$

The variables in these two equations have the same definitions as those used to calculate the flexural resistance of the stress-laminated deck; however, some of them are calculated differently because of differences in material (i.e. glued-laminated timber versus sawn wood) and geometric properties.

The material resistance factor, ϕ, is determined from CHBDC Table 9.1 to be equal to 0.9 for glued-laminated timber in flexure.

The load-duration factor is taken as $k_d = 1.0$.

The lateral stability factor, k_{ls}, requires explicit calculation because the depth-to-width ratio of the floor beams exceeds 1.0. It is calculated as a function of the slenderness factor, C_s, which is in turn a function of the laterally unsupported length, L_u. The latter is equal to the girder spacing, as the floor beams are restrained from lateral movement and twisting at the girders by their connections to the girders. The slenderness factor is therefore equal to

$$C_s = \sqrt{\frac{L_u d}{b^2}} = \sqrt{\frac{1150 \ mm \times 380 \ mm}{(365 \ mm)^2}} = 1.81 < 10$$

The lateral stability factor is determined from CHBDC Table 9.5 to be

$$k_{ls} = 1.0 \text{ for } d/b > 1.0 \text{ and } C_s = 1.81$$

The load-sharing factor, k_m, is equal to 1.0, per CHBDC clause 9.5.6, because glued-laminated timber girders are not covered in CHBDC Table 9.3. Some designers have been known to treat glued-laminated timber girders as sawn wood stringers for calculating the load-sharing factor. It is possible that this approach is somewhat unconservative. Recall that the load-sharing factor accounts for the reduced probability of material defects within adjacent members of given cross-section. Since glued-laminated timber inherently contains less defects than sawn wood, it is logical that the load-sharing benefit in glued-laminated timber members is less than that in sawn wood members. Research is underway currently to investigate this possibility.

The size-effect factor for glued-laminated timber, k_{sb}, is determined from CHBDC clause 9.6.2 to be equal to

$$k_{sb} = \left(\frac{130}{b}\right)^{\frac{1}{10}} \left(\frac{610}{d}\right)^{\frac{1}{10}} \left(\frac{9100}{L}\right)^{\frac{1}{10}} \leq 1.3$$

$$k_{sb} = \left(\frac{130}{365}\right)^{\frac{1}{10}} \left(\frac{610}{380}\right)^{\frac{1}{10}} \left(\frac{9100}{14110}\right)^{\frac{1}{10}} = 0.91 \leq 1.3$$

$$\therefore k_{sb} = 0.91$$

The section modulus of the floor beams does not have to be modified by the butt joint stiffness factor because the individual laminations in a glued-laminated timber member are finger-jointed and glued together to form a continuous lamination. These joints undergo strength testing to ensure their performance as continuous joints, per CSA O122, during the manufacture of glued-laminated timber. The section modulus is therefore equal to

$$S = \frac{bd^2}{6} = \frac{365\ mm \times (380\ mm)^2}{6} = 8784 \times 10^3\ mm^3$$

The resulting factored flexural resistance is equal to the lesser of

$$M_{r,floor} = \phi k_d k_{ls} k_m f_{bu} S$$
$$M_{r,floor} = 0.9 \times 1.0 \times 1.0 \times 1.0 \times 23.0\ MPa \times 8784 \times 10^3\ mm^3 = 182\ kNm$$

and

$$M_{r,floor} = \phi k_d k_m k_{sb} f_{bu} S$$
$$M_{r,floor} = 0.9 \times 1.0 \times 1.0 \times 0.91 \times 23.0\ MPa \times 8784 \times 10^3\ mm^3 = 165\ kNm$$

$$\therefore M_{r,floor} = 165\ kNm$$

The factored flexural demand was calculated to be equal to

$$M_{f,floor} = 100\ kNm < M_{r,floor} = 165\ kNm$$

Therefore, the floor beams have sufficient flexural capacity.

3.3.5.2.2. SHEAR DESIGN

The shear resistance of a floor beam, $V_{r,floor}$, is calculated per CHBDC clause 9.6.1 as

$$V_{r,floor} = \phi k_d k_m k_{sv} f_{vu} A / 1.5$$

The material resistance factor, ϕ , is determined from CHBDC Table 9.1 to be equal to 0.9 for glued-laminated timber in shear.

The load-duration factor and load-sharing factor are the same as for flexure, with $k_d = 1.0$ and $k_m = 1.0$.

The size-effect factor for glued-laminated timber in shear is determined using CHBDC clause 9.7.2 to be equal to

$$k_{sv} = V^{-0.18} = (0.365 \, m \times 0.380 \, m \times 14.110 \, m)^{-0.18} = 0.89$$

Like the section modulus, the cross-sectional area of a glued-laminated timber member does not need to be reduced in stiffness because there are no butt joints. The cross-sectional area is thus equal to

$$A = bd = 365 \, mm \times 380 \, mm = 138700 \, mm^2$$

The resulting factored shear resistance is equal to

$$V_{r,floor} = \frac{\phi k_d k_m k_{sv} f_{vu} A}{1.5} = \frac{0.9 \times 1.0 \times 1.0 \times 0.89 \times 1.18 \, MPa \times 138700 \, mm^2}{1.5} = 87 \, kN$$

The factored shear load was calculated to be equal to

$$V_{f,floor} = 80 \, kNm < V_{r,floor} = 87 \, kNm$$

Therefore, the floor beams have sufficient shear capacity.

3.3.5.2.3. SLS DESIGN

The SLS deflection due to live load was determined to be 1.1 mm, which is less than the permissible SLS deflection of $1150 \, mm \div 400 = 2.9 \, mm$. Therefore, SLS deflections are within the permissible limit.

3.3.5.3. GLUED-LAMINATED TIMBER GIRDERS

The bridge girders are designed for flexure and shear at the ultimate limit state, in accordance with CHBDC clauses 9.6 and 9.7, respectively. They are designed for deflection and vibrations at the serviceability limit state in accordance with clauses 9.4.2 and 3.4.4, respectively.

3.3.5.3.1 SLS DESIGN

The design of bridge girders are often governed by strength in flexure or shear. For wood bridges, however, the likelihood of deflection or vibration governing is much greater than for girders made of other traditional engineering materials. In fact, the girders in this example are governed by vibrations at SLS.

Clause 3.4.4 of the CHDBC mandates that the deflection due to live load, including dynamic load allowance, is to be less than the deflections limits prescribed by the curves in CHBDC Figure 3.1. The deflection limits are a function of the first flexural frequency of the bridge and consider whether the bridge has sidewalks. This criterion is an implicit means of limiting accelerations for user comfort (CSA 2014b). The SLS 2 load combination is used to calculate the deflection. The live load is to be positioned as specified in clause 3.8.4.1, and the deflection is to be measured at the inside face of the bridge barriers for a bridge without sidewalks. The first flexural frequency of the bridge in this design example was determined to be 5.1 Hz using the computer structural analysis model. From CHBDC Figure 3.1., the maximum permissible deflection corresponding to this first flexural frequency for a bridge without sidewalks is 15 mm. The live load deflection at the face of the railings was determined to be 14.8 mm using the computer structural analysis model, which is marginally less than the deflection limit. Therefore, superstructure vibrations have been addressed.

The Ontario Ministry of Transportation (MTO) Structural Manual includes guidelines for the design of bridges on low volume roads. A low volume road is defined as a road having an average annual daily traffic of 400 or less in both directions. These guidelines relax the superstructure vibration criterion defined in Clause 3.4.4 of the CHBDC to simply be a deflection limit of 1/360th the span under the SLS 2 load combination, irrespective of first flexural frequency. As is noted in the next paragraph, the maximum deflection due to SLS 1 live load is 12.2 mm, which is far less than the low volume road limit of $18000 \; mm \div 360 \; = 50 \; mm$. The bridge that is the subject of this design example is designed for use on an undivided arterial road, which would have an AADT well more than 400. Had it been designed for use on a low volume road, then the girder sizes could be reduced until either the deflection limit or strength governed.

Clause 9.4.2 of the CHBDC requires that the deflection due to live load, excluding dynamic load allowance, be less than 1/400th the span. The SLS 1 load combination is to be used for this purpose. The live load is to be positioned as specified in clause 3.8.4.1. The 50th percentile modulus of elasticity is to be used to determine the deflection. The corresponding maximum live load deflections for the exterior and interior girders of this design example were determined to be 12.2 mm and 8.5 mm, respectively. Both deflections are less than the deflection limit of $18\,000 \; mm \div 400 \; = 45 \; mm$. Therefore, superstructure deflections under live load have been addressed.

3.3.5.3.2. FLEXURAL DESIGN

The flexural resistance of the exterior girders, $M_{r,girder,ext}$, is calculated in the same manner as for the floor beams. The glued-laminated timber diaphragms are designed to be the points of lateral support against lateral-torsional buckling for calculating the lateral stability factor. The inputs for the flexural resistance are calculated as follows

$$\emptyset = 0.9$$

$$k_d = 1.0$$

$$k_m = 1.0$$

$$\frac{d}{b} = \frac{1634\,mm}{265\,mm} = 6.2 < 10$$

$$C_s = \sqrt{\frac{L_u d}{b^2}} = \sqrt{\frac{6000\,mm \times 1634\,mm}{(265\,mm)^2}} = 11.82 > 10 \; and \; < C_k$$

$$C_k = \sqrt{\frac{E_{05}}{f_{bu}}} = \sqrt{\frac{8512}{23.0}} = 19.24$$

$$\therefore k_{ls} = 1 - 0.3\left(\frac{C_s}{C_k}\right)^4 = 1 - 0.3\left(\frac{11.82}{19.24}\right)^4 = 0.96$$

$$k_{sb} = \left(\frac{130}{265}\right)^{\frac{1}{10}} \left(\frac{610}{1634}\right)^{\frac{1}{10}} \left(\frac{9100}{18000}\right)^{\frac{1}{10}} = 0.79 \leq 1.3$$

$$\therefore k_{sb} = 0.79$$

$$S = \frac{bd^2}{6} = \frac{265\,mm \times (1634\,mm)^2}{6} = 117923 \times 10^3 \, mm^3$$

The resulting factored flexural resistance is equal to the lesser of

$$M_{r,girder,ext} = \phi k_d k_{ls} k_m f_{bu} S$$
$$M_{r,girder,ext} = 0.9 \times 1.0 \times 0.96 \times 1.0 \times 23.0\,MPa \times 117923 \times 10^3 \, mm^3 = 2344\,kNm$$

and

$$M_{r,girder} = \phi k_d k_m k_{sb} f_{bu} S$$
$$M_{r,girder,ext} = 0.9 \times 1.0 \times 1.0 \times 0.79 \times 23.0\,MPa \times 117923 \times 10^3 \, mm^3 = 1928\,kNm$$

$$\therefore M_{r,girder,ext} = 1928\,kNm$$

Similar calculations for the interior girders yields $M_{r,girder,int} = 1592\ kNm$

The factored flexural demand was calculated to be equal to

$$M_{f,girder,ext} = 1400\ kNm < M_{r,girder,ext} = 1928\ kNm$$

$$M_{f,girder,int} = 778\ kNm < M_{r,girder,int} = 1592\ kNm$$

Therefore, the girders have sufficient flexural capacity.

3.3.5.3.3. SHEAR DESIGN

The shear resistance of the exterior girders, $V_{r,girder,ext}$, is calculated in the same manner as for the floor beams. The inputs for the shear resistance are calculated as follows

$$\emptyset = 0.9$$

$$k_d = 1.0$$

$$k_m = 1.0$$

$$k_{sv} = V^{-0.18} = (0.265\ m \times 1.634\ m \times 18.000\ m)^{-0.18} = 0.69$$

$$A = bd = 265\ mm \times 1634\ mm = 433010\ mm^2$$

The resulting factored shear resistance is equal to

$$V_{r,girder,ext} = \frac{\phi k_d k_m k_{sv} f_{vu} A}{1.5} = \frac{0.9 \times 1.0 \times 1.0 \times 0.69 \times 1.18\ MPa \times 433010\ mm^2}{1.5} = 211\ kN$$

Similar calculations for the interior girders yields $V_{r,girder,int} = 178\ kN$

The factored shear load was calculated to be equal to

$$V_{f,girder,ext} = 184\ kNm < V_{r,girder,ext} = 211\ kN$$

$$V_{f,girder,int} = 106\ kNm < V_{r,girder,int} = 178\ kN$$

Therefore, the girders have sufficient shear capacity.

3.3.5.4. GLUED-LAMINATED TIMBER DIAPHRAGMS

The glued-laminated timber diaphragms serve to brace the girders against lateral-torsional buckling, maintain the relative spacing of the girders, and transmit lateral load between girders (Ritter 1992). They are required at all supports, per clause 9.20.2 of the CHBDC. They are also required at midspan for spans less than 12 m and at the span third-points for spans of 12 m or greater. A designer may wish to add more diaphragms than required by the CHBDC to increase the lateral stability factor for girder flexure design. Ritter (1992) recommends that the diaphragm spacing not exceed 7.62 m (25 feet), regardless of the span. The diaphragms are provided at the abutments and span third-points in this design example because the bridge span exceeds 12 m. The abutment diaphragms are offset 600 mm forward from the bearings to not interfere with the bearing assemblies. CHBDC commentary clause C9.20.2 states that abutment diaphragms remain effective when offset up to one girder depth from the bearings.

The diaphragms should be as deep as possible, per CHBDC clause 9.20.2. Ritter (1992) advises leaving a gap of 50 mm to 125 mm between the top of the diaphragms and underside of the deck, to allow for air circulation and to preclude interference with deck attachment hardware. The diaphragms in this example have been detailed to provide a 76 mm gap between the diaphragms and the deck.

Ritter (1992) also advises locating the tie rods outside of the outer tension zone of the girders, which is usually considered to be the outer ten percent of the girder depth. The tie rods in this design example are located at 171 mm from the underside of the girder, which is outside the $1634\ mm \div 10 = 163\ mm$ deep outer tension zone. It is sometimes convenient to position the tie rods at the interface between adjacent laminations. In doing so, the tie rod holes can be routed into the outside faces of these laminations before they are glued together. The tie rods have been positioned 95 mm from the top and bottom faces of the diaphragms for this design example, which is exactly equal to 2.5 laminations.

The primary purpose of the diaphragms is to brace the girders against lateral-torsional buckling. Analogous to steel design, 2% of the total compression force acting within the compression zone of a girder at maximum factored bending moment is taken as the brace force. The maximum factored moment in a girder is 1400 kNm. Assuming a linear-elastic stress distribution, the lever between the resultant tensile and compression forces is

$$e = 2 \times \left(\frac{1.634\ m}{2} \times \frac{2}{3} \right) = 1.090\ m$$

The total compression force within the compression zone of the girder is

$$C_f = \frac{M_f}{e} = \frac{1400\ kNm}{1.090\ m} = 1285\ kN$$

The lateral brace force to be resisted by the diaphragm, in either tension or compression, is

$$F_{brace} = \pm\, 0.02 \times C_f = \pm\, 0.02 \times 1285\ kN = \pm\, 26\ kN$$

Brace forces acting in compression are transmitted to the diaphragms through direct bearing between the side faces of the girders and the end faces of the diaphragms. Brace forces acting in tension are transmitted to the diaphragms by the tie rods.

The diaphragms also experience forces arising from external loads acting on the bridge. The tie rods were modelled directly in the computer structural analysis model to capture these forces. The maximum factored force experienced by a tie rod due to external loads is 46 kN in tension. Thus, the maximum factored tie rod force, considering both forces arising from external loads and bracing of the girders, is equal to

$$T_f = 46 \ kN + 26 \ kN = 72 \ kN$$

It is typical practice to perform one tie rod design based on the maximum factored tie rod force. The use of different diameter tie rods can be confusing on site, leading to construction errors. This practice is not recommended. A single tie rod design is presented in this design example in section 7.3.

The diaphragms in this design example are 130 mm wide, 1482 mm deep, and 885 mm to 935 mm long, depending on which girders they are positioned between. From the computer structure analysis model, the maximum axial stress in a diaphragm is 1.09 MPa. Using the tie rod spacing of 1292 mm, the additional factored axial stress in the diaphragm due to bracing the girders at ULS is

$$\frac{26 \ kN}{130 \ mm \times 1482 \ mm} \times \frac{1000 \ N}{kN} + \frac{26 \ kN \times 1.292 \ m}{130 \times 1482^2 \div 6} \times \frac{1000 \ N}{kN} \times \frac{1000 \ mm}{m}$$
$$= 0.14 \ MPa + 0.71 \ MPa = 0.85 \ MPa$$

Therefore, the total factored axial stress in the diaphragms is

$$\sigma_{f,diaphragm} = 1.09 \ MPa + 0.85 \ MPa = 1.94 \ MPa$$

Analogous to the flexural resistance presented for the girder design, the flexural resistance of the diaphragms is taken as the lesser of

$$\sigma_{r,diaphragm} = M_{r,diaphragm} \div S = \phi k_d k_{ls} k_m f_{bu}$$

and

$$\sigma_{r,diaphragm} = M_{r,diaphragm} \div S = \phi k_d k_m k_{sb} f_{bu}$$

where

$$\emptyset = 0.9$$

$$k_d = 1.0$$

$$k_m = 1.0$$

$$\frac{d}{b} = \frac{1406\ mm}{130\ mm} = 10.82 < 10$$

$$C_s = \sqrt{\frac{L_u d}{b^2}} = \sqrt{\frac{6000\ mm \times 1634\ mm}{(265\ mm)^2}} = 11.82 > 10\ and\ < C_k$$

$$C_k = \sqrt{\frac{E_{05}}{f_{bu}}} = \sqrt{\frac{8512}{23.0}} = 19.24$$

$$\therefore k_{ls} = 1 - 0.3 \left(\frac{C_s}{C_k}\right)^4 = 1 - 0.3 \left(\frac{10.82}{19.24}\right)^4 = 0.97$$

$$k_{sb} = \left(\frac{130}{130}\right)^{\frac{1}{10}} \left(\frac{610}{1406}\right)^{\frac{1}{10}} \left(\frac{9100}{935}\right)^{\frac{1}{10}} = 1.15 \leq 1.3$$

$$\therefore k_{sb} = 1.15$$

$$M_{r,diaphragm} \div S = \phi k_d k_{ls} k_m f_{bu}$$
$$\sigma_{r,diaphragm} = 0.9 \times 1.0 \times 0.97 \times 1.0 \times 23.0\ MPa = 20.1\ MPa$$

and

$$M_{r,diaphragm} \div S = \phi k_d k_m k_{sb} f_{bu}$$
$$\sigma_{r,diaphragm} = 0.9 \times 1.0 \times 1.0 \times 1.15 \times 23.0\ MPa = 23.8\ MPa$$

$$\therefore \sigma_{r,diaphragm} = 20.1\ MPa > \sigma_{f,diaphragm} = 1.94\ MPa$$

Therefore, the diaphragms have sufficient capacity.

3.3.6. CONNECTION DESIGN

The bridge design features four major connections:

- The deck-to-floor beam, connection;
- The floor beam-to-girder connection;
- The diaphragm connection; and,
- The girder bearing connection.

The design of these four major connections is explained in this section of the bridge design example.

3.3.6.1. DECK-TO-FLOOR BEAM CONNECTION

The deck-to-floor beam connections must resist vertical forces acting upwards and downwards, transverse forces, and longitudinal forces. The connection consists of a series of 25 mm diameter through-bolts passing through the deck and floor beams. The bolts are spaced at an average of 383 mm on centre, which exceeds the minimum of two 19 mm bolts per metre that is required by CHDBC clause 9.23.5.5.

The bolts have been designed to resist the forces experienced by the loads described in section 4.0 of this example. Uplift forces are resisted by the bolts in tension. Downward forces are resisted by direct bearing between the deck and floor beams. Longitudinal and transverse forces are resisted by the bolts in shear.

The design of the deck-to-floor beam connections is as follows:
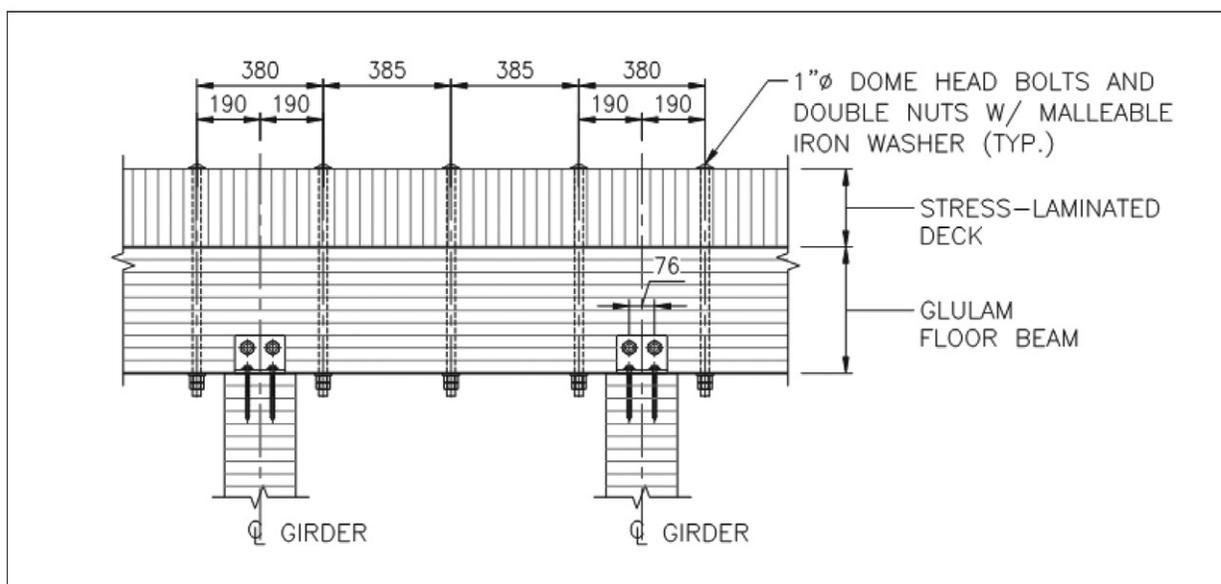
Figure 3.3.11. – Deck to floor beam connection

25Ø ENLARGED HEAD BOLT

$$T_f^{UP} = 5kN$$

WASHER: 64Ø

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(2520)x1.0x1.0x10^{-3}$$

$$= 7.8kN > T_f^{up} = 5kN \; OK \; (64\%)$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \; (WET \; SERVICE \; CONDITION \; AT \; CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

BOLT IN GLULAM (CSA O86-14)

$$V_f = \sqrt{8.2^2 + 0.2^2} = 8.2kN/BOLT \; , SINGLE \; SHEAR$$

$$K_{SF} = 0.67 \; (WET \; SERVICE)$$

$$K_{SV} = 0.87$$

$$K_{st} = 0.75$$

$$K_{lS} = 0.65$$

NOTE: ASSUME WOOD DECK IS MONOLITHIC, SOLID TIMBER LOADED PERPENDICULAR -TO-GRAIN.

TRY 1 ROW OF BOLTS, 25Ø

STIFFENER

$$P_r = 64kN > V_f = 8.2kN \ (BRITTLE \ RESISTANCE) \ OK$$

$$QS_r = 56kN > T_f^{UP} = 5kN \ (SPLITTING \ RESISTANCE) \ OK$$

$$N_r = 56kN > V_f = 8.2kN \ (BRITTLE \ RESISTANCE) \ OK$$

DECK

$$P_r = 103kN > V_f = 8.2kN \ (BRITTLE \ RESISTANCE) \ OK$$

$$QS_r = 28kN > T_f^{UP} = 5kN \ (SPLITTING \ RESISTANCE) \ OK$$

$$N_r = 103kN > V_f = 8.2kN \ (BRITTLE \ RESISTANCE) \ OK$$

$$N_r = 13kN > V_f = 8.2kN \ (YIELDING \ RESISTANCE) \ OK$$

3.3.6.2. FLOOR BEAM-TO-GIRDER CONNECTION

The floor beam-to-girder connections must resist vertical forces acting upwards and downwards, transverse forces, and longitudinal forces. The connection consists of two steel angles through-bolted to a floor beam. The angles are fastened to the tops of the girders using lag screws. This connection is present at every intersection of the floor beams and girders (i.e. 7 floor beams x 12 girders = 84 connections).

The connection has been designed to resist the forces experienced by the loads described in section 4.0 of this example. Uplift forces are resisted by shear in the through-bolts, tension in the vertical legs of the angles, and tension in the lag screws. Downward forces are resisted by direct bearing between the floor beams and girders. Longitudinal forces are resisted by the floor beams bearing on the angles and shear in the lag screws. Transverse forces are resisted by shear in the through-bolts, shear in the vertical legs of the angles, and shear in the lag screws.

The design of the floor beam-to-girder connections is as follows:
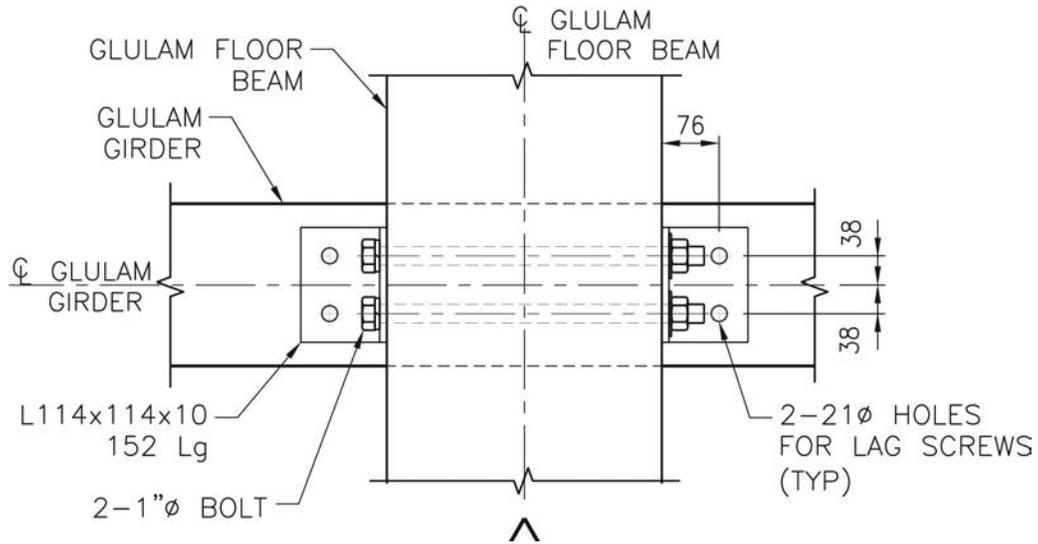
Figure 3.3.12. – Floor beam to girder connection

LAG SCREWS
try 4-19Ø Lags by 152 long

SPF, WET

$$P'_{rw} = 84x141x4lagsx(1.0x0.67x1.0x1.0)x10^{-3} = 31.7kN > T_f^{up} = 13kN\ (41\%), OK$$

note: check lags for prying (not shown)

$$Q_r = Q'_r n_{Fe} n_R K'$$

$$Q_r = 8.74x2x2x(1.0x0.67x1.0) = 23.4kN > V_f = 11kN\ (47\%), OK$$

Where,

$$Q_r' = 8.74 kN$$

$$K' = K_D K_{SF} K_T$$

$$n_{Fe} = 2$$

$$n_R = 2$$

$$K_D = 1.0$$

$$K_{SF} = 0.67$$

$$K_T = 1.0$$

BOLTS IN GLULAM (CSA O86-14)

$$P_f = 11 kN \text{ (TRANSVERSE LOAD)}$$

$$Q_f^{UP} = 13 kN$$

$$N_f^{40°} = \sqrt{13^2 + 11^2} = 17 kN$$

$$K_{SF} = 0.67 \ (WET \ SERVICE)$$

$$K_{SV} = 0.87$$

$$K_{St} = 0.75$$

$$\theta = \text{atan}\left(\frac{11}{13}\right) = 5.9°$$

S-W-S (STEEL SIDE PLATES)

TRY 1 ROW OF 2 BOLTS, 19Ø, A307:

$$P_r = 71kN > P_f = 11kN \ (BRITTLE \ RESISTANCE) \ OK$$

$$QS_r = 23kN > Q_f^{UP} = 13kN \ (SPLITTING \ RESISTANCE) \ OK$$

$$N_r = 43kN > N_f^{40°} = 17kN \ (YIELDING \ RESISTANCE) \ OK$$

$$N_r = 32kN > N_f^{40°} = 17kN \ (BRITTLE \ RESISTANCE) \ OK$$

3.3.6.3. DIAPHRAGM-TO-GIRDER CONNECTION

The diaphragm-to-girder connection consists of a pair of threaded rods through-bolted through a routed slot along the length of the diaphragms and through the side faces of adjacent girders. The design is as follows:

RODS

$$T_f = 73kN$$

Use 22Ø A307 rod with threaded ends

$$T_r = 80.7kN > T_f = 73kN \ OK \ (91\%)$$

WASHER: 170X170X12.7

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(170x170)x1.0x1.0x10^{-3}$$

$$= 90kN > T_f = 73kN \ OK \ (82\%)$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp}\left(K_D K_{S_{cp}} K_T\right)$$

$$f_{cp} = 5.8 MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET\ SERVICE\ CONDITION\ AT\ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

NOTE: Designer to also check washer plate thickness.

BEARING ON SIDE OF GIRDER

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(130x200)x1.0x1.0x10^{-3} = 81kN > T_f$$
$$= 73kN\ OK\ (90\%)$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp}\left(K_D K_{S_{cp}} K_T\right)$$

$$f_{cp} = 5.8 MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET\ SERVICE\ CONDITION\ AT\ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

3.3.6.4. GIRDER-TO-BEARING CONNECTION

The girder-to-bearing connection design is as follows.

BEARING (6.5.9.2 O86-01)

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(215x500)x1.0x1.15x10^{-3} = 385kN \geq F = 371kN$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp}\left(K_D K_{S_{cp}} K_T\right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.15 \ (RATIO = \frac{215}{38} = 5.65 > 2)$$

BOLTS IN GLUED-LAMINATED (10 O86-01)

$$P_f = 87kN$$

$$Q_f = 9kN$$

$$N_f^{3.4°} = \sqrt{87^2 + 9^2} = 87.5kN$$

$$K_{SF} = 0.67 \ (WET \ SERVICE)$$

$$K_{SV} = 0.87$$

$$K_{st} = 0.75$$

$$\theta = atan\left(\frac{9}{87}\right) = 5.9°$$

S-W-S (STEEL SIDE PLATES)

TRY 2 ROWS OF 2 BOLTS, 19Ø, A307:

$$P_r = 104kN > F_x = 87kN \ (BRITTLE \ RESISTANCE) \ OK$$

$$QS_R = 17kN > F_z^{UP} = 9kN \ (SPLITTING \ RESISTANCE) \ OK$$

$$N_r = 114kN > N_f^{5.9°} = 87.5kN \ (YIELDING \ RESISTANCE) \ OK$$

$$N_r = 103kN > N_f = 87.5kN \ (BRITTLE \ RESISTANCE) \ OK$$

ANCHOR BOLTS (10 O86-01)

Use 4-13 Ø galvanized anchor bolts using epoxy system per manufacturer's requirements.

$$T_r = 33kN/BOLT \ x0.9x0.78x4 \ BOLTS = 93kN \ > F_z^{UP} = 9kN$$

$$V_r = 83kN/BOLT \ x0.9x0.42x4 \ BOLTS = 125kN \ > F_x = 87kN$$

Unity check:

$$\left(\frac{9}{93}\right)^{\frac{5}{3}} + \left(\frac{87}{125}\right)^{\frac{5}{3}} = 0.57 < 1.0 \ OK$$

ECCENTRICITY OF F_X ON ANCHORS:

$$M_f = 87x0.14m = 12.2kN.m$$

$$T_f = C_f = \frac{M_f}{e} = \frac{12.2}{0.204} = 60kN \div 2 \ BOLTS = 30kN/BOLT$$

$$T_r = \frac{93}{4} = 23.3kN/BOLT$$

∴ USE LARGER BOLT: TRY 19 Ø

$$T_r = 55x0.91x0.88 = 44kN/BOLT \ > T_f = \frac{9}{4BOLTS} + 30 = 32kN/BOLT$$

$$V_r = 107x0.91x0.56 = 54.5kN/BOLT \ > V_f = \frac{87}{4BOLTS} = 22kN/BOLT$$

UNITY CHECK:

$$\left(\frac{32}{44}\right)^{\frac{5}{3}} + \left(\frac{22}{54.5}\right)^{\frac{5}{3}} = 0.81 < 1.0 \ OK$$

Anchor bolt strength

$$T_r = 66kN > T_f = 32kN$$

$$V_r = 34kN > V_f = 22kN$$

∴ USE 4-19 Ø galvanized anchor bolt with 170 MIN. EMBED

NOTE: Designer to check bearing plate thickness for gravity and uplift forces

UPPER CLIPS (10 O86-01)
ESTIMATE SIDE FORCE

$$M_r = 1380kN.m$$

$$T_f = C_f = \frac{1380}{1.55} = 886kN \ (AT \ MID - SPAN)$$

say 2% is torsional components for restraint:

$$C_f^b = 0.02x886 = 17.7kN$$

assume 200x100 bearing area on steel angle (6.5.9.2 O86-01):

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1x0.67x1.0)x(200x100)x1.0x1.0x10^{-3} = 62kN > 17.7kN \ OK$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp} \left(K_D K_{S_{cp}} K_T \right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{z_{cp}} = 1.0$$

Note: Diaphragms near support will also restrain. Designer to check angles for bending & prying on anchor bolts.

SIDE PLATES AT BOLTS (6.5.9.2 O86-01)
Bearing on wood

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$

$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(356x25)x1.0x1.0x10^{-3} = 28kN > F_y = 12kN$$

Where,

$$\phi = 0.8$$

$$F_{cp} = f_{cp}\left(K_D K_{S_{cp}} K_T\right)$$

$$f_{cp} = 5.8MPa$$

$$K_D = 1.0$$

$$K_{S_{cp}} = 0.67 \ (WET \ SERVICE \ CONDITION \ AT \ CONNECTION)$$

$$K_T = 1.0$$

$$K_B = 1.0$$

$$K_{Z_{cp}} = 1.0$$

Note: Designer to check plate bending.

3.4. OTHER DESIGN ELEMENTS

3.4.1. BEARINGS

Each bridge girder is supported on an elastomeric bearing at each abutment. It is preferable to specify the bearing width, measured perpendicular to the direction of traffic, to be almost as wide as the girder. Ritter (1992) recommends that the bearing be one quarter inch (6 mm) narrower than the girder. The bearing length is established by satisfying the average bearing pressure limits specified in clause 11.6.6.7 of the CHBDC. The bearing length should also be long enough to preclude a perpendicular to grain compression failure at the underside of the girder.

Elastomeric bearings used in bridges tend to be laminated elastomeric bearings. These bearings contain steel plates that afford the elastomer internal restraint against volumetric changes due to the Poisson effect. Elastomeric bearings for wood bridges have traditionally been plain elastomeric bearings. Recent field observations suggest poor performance of unconfined plain elastomeric bearings. These bearings have been observed to undergo significant bulging under compressive loads due the Poisson effect, because unlike laminated bearings, they contain no inherent internal restraint against volumetric changes. When the compressive force is reduced, a plain bearing tends to bulge less and become restored to its unloaded shape. This cycling of bulging and shape restoration creates the potential for a plain bearing to migrate from its initial position due to inconsistent friction between the bearing and the materials that it is in contact with. Furthermore, the bulging of a plain bearing can result in compressive deflections more than the limit of 7% of the total effective elastomer thickness that is specified in clause 11.6.6.4 of the CHBDC. For these reasons, it is recommended that homogenous fabric fibre reinforced elastomeric bearings or laminated elastomeric bearings be used to support wood bridge girders. The former has been specified for the bearings in this design example.

Per clause 11.6.6.6 of the CHBDC, positive attachment of the bearings is required to prevent displacement of the bearings when the average bearing pressure at SLS drops below 1.5 MPa or when the shear force in the bearing exceeds the frictional force acting between the bearing and the materials that it is in contact with. This possibility is common for wood bridges because they are light structures. A simple form of positive attachment is to provide keeper plates around the bearing to prevent its displacement. Keeper plates have been specified for the bearings in this design example because the minimum average SLS bearing pressure is less than 1.5 MPa.

Although not included in this design example, the substructure design should consider jacking and shoring of the superstructure to facilitate bearing replacement. The detailing of wood bridges at the abutments does not tend to leave space for the placement of hydraulic jacks to lift the superstructure. Furthermore, the connections between the diaphragms and girders in wood bridges are not typically designed to transfer the forces generated by using the diaphragms as jacking points. As a result, most wood bridges cannot be jacked directly off the abutment bearing seat. Potential solutions for jacking the superstructure include the use of jacking corbels anchored into the outside face of the abutment stem, or the use of jacking posts extending upward off of the toe of the abutment footing. The design of the substructure should consider the eccentricity resulting from either of these approaches.

3.4.2. BARRIERS

The bridge barrier specified in this design example is a crash-tested TL-4 barrier comprised of glued-laminated timber. The reference documents cited for the barrier also provides details for a steel barrier that can be connected to timber bridge decks. Refer to these documents if a steel bridge barrier is preferential relative to a timber bridge barrier.

Some jurisdictions have experienced difficulties with the maintenance of timber bridge railings due to damage caused by snowplow blades. An approach guiderail transition should always be specified at the end of the bridge barrier, so as to eliminate the potential for direct impact by snowplow blades and vehicles. The reference documents for the timber barrier provides details of a crash-tested approach guiderail transition. The local bridge authority may also have standard details for this transition. Consideration should be given to providing a steel armouring plate on the inside face of the curb rails to prevent localized crushing failure caused by impact from snowplow blades.

The crash tests performed on the TL-4 barriers specified in this design example were performed on barriers comprised of American Southern Pine. These tests were performed in the 1990's. The design material properties of Southern Pine have changed since that time. Table 6.2.1.3 of CSA O86 provides lumber species equivalents for Canadian and American species. Southern Pine is not presently listed in the table. It was, however, stated as equivalent to Canadian S-P-F in the 2001 edition of CSA O86. It currently uncertain if using Canadian S-P-F for the specified TL-4 barrier is equivalent or better than using American Southern Pine. The designer is cautioned to ensure that the specified barrier material has sufficient strength to be equivalent to the crash-tested barrier.

3.4.3. DURABILITY

All wood components should be preservative treated with a preservative specified in clause 9.17.1 of the CHBDC. Wood preservatives chemicals mitigate decay by fungus and insect attack. It is generally preferential to treat glued-laminated timber members after gluing with an oil-borne preservative. Oil-borne preservatives provide superior dimensional stability due to variations in moisture, ultimately minimizing the potential for the development of checks that can expose untreated wood to decay. The wood should be incised to increase the depth of preservative treatment. The treatment process should not leave the member with excess preservative chemicals, as these chemicals tend to leach out into the surrounding environment and/or interfere with the bond between the wearing surface and the wood deck.

The choice of wood preservative should consider the potential for pedestrian contact. While heavy oil-borne preservatives are preferred for treating structural wood members, they may not be preferential for treating handrails that will be contacted by pedestrians. Clause 9.17.2 of the CHBDC specifies preservatives that are suitable for pedestrian contact. Another approach is to specify a naturally durable wood species for elements that experience pedestrian contact, to preclude the need for preservative treatment.

Preservative treatment is one method of enhancing the durability of wood bridges. The bridge should also be "detailed for durability". In doing so, the bridge should be detailed to minimize the potential for trapped moisture. Some designers and owners have also detailed exposed wood elements with protective metal flashing.

All steel hardware for the stress-laminated deck, including the post-tensioning bars, anchorage plates, anchorage nuts, and steel channel bulkhead should be hot-dipped galvanized in accordance with ASTM A123/A123M. Any damage to the galvanizing should be repaired using two coats of zinc-rich paint. The bars should also be coated with an anti-corrosion paste after galvanizing. A plastic cap should be fitted around the post-tensioning bar extensions and anchorage nuts after all stressing operations have been completed, in order to protect against corrosion and tampering. The cap should be packed with anti-corrosion paste.

3.4.4. INSPECTION

The design of bridge should always include provisions for inspection of the completed structure. The end faces of wood members are more prone to decay than the other faces. As such, it is recommended that a gap be detailed between the ends of the girders and the ballast wall for these faces to be accessible for inspection. This feature can be accomplished by detailing the top of the outside face of the ballast wall with a corbel. The corbel will serve to create this gap, without leaving a gap for traffic. To further enhance this accessibility, the slope protection should be detailed with a bench in front of the abutment, so that future inspectors have flat, competent material to stand on for inspection.

3.5. REFERENCES

American Institute of Timber Construction. (2012). *Timber Construction Manual*. Hoboken, NJ: John Wiley & Sons Inc.

APA – The Engineered Wood Association (2013). *Technical Note: Preservative Treatment of Glued Laminated Timber*. Tacoma, WA.

Bakht, B. (1988). Load Distribution in Laminated Timber Decks, *Journal of Structural Engineering*, 114(7), pp. 1551-1570.

Bakht, B. and Jaeger, L.G. (1991). Load Sharing Factor in Timber Bridge Designs, *Canadian Journal of Civil Engineering*, 18(2), pp. 312-319.

Canadian Standards Association. (2006). *CAN/CSA-O177-06 Qualification Code for Manufacturers of Structural Glued-Laminated Timber*. Mississauga: Canadian Standards Association.

Canadian Standards Association. (2014a). *CAN/CSA-S6-14 Canadian Highway Bridge Design Code*. Mississauga: Canadian Standards Association.

Canadian Standards Association. (2014b). *S6.1-14 Commentary on CAN/CSA-S6-06, Canadian Highway Bridge Design Code*. Mississauga: Canadian Standards Association.

Canadian Standards Association. (2014c). *CAN/CSA-O86-04 Engineering Design in Wood*. Mississauga: Canadian Standards Association.

Canadian Standards Association. (2015). *CSA O80 Series-15 Wood Preservation*. Mississauga: Canadian Standards Association.

Canadian Standards Association. (2016). *CAN/CSA-O122-16 Structural Glued-Laminated Timber*. Mississauga: Canadian Standards Association.

Eriksson, M., Wheeler, H., and Kosmalski, S. (2003). *Asphalt Paving of Treated Timber Bridge Decks*. Technical Report 0371-2809P-MTDC. Missoula, MT: United States Department of Agriculture, Forest Service, Missoula Technology and Development Center.

Faller, R.K., Ritter, M.A., Rosson, B.T., Fowler, M.D., and Duwadi, S.R. (2000). Two Test Level 4 Bridge Railing and Transition Systems for Transverse Timber Deck Bridges. *Transportation Research Record: Journal of the Transportation Research Board*, Volume 1696: Fifth International Bridge Conference, Vols. 1 & 2, pp. 334-351.

Forest Products Laboratory (2010). *Wood Handbook – Wood as an Engineering Material*. General Technical Report FPL-GTR-190. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

Jaeger, L.G. and Bakht, B. (1990). Effect of Butt Joints on the Flexural Stiffness of Laminated Timber Bridges, *Canadian Journal of Civil Engineering*, 18(2), pp. 859-864.

McCutcheon, W.J., and Tuomi, R.L. (1973). *Procedure for Design of Glued-Laminated Orthotropic Bridge Decks*. Research Paper FPL 210. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

McCutcheon, W.J., and Tuomi, R.L. (1974). *Simplified Design Procedure for Glued-Laminated Bridge Decks*. Research Paper FPL 233. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

Ontario Ministry of Transportation. (1985). *Geometric Design Standards for Ontario Highways*. North York: Surveys and Design Office.

Ontario Ministry of Transportation. (2002). *Revision Information Sheet for Geometric Design Standards for Ontario Highways*. North York: Surveys and Design Office.

Polivka, K.A., Faller, R.K., Ritter, M.A., Rosson, B.T., Fowler, M.D., and Duwadi, S.R. (2002). *Two Test Level 4 Bridge Railing and Transition Systems for Transverse Timber Deck Bridges, FP-95-RJVA-2630*. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

Ritter, M.A. (1992). *Timber Bridges – Design, Construction, Inspection, and Maintenance*. Washington, D.C.: United States Department of Agriculture, Forest Service, Engineering Staff.

Ritter, M.A., Faller, R.K., Lee, P.D.H., Faller, R.K., Rosson, B.T., and Duwadi, S.R. (1995). *Plans for Crash Tested Bridge Railings for Longitudinal Wood Decks*. General Technical Report FPL-GTR-87. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

Sexsmith, R.G., Boyle, P.D., Rovner, B., and Abbott, R.A. (1979). *Load Sharing in Vertically Laminated Post-Tensioned Bridge Decking*. Vancouver, BC: Forintek Canada Corporation.

Wacker, J.P. and Smith, M.S. (2001). *Standard Plans for Timber Bridge Superstructures*. General Technical Report FPL-GTR-125. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

Weyers, R.E., Loferski, J.R., Dolan, J.D., Haramis, J.E., Howard, J.H., and Hislop, L. (2001). *Guidelines for Design, Installation, and Maintenance of a Waterproof Wearing Surface for Timber Bridge Decks*. FPL-GTR-123. Madison, WI: United States Department of Agriculture, Forest Service, Forest Products Laboratory.

Witmer, R.W., Manbeck, H.B., Janowiak, J.J., and Schram, P. (2002). Reinforcing Transverse Glulam Deck Panels with Through-bolted Glulam Stiffener Beams: Theoretical Analysis, *Journal of Bridge Engineering*, 7(6), pp. 367-371.

3.6 LIST OF ABBREVIATIONS

A = cross-sectional area of a member or the bearing area, mm^2

$ADTT$ = average daily truck traffic

$ADTT_f$ = single lane average daily truck traffic for fatigue

$AADT_1$ = average annual daily traffic for the first year after construction

b = width of a member or lamination, mm

B = bridge width

B_e = barrier exposure index

C_e = wind exposure coefficient

C_g = wind gust effect coefficient

C_h = horizontal wind drag coefficient

C_L = correction factor for fatigue truck weight

C_v = vertical wind load coefficient

d = depth of a member or lamination, mm

D = dead load

D_e = width over which elements sharing load deform substantially uniformly, m

D_T = truck load distribution width, m

E = modulus of elasticity

E_{05} = 5th percentile of the modulus of elasticity, MPa

E_{50} = 50th percentile of the modulus of elasticity, MPa

f_b = specific bending strength, MPa

f_c = specified compression strength parallel to grain, MPa

f_{cp} = specified compression strength perpendicular to grain, MPa

f_{ql} = limiting pressure perpendicular to the grain, MPa

f_{sr} = calculated FLS stress range at the detail due to passage of the CL-W Truck or of a tandem set of axles, MPa

f_t = specified tensile strength parallel to the grain, MPa

f_v = specified shear strength, MPa

F_S = skew factor in Clause 5.6.

F_{sr} = fatigue stress range resistance, MPa

F_{srt} = constant amplitude threshold stress range, MPa

F_T = truck load fraction as calculated by the simplified method of analysis and used to generate the design longitudinal load effects

F_h = horizontal wind load per unit exposed frontal area, Pa

F_v = vertical wind load per unit exposed plan area, Pa

G = shear modulus

I = moment of inertia of a section, mm^4; nominal (unfactored) dynamic component of the live load, expressed as a fraction of the nominal static live load effect (dynamic load allowance)

I_D = dynamic load allowance, as specified in Clause 3.8.4.5

K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains

K_b = modification factor for the effect of butt joints on the stiffness of laminated wood decks

K_c = highway curvature factor

K_D = modification factor for duration of load

K_g	=	highway grade factor
K_h	=	highway type factor
K_L	=	modification factor for lateral stability
K_m	=	modification factor for load sharing
K_{Sb}	=	modification factor for service condition for bending
K_s	=	superstructure height factor
K_{Sc}	=	modification factor for service condition for compression parallel to grain
K_{Scp}	=	modification factor for service condition for compression perpendicular to grain
K_{SE}	=	modification factor for service condition for modulus of elasticity
K_{SF}	=	modification factor for service condition for connections
K_{St}	=	modification factor for service condition for tension parallel to grain
K_{Stp}	=	modification factor for service condition for tension perpendicular to grain
K_{Sv}	=	modification factor for service condition for longitudinal shear
K_T	=	modification factor for treatment
K_V	=	shear modification factor
K_{Zbg}	=	modification factor for the size effect for flexure for glued-laminated timber
K_{Zc}	=	modification factor for the size effect for compression parallel to the grain
K_{Zcp}	=	modification factor for the length of bearing for compression perpendicular to the grain
K_{Zv}	=	modification factor for the size effect for shear
L	=	length of a component, mm; span length, m; live load
L_e	=	the equivalent span length specified for the uses of the beam analogy method in Clause 5.6, m

L_u = laterally unsupported length of a component, mm

l_u = unsupported length, mm

M_L = for girder-type bridges, the longitudinal moment per girder due to the CL-W loading

M_r = factored resistance of a member in flexure, Nmm

M_T = longitudinal moment generated by one lane of CL-W loading

n = number of design lanes on a bridge

N = number of girders or longitudinal wood beams in the bridge deck width B

N_b = a measure of the frequency of butt joints in laminated wood decks, being, for any 1.0 m wide band perpendicular to the laminates, the minimum number of laminates without joints adjacent to a laminate having a butt joint

N_c = specified number of design stress cycles

N_d = number of design stress cycles experienced for each passage of the design truck

P = wheel load of the CL-625 Truck

P_l = longitudinal traffic load on barrier, kN

P_t = transverse traffic load on barrier, kN

P_v = vertical traffic load on barrier, kN

q = hourly mean reference wind pressure for the design return period, Pa

R_L modification factor for multi-lane loading

S = section modulus, mm; centre-to-centre spacing of longitudinal girders of a deck-on-girder bridge, m

V = wind load on traffic

V_f = factored shear load on a member, kN

V_L = for girder-type bridges, the longitudinal shear per girder due to the CL-W loading

V_r	=	factored shear resistance, N
V_T	=	longitudinal shear generated by one lane of CL-W loading
W	=	wind load on structure
W_c	=	deck width, m
W_e	=	width of design lane, m
y	=	design life, years
Φ	=	resistance factor for wood components
Φ_s	=	resistance factor for steel components
α_D	=	load factor for dead load
α_w	=	load factor for wind effects
Δ_T	=	temperature differential, °C
ε	=	skew parameter for slab-on-girder bridges
γ	=	fatigue life constant
γ_c	=	truck load modification factor for slab-on-girder bridges
γ_e	=	truck load modification factor for the exterior girder of slab-on-girder bridges
λ	=	lane width parameter
μ	=	lane width modification factor
Ψ	=	skew angle

NORTH FOR CONSTRUCTION

NOTE:
ALL DIMENSION SHOWN ARE IN METRES AND/OR MILLIMETRES UNLESS NOTED OTHERWISE.
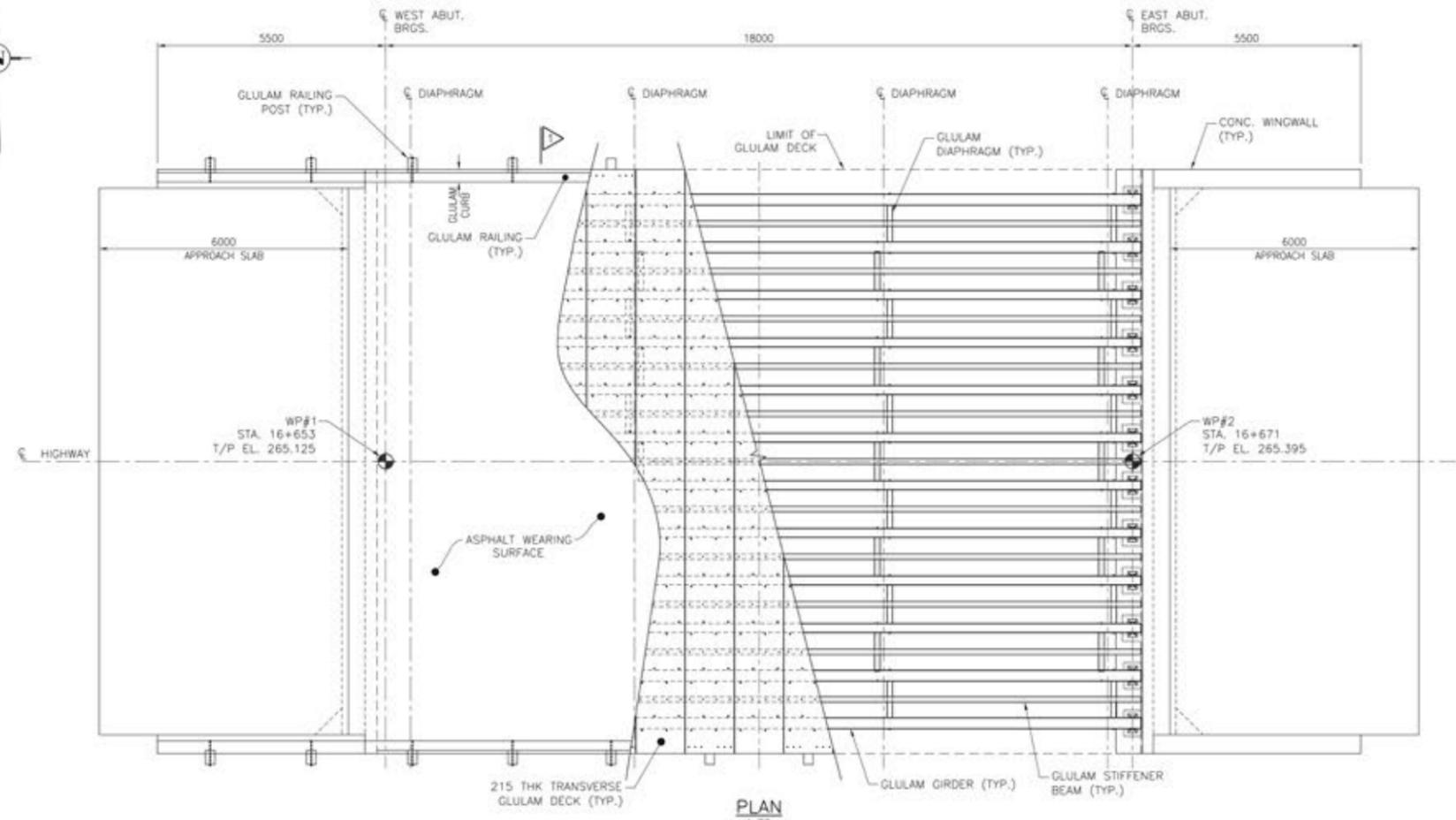
PLAN 1:75
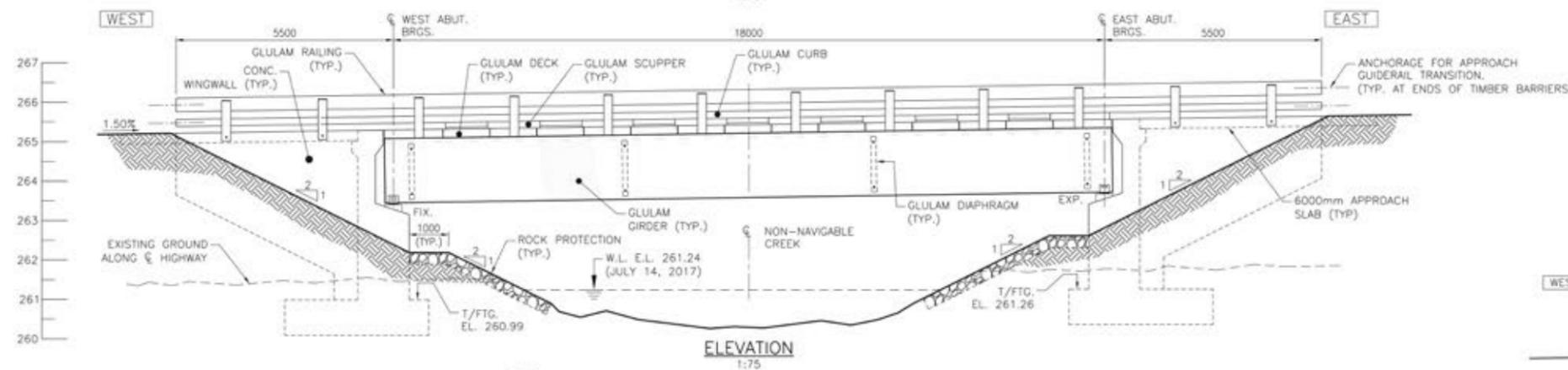
ELEVATION 1:75

SECTION 1:75

PROFILE OF HWY N.T.S.
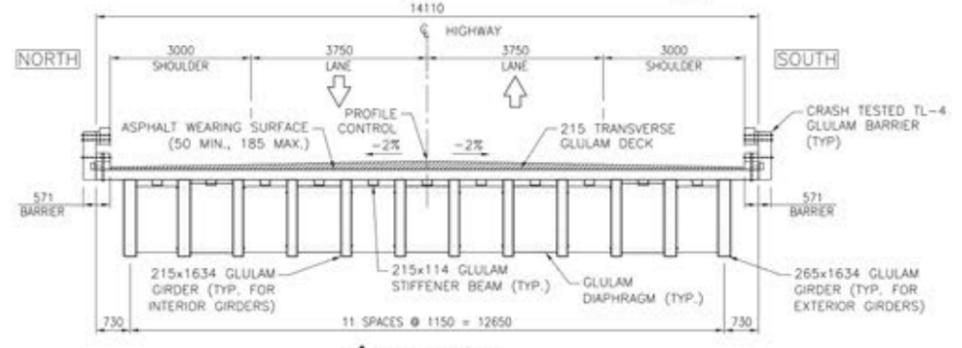
LIST OF DRAWINGS:

SWB-01	GENERAL ARRANGEMENT
SWB-02	FOOTING LAYOUT AND REINFORCEMENT*
SWB-03	ABUTMENTS AND WINGWALLS LAYOUT*
SWB-04	ABUTMENTS & WINGWALLS REINFORCEMENT*
SWB-05	GLUED-LAMINATED TIMBER FRAMING I
SWB-06	GLUED-LAMINATED TIMBER FRAMING II
SWB-07	GLUED-LAMINATED TIMBER DECK
SWB-08	GLUED-LAMINATED TIMBER BARRIER I
SWB-09	GLUED-LAMINATED TIMBER BARRIER II
SWB-10	6000mm APPROACH SLABS*
SWB-11	ROCK PROTECTION*

* - NOT INCLUDED IN THIS DESIGN EXAMPLE

LIST OF ABBREVIATIONS

WP	-	DENOTES WORK POINT
T/D	-	DENOTES TOP OF DECK
T/P	-	DENOTES TOP OF PAVEMENT
T/FTG.	-	DENOTES TOP OF FOOTING
ABUT.	-	DENOTES ABUTMENT
BRGS.	-	DENOTES BEARINGS
C.J.	-	DENOTES CONSTRUCTION JOINT
CONC.	-	DENOTES CONCRETE
FIX.	-	DENOTES FIXED BEARINGS
EXP.	-	DENOTES EXPANSION BEARINGS
TYP.	-	DENOTES TYPICAL
EXIST.	-	DENOTES EXISTING
EL.	-	DENOTES ELEVATION
MAX.	-	DENOTES MAXIMUM
MIN.	-	DENOTES MINIMUM

APPLICABLE STANDARD DRAWINGS

OPSD-3101.150	WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT.
OPSD-3102.100	WALLS, ABUTMENT, BACKFILL, DRAIN.
OPSD-3370.100	DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD.
OPSD-3941.200	FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT.
OPSD-3190.100	WALLS RETAINING AND ABUTMENT WALL DRAIN.

GENERAL NOTES:

1. DESIGN CODE
- CAN/CSA-S6-14 (CANADIAN HIGHWAY BRIDGE DESIGN CODE)
- LL: CL-625-ONT

2. TIMBER
- ALL WOOD MEMBERS SHALL BE COMPRISED OF S-P-F GLUED-LAMINATED TIMBER MANUFACTURED IN ACCORDANCE WITH CAN/CSA-0122 BY A PLANT CERTIFIED IN ACCORDANCE WITH CAN/CSA-0177.
- GLUED-LAMINATED STRESS GRADES SHALL BE AS FOLLOWS:
  GIRDERS   20f-E
  DECK PANELS   14t-E
  DIAPHRAGMS   20f-EX
  BARRIERS   20f-EX
- ALL GLUED-LAMINATED TIMBER SHALL HAVE A QUALITY APPEARANCE GRADE.
- ALL SPECIFIED WOOD SIZES ARE DRESSED SIZES.
- ALL WOOD SHALL BE PRESSURE PRESERVATIVE TREATED IN ACCORDANCE WITH CAN/CSA-080 AND AWPA STANDARDS M1, M2, AND M4.
- ALL TREATED WOOD SHALL BE CUT TO FINISHED SIZE. ALL SURFACING, HOLES, AND OTHER CUTS SHALL BE COMPLETED PRIOR TO PRESSURE PRESERVATIVE TREATMENT.
- NO CHAINS, HOOKS, OR PEAVES SHALL BE USED IN HANDLING TREATED WOOD. SURFACES EXPOSED BY FIELD CUTTING OR DRILLING SHALL BE TREATED BY SOAKING WITH AN APPROVED PRESERVATIVE EQUAL IN TOXICITY TO THE ORIGINAL PRESERVATIVE. THREE APPLICATIONS ARE REQUIRED, EACH TO AIR DRIED SURFACES.

3. CLASS OF CONCRETE
ALL CONCRETE   30 MPa

4. CLEAR COVER TO REINFORCING STEEL
FOOTINGS   100 ± 25mm
REMAINDER   70 ± 20mm

5. REINFORCING STEEL
- BAR MARKS WITH PREFIX "S" DENOTES STAINLESS STEEL BARS.
- STAINLESS REINFORCING STEEL SHALL BE TYPE 316 LN OR TYPE 2205 DUPLEX AND HAVE A MINIMUM YIELD STRENGTH OF 500MPa.
- NON-STAINLESS REINFORCING STEEL SHALL BE GRADE 400w UNLESS NOTED OTHERWISE.
- TENSION LAP LENGTHS SHALL BE CLASS B, UNLESS NOTED OTHERWISE.
- BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS. STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH MTO STRUCTURAL STANDARD DWG SS12-1 UNLESS INDICATED OTHERWISE.

6. CONSTRUCTION
- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO THE COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
- ALL ELEVATIONS ARE TO GEODETIC DATUM.
- THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

NOT FOR CONSTRUCTION

BROWN | CO

PREPARED FOR: **MOSES STRUCTURAL ENGINEERS**

WOOD BRIDGE DESIGN EXAMPLE
GLUED-LAMINATED DECK OPTION
GENERAL ARRANGEMENT

		FILE NAME:	JOB	DRAWING	REV
DES A.L.	JULY2017				
DRW G.D.	JULY2017		2017-075	SWB-01	0
CHK S.B.	JULY2017	SCALE AS INDICATED			

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOTES:

1. STRUCTURAL STEEL FOR PLATES SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 300W. STRUCTURAL STEEL FOR ROLLED SECTIONS SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 350W.

2. GALVANIZE ALL STEEL COMPONENTS. GALVANIZING TO COMFORM TO ASTM A123/A123M WITH A MINIMUM COATING OF 600 g/m². TOUCH UP ALL FIELD DAMAGE OF GALVANIZING WITH TWO COATS OF ZINC RICH PAINT.

3. METRIC BOLTS SHALL BE GALVANIZED ASTM A325M TYPE I. DOME HEAD BOLTS SHALL BE GALVANIZED ASTM A307 GRADE A. BOLT THREADS TO BE EXCLUDED FROM THE SHEAR PLANES.

4. BOLTS SHALL BE TIGHTENED TO A SNUG-TIGHT CONDITION.

5. MALLEABLE IRON WASHERS SHALL BE GALVANIZED ASTM A47 GRADE 32510.

6. TIE RODS SHALL BE GALVANIZED ASTM A307 GRADE A.

7. THE CONTRACTOR SHALL ENSURE THE STABILITY OF ALL COMPONENTS DURING HANDLING, TRANSPORTATION, AND ERECTION UNTIL THE TIMBER FRAMING IS IN ITS FINAL LOCATION WITH ALL PERMANENT BRACING, CONNECTIONS, AND SUPPORTS IN PLACE.

8. GIRDERS CAMBER INCLUDES AN ALLOWANCE FOR TWICE THE DEFLECTION DUE TO UNFACTORED DEAD LOADS PLUS 1/600TH THE SPAN.

9. DIAPHRAGMS HAVE NOT BEEN DESIGNED FOR USE AS JACKING POINTS.

TYPICAL DIAPHRAGM OFFSET
1:10

FIXED END
1:10

EXP. END
1:10

2-22⌀x40 SLOTS FOR M20 BOLT (TYP.)

4-22⌀ DRILLED HOLE FOR M20 BOLT

TIMBER FRAMING PLAN
1:75

TYPICAL AT ALL DIAPHRAGM LOCATIONS
1:20

25x25 ROUTED HOLE (2 HOLES FOR EXTERIOR GIRDERS 1 & 12, 4 HOLES FOR INTERIOR GIRDERS 2 TO 11) (TYP.)

TYPICAL DIAPHRAGM ELEVATION
1:20

22⌀ TIE RODS (TYP.)

MALLEABLE IRON WASHER (TYP.)

STIFFENER BEAM

DIAPHRAGM LENGTHS	
MARK	LENGTH (mm)
1	935
2	910
3	885

ELEVATION
1:50

26 @ 594 = 15445

ROUTED SLOT (SEE DETAIL)

TYPICAL GIRDER CAMBER DIAGRAM
NTS

TYPICAL GLULAM GIRDER CROSS-SECTION
1:20

NOTE:
EXTERIOR GIRDERS - 1, 2, 11, & 12
INTERIOR GIRDERS - 3 TO 10

215 (INTERIOR GIRDERS)
265 (EXTERIOR GIRDERS)
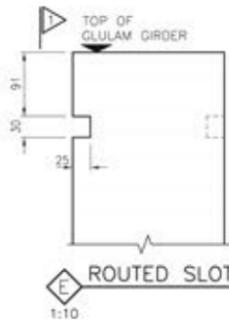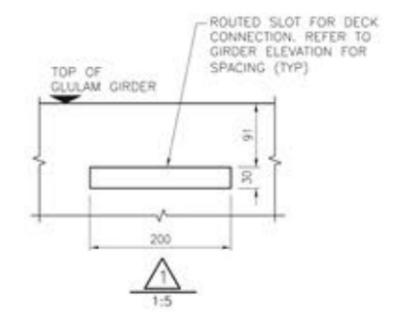
ROUTED SLOT
1:10

ROUTED SLOT FOR DECK CONNECTION. REFER TO GIRDER ELEVATION FOR SPACING (TYP)
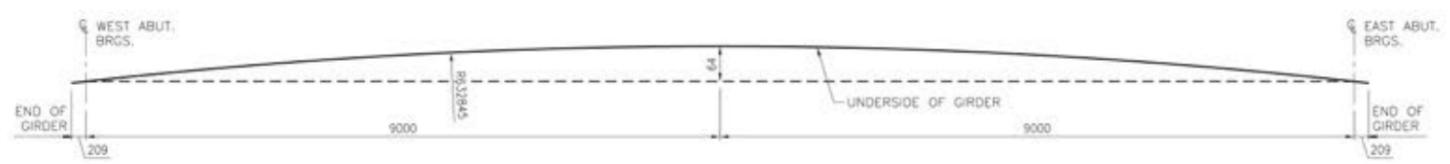
NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

BROWN | CO

CONSULTANT:

PREPARED FOR:

MOSES STRUCTURAL ENGINEERS

WOOD BRIDGE DESIGN EXAMPLE
GLUED-LAMINATED DECK OPTION
GLUED-LAMINATED TIMBER FRAMING I

		FILE NAME:	JOB	DRAWING	REV
DES	A.L.	JULY2017			
DRN	G.D.	JULY2017	2017-075	SWB-05	0
CHK	S.B.	JULY2017 SCALE: AS INDICATED			

Detail 3

- ℄ BRG./GIRDER
- 4-M20 BOLT (TYP.)
- L203x102x13 (TYP.)
- GLULAM GIRDER
- KEEPER ℞6 (NEAR SIDE)
- HOMOGENEOUS FABRIC FIBRE REINFORCED ELASTOMERIC BEARING PAD (SEE SCHEDULE)
- 38 (TYP.)
- 19∅ ANCHOR BOLT. (TYP.)
- BRG. ℞
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF PEDESTAL
- CONC. BRG. PEDESTAL
- BRG. PLATE LENGTH (SEE SCHEDULE)

Detail 5

- ℄ BRG./GIRDER
- 2-M20 BOLT (TYP.)
- L203x102x13 (TYP.)
- GLULAM GIRDER
- KEEPER ℞6 (NEAR SIDE)
- HOMOGENEOUS FABRIC FIBRE REINFORCED ELASTOMERIC BEARING PAD (SEE SCHEDULE)
- 38 (TYP.)
- 19∅ ANCHOR BOLT. (TYP.)
- BRG. ℞
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF PEDESTAL
- CONC. BRG. PEDESTAL
- BRG. PLATE LENGTH (SEE SCHEDULE)

NOTES:

1. THIS DRAWING TO BE READ IN CONJUNCTION WITH DRAWING SWB-05.

2. NON-SHRINK GROUT SHALL BE IN AS LISTED ON THE MTO D.S.M. LIST #9.15.35.

ELASTOMERIC BEARING PAD AND BEARING PLATE SCHEDULE

LOCATION	BRG PAD DIMENSIONS			BRG ℞ DIMENSIONS		
	THK	WIDTH	LENGTH	THK	WIDTH	LENGTH
GIRDERS 3 TO 10	20	200	190	10	432	240
GIRDERS 1,2,11, and 12	20	250	190	10	482	240

Detail 2

- ℄ BRG.
- 240
- 73 / 95 / 73
- L203x102
- 4-22∅ HOLES FOR M20 BOLT (TYP.)
- GLULAM GIRDER
- KEEPER ℞6 (BEYOND)
- 52 / 60 / 91 / 203
- TOP OF PEDESTAL
- BRG. ℞
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF BEARING SEAT
- 170
- 4-19∅ ANCHOR BOLT. EMBED 170mm MIN. INTO CONCRETE USING APPROVED EPOXY SYSTEM (TYP.)

Detail 4

- ℄ BRG.
- 240
- 73 / 95 / 73
- L203x102
- 2-22∅x40 SLOTS FOR M20 BOLT (TYP.)
- GLULAM GIRDER
- KEEPER ℞6 (BEYOND)
- 52 / 151 / 203
- NON-SHRINK GROUT BEDDING (TYP.)
- BRG.
- TOP OF PEDESTAL
- TOP OF BEARING SEAT
- 170
- 4-19∅ ANCHOR BOLT. EMBED 170mm MIN. INTO CONCRETE USING APPROVED EPOXY SYSTEM (TYP.)

BEARING PLATE DETAIL
1:5

- BRG. PLATE LENGTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG. ℞
- 38 (TYP.)
- M20
- 22∅ HOLES (TYP. 4 LOCATIONS)
- KEEPER ℞6x10x150
- BRG. PLATE WIDTH (SEE SCHEDULE)
- ℄ BRG. PLATE
- E.Q. / E.Q.
- 150
- 212
- 5° (TYP.)

TYPICAL GIRDER BEARING DETAIL - FIXED END
1:5 — PLAN

- 430
- BRG. PLATE LENGTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG. PAD / ℄ BRG. ℞
- 430x632 CONC. BEARING PEDESTAL
- 4-19∅ ANCHOR BOLT (TYP.)
- L203x102x13 (TYP.)
- KEEPER ℞6x10x150
- 102
- 632
- BRG. PLATE WIDTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG./GIRDER
- BEARING ℞ (SEE DETAIL AND SCHEDULE)
- ELASTOMERIC BRG. PAD (SEE SCHEDULE) CENTERED ON THE BEARING PLATE (TYP.)
- 4-M20 BOLT (TYP.)

TYPICAL GIRDER BEARING DETAIL - EXP. END
1:5 — PLAN

- 430
- BRG. PLATE LENGTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG. PAD / ℄ BRG. ℞
- 430x632 CONC. BEARING PEDESTAL
- 4-19∅ ANCHOR BOLT (TYP.)
- L203x102x13 (TYP.)
- KEEPER ℞6x10x150
- 102
- 632
- BRG. PLATE WIDTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG./GIRDER
- BEARING ℞ (SEE DETAIL AND SCHEDULE)
- ELASTOMERIC BRG. PAD (SEE SCHEDULE) CENTERED ON THE BEARING PLATE (TYP.)
- 2-M20 BOLT (TYP.)

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOTES:

1. DOME HEAD BOLTS SHALL BE GALVANIZED ASTM A307 GRADE A. BOLT THREADS TO BE EXCLUDED FROM THE SHEAR PLANES.
2. BOLTS SHALL BE TIGHTENED TO A SNUG-TIGHT CONDITION.
3. MALLEABLE IRON WASHERS SHALL BE GALVANIZED ASTM A47 GRADE 32510.
4. ALL GIRDERS AND DIAPHRAGMS ARE TO BE ERECTED, AND THEIR CONNECTIONS FASTENED, BEFORE ERECTING DECK PANELS.
5. DECK PANEL LIVE LOAD DEFLECTION HAS BEEN LIMITED TO A MAXIMUM OF 1.3 mm [0.05 in].

DECK LAYOUT PLAN
1:75

NORTH FOR CONSTRUCTION

ABUT. BRGS. (FIXED END)
ABUT. BRGS. (EXP. END)

18418
18000
209
209

GIRDER IDENTIFICATION NUMBER
GLULAM GIRDER (TYP.)
GLULAM STIFFENER BEAM (TYP.)

1482 PANEL 'A'
15454
(13 PANEL 'B' @ 1178 = 15314) + (14 GAPS @ 10 = 140)
1482 PANEL 'A'

10mm GAP (TYP.)
WP#1
WP#2
€ HWY
14110

RAILING-TO-DECK CONNECTION - PANEL 'A'
(REFER TO RAILING DWGS. FOR POST LAYOUT)
1:10

140 152 152 107 299 152 152 119
€ PANEL 'A'
BRG.
155
CUT GROOVE FOR 4"ø SPLIT RING (TYP.)
22ø HOLE FOR ⅞"ø DOME HEAD BOLT (TYP.)
GLULAM DECK PANEL 'A'

RAILING-TO-DECK CONNECTION - PANEL 'B'
(REFER TO RAILING DWGS. FOR POST LAYOUT)
1:10

152 152 203 203 152 152
€ PANEL 'B'
155
CUT GROOVE FOR 4"ø SPLIT RING (TYP.)
22ø HOLE FOR ⅞"ø DOME HEAD BOLT (TYP.)
GLULAM DECK PANEL 'B'

TYPICAL DECK-TO-STIFFENER BEAM CONNECTION
1:10

STIFFENER BEAM
GLULAM DECK PANEL (TYP.)
⅞"ø DOME HEAD BOLT (TYP.)
GLULAM STIFFENER BEAM
WASHER ₽12.7x140x140 AND NUT
114
215

STIFFENER BEAM
GLULAM STIFFENER BEAM
22ø x40 LONG SLOTTED HOLE (TYP.)
1:10

TYPCAL DECK PANEL CROSS SECTIONS
1:20

1482
215
PANEL 'A'

1178
215
PANEL 'B'
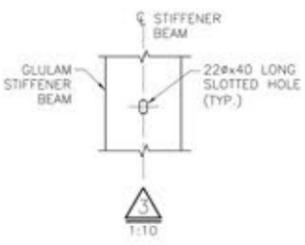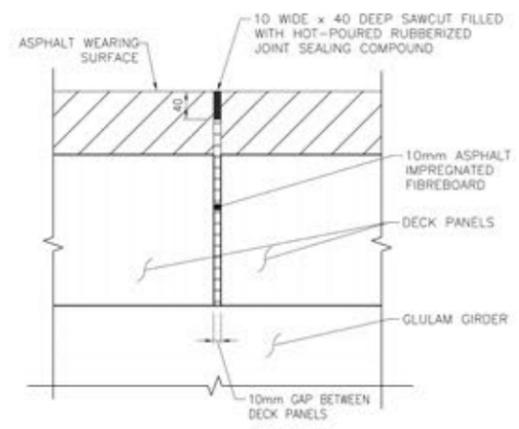
TYPCAL JOINT DETAIL BETWEEN DECK PANELS
1:5

10 WIDE x 40 DEEP SAWCUT FILLED WITH HOT-POURED RUBBERIZED JOINT SEALING COMPOUND
ASPHALT WEARING SURFACE
10mm ASPHALT IMPREGNATED FIBREBOARD
DECK PANELS
GLULAM GIRDER
10mm GAP BETWEEN DECK PANELS

DECK PANEL CONNECTION
1:20

GLULAM DECK PANEL (TYP.)
DECK BRACKET (TYP.)
ROUTED SLOT. (REFER TO GIRDER DWGS. FOR DETAILS) (TYP.)
GLULAM GIRDER
1:20

⅞"ø DOME HEAD BOLT + 76ø WASHER (TYP.)
LAG SCREW
GLULAM DECK PANEL (TYP.)
GLULAM GIRDER

DECK BRACKET DETAIL
(MATERIAL: CAST ALUMINUM ALLOY)
1:5

38
21ø HOLE (TYP.)
21
19
19
19
66
25 (TYP.)
110
125

DECK-TO-GIRDER CONNECTION PANEL 'A'
1:20

8 LAG SCREW (¾"ø x380 Lg) @ EQUAL SPACING WITH 150 Lg THREADS, 80mm EDGE DISTANCE AND WASHERS (TYP.)
18ø HOLE FOR ⅝"ø DOME HEAD BOLT (TYP.)
GLULAM GIRDER (TYP.)
ROUTED SLOT IN GLULAM GIRDER (TYP.)
GLULAM DECK PANEL (TYP.)
€ GIRDER
38 (TYP.)
144 1194 144

DECK-TO-GIRDER CONNECTION PANEL 'B'
1:20

GLULAM GIRDER (TYP.)
€ GIRDER
38 (TYP.)
143 297 297 297 144

DECK-TO-STIFFENER BEAM CONNECTION
1:20

3 EQUAL SPACES (TOTAL = 1282)
3 SPACES @ 326 (TOTAL = 978)
100
100
100
10 GAP
22ø HOLE FOR ⅞"ø DOME HEAD BOLT (TYP.)
€ STIFFENER BEAM
GLULAM DECK PANEL (TYP.)
GLULAM STIFFENER BEAM (TYP.)
END OF DECK
PANEL 'A'
PANEL 'B'
PANEL 'B'
10 GAP
10 GAP (TYP.)

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

ANCHORAGE FOR APPROACH GUIDERAIL TRANSITION. (TYP. AT ENDS OF TIMBER BARRIERS.)

℄ WEST ABUT. BRGS.
℄ EAST ABUT. BRGS.

5500
1271 2438 2438
647
2413
18000
5 EQUAL SPACES @ 2376 = 11880
2413
647
2438 2438 1271
5500

GLULAM CURB (TYP.)
GLULAM RAIL (TYP.)
GLULAM POST (TYP.)

GLULAM DECK PANEL (TYP.)
GLULAM SCUPPER (TYP.)

ELEVATION
(EXTERIOR FACE SHOWN)
1:40

PARTIAL ELEVATION – CONCRETE WINGWALL ANCHORAGE
(EXTERIOR FACE SHOWN)
1:10

WASHER ℝ6x152x152 W/ 35ø CENTERED
SYMMETRICAL ABOUT ℄
6-3/4"ø DOME HEAD BOLTS & NUTS W/ MALLEABLE IRON WASHERS (TYP.)
POST ANGLE
4"ø SPLIT RINGS OR BACK TO BACK SHEAR PLATES. (TYP)
TOP OF CONCRETE WINGWALL
152 152 203
162 162
1¼"ø DOME HEAD BOLT (TYP)

PARTIAL ELEVATION – CONCRETE WINGWALL ANCHORAGE
(INTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ℄
2-DOME HEAD BOLTS & NUTS, ¾"ø, W/ MALLEABLE IRON WASHER (TYP.)
152
102
WASHER ℝ6x152x152 W/ 35ø CENTERED
1¼"ø DOME HEAD BOLT & NUT (TYP.)

PARTIAL ELEVATION – GLULAM DECK ANCHORAGE
(EXTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ℄
WASHER ℝ6x152x152 W/ 35ø HOLE CENTERED
6-3/4"ø DOME HEAD BOLTS & NUTS W/ MALLEABLE IRON WASHERS (TYP.)
POST ANGLE
4"ø SPLIT RINGS OR BACK TO BACK SHEAR PLATES. (TYP)
162 162
152 152 203

PARTIAL ELEVATION – GLULAM DECK ANCHORAGE
(INTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ℄
2-DOME HEAD BOLTS & NUTS, ¾"ø, W/ MALLEABLE IRON WASHER (TYP.)
152
102
WASHER ℝ6x152x152 W/ 35ø CENTERED
1¼"ø DOME HEAD BOLT & NUT (TYP.)

(TIMBER BARRIER ON CONCRETE WINGWALL) ⟨2⟩
1:10

571
267 79 225
12
79x266x225 GLULAM BLOCK
GLULAM RAIL
225x267x1027 GLULAM POST
1015
152
102
343
305
E.Q. E.Q.
175x305 GLULAM CURB RAIL
175
E.Q. E.Q.
889
175
16ø x152 Lg. LAG SCREW W/ WASHER
POST ANGLE
120
600
300
DRILL AND BOND 1¼"ø BOLT IN 35ø HOLE USING APPROVED EPOXY GROUT FROM DSM
DRILL AND BOND ¾"ø BOLT IN 22ø HOLES USING APPROVED EPOXY GROUT FROM DSM
CONCRETE WINGWALL

(TIMBER BARRIER ON GLULAM DECK) ⟨1⟩
1:10

571
267 79 225
12
79x266x225 GLULAM BLOCK
GLULAM RAIL
225x267x1027 GLULAM POST
1015
152
102
343
305
E.Q. E.Q.
175x305 GLULAM CURB RAIL
175
E.Q. E.Q.
889
175
POST ANGLE
WEARING SURFACE
16ø x152 Lg. LAG SCREW W/ WASHER
38x89 SAWN WOOD RETAINER BLOCK (BEYOND)
175x305x1372 GLULAM SCUPPER
215 THK TRANSVERSE GLULAM DECK

POST ANGLE DETAIL
1:5

L102x102x9.5
127
19ø x38 Lg. SLOTTED HOLE
52
6 HOLE

NOTES:

1. STRUCTURAL STEEL FOR PLATES SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 300W. STRUCTURAL STEEL FOR ROLLED SECTIONS SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 350W.

2. GALVANIZE ALL STEEL COMPONENTS. GALVANIZING TO COMFORM TO ASTM A123/A123M WITH A MINIMUM COATING OF 600 g/m2. TOUCH UP ALL DAMAGED OR REMOVED GALVANIZING WITH TWO COATS OF ZINC RICH PAINT.

3. ALL WELDING TO CONFORM TO CAN/CSA-W59.1 AND IS TO BE UNDERTAKEN BY A FABRICATOR QUALIFIED IN ACCORDANCE WITH CAN/CSA-W47.1.

4. DOME HEAD BOLTS SHALL BE GALVANIZED ASTM A307 GRADE A. BOLT THREADS TO BE EXCLUDED FROM THE SHEAR PLANES.

5. BOLTS SHALL BE TIGHTENED TO A SNUG-TIGHT CONDITION.

6. MALLEABLE IRON WASHERS SHALL BE GALVANIZED ASTM A47 GRADE 32510.

7. SPLIT RINGS SHALL BE GALVANIZED.

8. LAG SCREWS SHALL BE GALVANIZED AND CONFORM TO CAN/CSA-B34.

9. TIMBER RAILINGS TO BE PRE-ASSEMBLED IN THE SHOP WITH DECK PANELS TO ENSURE PROPER FIT-UP.

NOT FOR CONSTRUCTION

| REV	DATE	DESCRIPTION	BY

CONSULTANT:

BROWN | CO

PREPARED FOR:

MOSES STRUCTURAL ENGINEERS

WOOD BRIDGE DESIGN EXAMPLE
GLUED-LAMINATED DECK OPTION
**GLUED-LAMINATED TIMBER BARRIER I**

		FILE NAME:	JOB	DRAWING	REV
DES	A.L.	JULY2017			
DRW	G.D.	JULY2017	2017-075	**SWB-08**	0
CHK	S.B.	JULY2017	SCALE: AS INDICATED		

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOTES:

1. THIS DRAWING TO BE READ IN CONJUNCTION WITH DRAWING SWB-08.

SYMMETRICAL ABOUT ℄

SPLICE ℄10 (TYP. EACH SIDE OF RAIL)

UPPER RAIL

127 127 51

191 165 89

POST

356

1 1:10

SYMMETRICAL ABOUT ℄

SPLICE ℄10x342x712

4-22Ø HOLES
8-34Ø HOLES

185 102 89 343

127 127 51

712

STEEL SPLICE PLATE

SYMMETRICAL ABOUT ℄

4-DOME HEAD BOLTS & NUTS, ¾"Ø, W/ MALLEABLE IRON WASHER

POST

8-1¼" DOME HEAD BOLT & NUT W/ WASHER

SPACER

UPPER RAIL

ROUTE RAIL BOTH SIDES 10mm TO RECESS SPLICE PLATES (TYP.)

6mm GAP (MAX.)

1

PLAN

GLULAM RAIL SPLICE
1:10

712

℄10x171x712

285

6-17Ø (TYP.)

℄10x171x285

SPLICE CONNECTOR ASSEMBLY

12-DOME HEAD BOLTS & NUTS, ¾"Ø, W/ MALLEABLE IRON WASHERS (TYP.)

SYMMETRICAL ABOUT ℄

CURB SPLICE CONNECTOR

CURB RAIL

ROUTE RAIL BOTH SIDES 10mm TO RECESS SPLICE PLATES (TYP.)

PLAN

2

GLULAM CURB SPLICE
1:10

SYMMETRICAL ABOUT ℄

12-21Ø HOLES

171 48 75 48

51 76 76

712

2 1:10

NOT FOR CONSTRUCTION

NOTE:
ALL DIMENSION SHOWN ARE IN METRES AND/OR MILLIMÉTRES UNLESS NOTED OTHERWISE.

NORTH FOR CONSTRUCTION

PLAN
1:75

Labels in PLAN:
- 5500
- ₵ WEST ABUT. BRGS.
- 18000
- ₵ EAST ABUT. BRGS.
- 5500
- GLULAM RAILING (TYP.)
- ₵ DIAPHRAGM
- GLULAM RAILING POST (TYP.)
- ₵ DIAPHRAGM
- MC230 (TYP.)
- ₵ DIAPHRAGM
- GLULAM GIRDER (TYP.)
- ₵ DIAPHRAGM
- LIMIT OF STRESS-LAMINATED DECK
- GLULAM CURB
- 6000 APPROACH SLAB
- GLULAM DIAPHRAGM (TYP.)
- 6000 APPROACH SLAB
- ₵ HIGHWAY
- WP#1 STA. 16+653 T/P EL. 265.125
- WP#2 STA. 16+671 T/P EL. 265.395
- ASPHALT WEARING SURFACE
- MC230 (TYP.)
- 235mm DEEP STRESS-LAMINATED DECK (TYP.)
- 18ø HIGH STRENGTH POST-TENSIONING BAR (TYP.)
- GLULAM FLOOR BEAM. 365mm WIDE x 380mm DEEP @ 3000 O.C (TYP.)
- CONC. WINGWALL (TYP.)

ELEVATION
1:75

Labels in ELEVATION:
- WEST / EAST
- 267, 266, 265, 264, 263, 262, 261, 260
- ₵ WEST ABUT. BRGS.
- 5500 / 18000 / 5500
- ₵ EAST ABUT. BRGS.
- GLULAM RAILING (TYP.)
- 18ø HIGH STRENGTH PORT-TENSIONING BAR (TYP.)
- ANCHOR ₵ (TYP.)
- CONC. WINGWALL (TYP.)
- STRESS-LAMINATED DECK (TYP.)
- GLULAM SCUPPER (TYP.)
- GLULAM CURB (TYP.)
- ANCHORAGE FOR APPROACH GUIDERAIL TRANSITION. (TYP. AT ENDS OF TIMBER BARRIERS)
- 1.50%
- EXISTING GROUND ALONG ₵ OF HIGHWAY
- FIX. 1000 (TYP.)
- ROCK PROTECTION (TYP.)
- GLULAM GIRDER (TYP.)
- GLULAM FLOOR BEAM (TYP.)
- GLULAM DIAPHRAGM (TYP.)
- EXP.
- 6000mm APPROACH SLAB (TYP)
- W.L. EL. 261.24 (JULY 14, 2017)
- NON-NAVIGABLE CREEK
- T/FTG. EL. 261.26
- T/FTG. EL. 260.99

PROFILE OF HWY
N.T.S.

- WEST / EAST
- STA. 16+500 EL. 262.850
- ₵ CREEK
- STA. 16+810 EL. 267.460
- PROFILE CONTROL (TYP.)
- +1.50%
- LIMIT OF STRUCTURE

SECTION
1:75

- NORTH / SOUTH
- 14110
- ₵ HIGHWAY
- 3000 SHOULDER / 3750 LANE / 3750 LANE / 3000 SHOULDER
- PROFILE CONTROL
- -2% / -2%
- CRASH TESTED TL-4 GLULAM BARRIER (TYP)
- ASPHALT WEARING SURFACE (50MIN., 185 MAX.)
- 38x235 DEEP STRESS-LAMINATED DECK
- MC230x37.8 CONTINUOUS GALVANIZED CHANNEL BULKHEAD (TYP.)
- 673 BARRIER
- 673 BARRIER
- GLULAM DIAPHRAGM (TYP.)
- 365mm WIDE x 380mm DEEP FLOOR BEAM (TYP.)
- 18ø HIGH STRENGTH POST-TENSIONING BAR IN 25ø PRE-DRILLED HOLES (TYP.)
- 265x1634 GLULAM GIRDER (TYP. FOR EXTERIOR PAIRS OF GIRDERS)
- 215x1634 GLULAM GIRDER (TYP. FOR INTERIOR GIRDERS)
- 730 / 11 SPACES @ 1150 = 12650 / 730

LIST OF ABBREVIATIONS
- WP — DENOTES WORK POINT
- T/D — DENOTES TOP OF DECK
- T/P — DENOTES TOP OF PAVEMENT
- T/FTG. — DENOTES TOP OF FOOTING
- ABUT. — DENOTES ABUTMENT
- BRGS. — DENOTES BEARINGS
- C.J. — DENOTES CONSTRUCTION JOINT
- CONC. — DENOTES CONCRETE
- FIX. — DENOTES FIXED BEARINGS
- EXP. — DENOTES EXPANSION BEARINGS
- TYP. — DENOTES TYPICAL
- EXIST. — DENOTES EXISTING
- EL. — DENOTES ELEVATION
- MAX. — DENOTES MAXIMUM
- MIN. — DENOTES MINIMUM

APPLICABLE STANDARD DRAWINGS
- OPSD-3101.150 WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT.
- OPSD-3102.100 WALLS, ABUTMENT, BACKFILL, DRAIN.
- OPSD-3370.100 DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD.
- OPSD-3941.200 FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT.
- OPSD-3190.100 WALLS RETAINING AND ABUTMENT WALL DRAIN.

GENERAL NOTES:

1. **DESIGN CODE**
   - CAN/CSA-S6-14 (CANADIAN HIGHWAY BRIDGE DESIGN CODE)
   - LL: CL-625-ONT

2. **TIMBER**
   - ALL LUMBER FOR STRESS-LAMINATED DECK SHALL BE LODGEPOLE PINE SELECT STRUCTURAL GRADE.
   - ALL GLUED-LAMINATED TIMBER SHALL BE S-P-F MANUFACTURED IN ACCORDANCE WITH CAN/CSA-O122 BY A PLANT CERTIFIED IN ACCORDANCE WITH CAN/CSA-O177.
   - GLUED-LAMINATED STRESS GRADES SHALL BE AS FOLLOWS:
     - GIRDERS 20f-E
     - DIAPHRAGMS 20f-EX
     - BARRIERS 20f-EX
   - ALL GLUED-LAMINATED TIMBER SHALL HAVE A QUALITY APPEARANCE GRADE.
   - ALL SPECIFIED WOOD SIZES ARE DRESSED SIZES.
   - ALL WOOD SHALL BE PRESSURE PRESERVATIVE TREATED IN ACCORDANCE WITH CAN/CSA-O80 AND AWPA STANDARDS M1, M2, AND M4.
   - ALL TREATED WOOD SHALL BE CUT TO FINISHED SIZE. ALL SURFACING, HOLES, AND OTHER CUTS SHALL BE COMPLETED PRIOR TO PRESSURE PRESERVATIVE TREATMENT.
   - NO CHAINS, HOOKS, OR PEAVES SHALL BE USED IN HANDLING TREATED WOOD. SURFACES EXPOSED BY FIELD CUTTING OR DRILLING SHALL BE TREATED BY SOAKING WITH AN APPROVED PRESERVATIVE EQUAL IN TOXICITY TO THE ORIGINAL PRESERVATIVE. THREE APPLICATIONS ARE REQUIRED, EACH TO AIR DRIED SURFACES.

3. **CLASS OF CONCRETE**
   - ALL CONCRETE 30 MPa

4. **CLEAR COVER TO REINFORCING STEEL**
   - FOOTINGS 100 ± 25mm
   - REMAINDER 70 ± 20mm

5. **REINFORCING STEEL**
   - BAR MARKS WITH PREFIX "S" DENOTES STAINLESS STEEL BARS.
   - STAINLESS REINFORCING STEEL SHALL BE TYPE 316 LN OR TYPE 2205 DUPLEX AND HAVE A MINIMUM YIELD STRENGTH OF 500MPa.
   - NON-STAINLESS REINFORCING STEEL SHALL BE GRADE 400W UNLESS NOTED OTHERWISE.
   - TENSION LAP LENGTHS SHALL BE CLASS B, UNLESS NOTED OTHERWISE.
   - BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS. STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH MTO STRUCTURAL STANDARD DWG SS12-1 UNLESS INDICATED OTHERWISE.

6. **CONSTRUCTION**
   - THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO THE COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
   - ALL ELEVATIONS ARE TO GEODETIC DATUM.
   - THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

BROWN | CO

PREPARED FOR:
MOSES STRUCTURAL ENGINEERS

WOOD BRIDGE DESIGN EXAMPLE
STRESS-LAMINATED DECK OPTION
GENERAL ARRANGEMENT

DES	A.L.	NOV2017		JOB	DRAWING	REV
DRW	G.D.	NOV2017		2017-075	SWA-01	0
CHK	S.B.	NOV2017	SCALE: AS INDICATED			

REV / DATE / DESCRIPTION / BY

NOTES:

1. STRUCTURAL STEEL FOR PLATES SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 300W. STRUCTURAL STEEL FOR ROLLED SECTIONS SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 350W.

2. GALVANIZE ALL STEEL COMPONENTS. GALVANIZING TO COMFORM TO ASTM A123/A123M WITH A MINIMUM COATING OF 600 g/m2. TOUCH UP ALL FIELD DAMAGE OF GALVANIZING WITH TWO COATS OF ZINC RICH PAINT.

3. METRIC BOLTS SHALL BE GALVANIZED ASTM A325M TYPE I, DOME HEAD BOLTS SHALL BE GALVANIZED ASTM A307 GRADE A. BOLT THREADS TO BE EXCLUDED FROM THE SHEAR PLANES.

4. BOLTS SHALL BE TIGHTENED TO A SNUG-TIGHT CONDITION.

5. MALLEABLE IRON WASHERS SHALL BE GALVANIZED ASTM A47 GRADE 32510.

6. TIE RODS SHALL BE GALVANIZED ASTM A307 GRADE A.

7. THE CONTRACTOR SHALL ENSURE THE STABILITY OF ALL COMPONENTS DURING HANDLING, TRANSPORTATION, AND ERECTION UNTIL THE TIMBER FRAMING IS IN ITS FINAL LOCATION WITH ALL PERMANENT BRACING, CONNECTIONS, AND SUPPORTS IN PLACE.

8. GIRDERS CAMBER INCLUDES AN ALLOWANCE FOR TWICE THE DEFLECTION DUE TO UNFACTORED DEAD LOADS PLUS 1/600TH THE SPAN.

9. DIAPHRAGMS HAVE NOT BEEN DESIGNED FOR USE AS JACKING POINTS.

TIMBER FRAMING PLAN 1:75

ELEVATION 1:50

TYPICAL GIRDER CAMBER DIAGRAM NTS

A TYPICAL DIAPHRAGM OFFSET 1:10

B FIXED END 1:10

C EXP. END 1:10

F STRESS-LAMINATED DECK TO GLULAM FLOOR BEAM CONNECTION 1:20
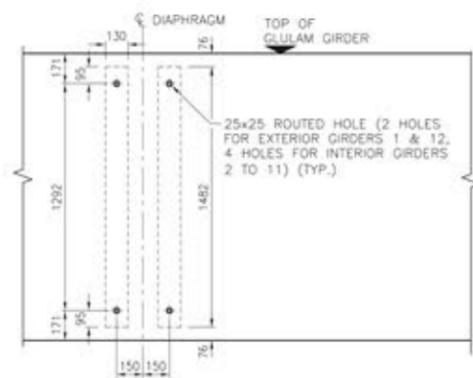
D TYPICAL TO ALL DIAPHRAGM LOCATIONS 1:20
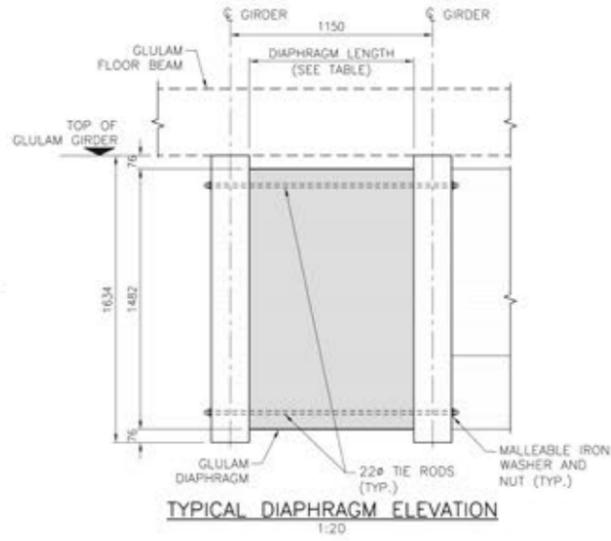
TYPICAL DIAPHRAGM ELEVATION 1:20
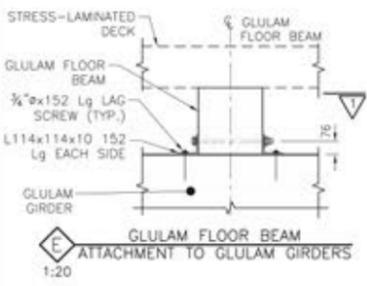
E GLULAM FLOOR BEAM ATTACHMENT TO GLULAM GIRDERS 1:20

1 1:10
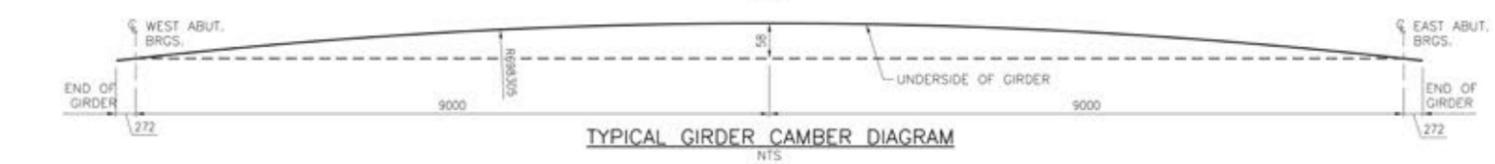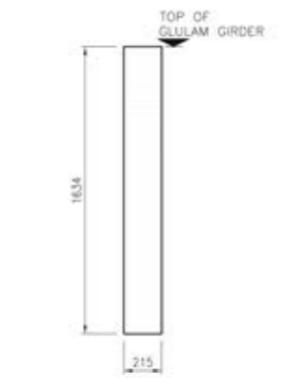
TYPICAL GLULAM INTERIOR GIRDER CROSS-SECTION (GIRDERS 3 TO 10) 1:20
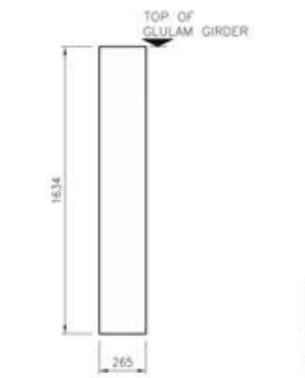
TYPICAL GLULAM EXTERIOR GIRDER CROSS-SECTION (GIRDERS 1,2,11, and 12) 1:20

DIAPHRAGM LENGTHS

MARK	LENGTH (mm)
1	935
2	910
3	885

NOT FOR CONSTRUCTION

BROWN | CO

MOSES STRUCTURAL ENGINEERS

WOOD BRIDGE DESIGN EXAMPLE
STRESS-LAMINATED DECK OPTION
GLUED-LAMINATED TIMBER FRAMING I

DES	A.L.	NOV2017	JOB	DRAWING	REV.
DRW	G.D.	NOV2017	2017-075	SWA-05	0
CHK	S.B.	NOV2017			

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

ELASTOMERIC BEARING PAD AND BEARING PLATE SCHEDULE

LOCATION	BRG PAD DIMENSIONS			BRG ℄ DIMENSIONS		
	THK	WIDTH	LENGTH	THK	WIDTH	LENGTH
GIRDERS 3 TO 10	20	200	190	10	432	240
GIRDERS 1,2,11, and 12	20	250	190	10	482	240

Detail 3:
- ℄ BRG./GIRDER
- 4-M20 BOLT (TYP.)
- L203x102x13 (TYP.)
- GLULAM GIRDER
- KEEPER ℝ6 (NEAR SIDE)
- HOMOGENEOUS FABRIC FIBRE REINFORCED ELASTOMERIC BEARING PAD (SEE SCHEDULE)
- 19∅ ANCHOR BOLT. (TYP.)
- 38 (TYP.)
- BRG. ℄
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF PEDESTAL
- CONC. BRG. PEDESTAL
- BRG. PLATE LENGTH (SEE SCHEDULE)

Detail 5:
- ℄ BRG./GIRDER
- 2-M20 BOLT (TYP.)
- L203x102x13 (TYP.)
- GLULAM GIRDER
- KEEPER ℝ6 (NEAR SIDE)
- HOMOGENEOUS FABRIC FIBRE REINFORCED ELASTOMERIC BEARING PAD (SEE SCHEDULE)
- 19∅ ANCHOR BOLT. (TYP.)
- 38 (TYP.)
- BRG. ℄
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF PEDESTAL
- CONC. BRG. PEDESTAL
- BRG. PLATE LENGTH (SEE SCHEDULE)

Detail 2:
- ℄ BRG.
- 240 / 73 / 95 / 73
- L203x102
- 4-22∅ HOLES FOR M20 BOLT (TYP.)
- GLULAM GIRDER
- KEEPER ℝ6 (BEYOND)
- 52 / 60 / 203 / 91
- BRG. ℄
- TOP OF PEDESTAL
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF BEARING SEAT
- 170
- 4-19∅ ANCHOR BOLT. EMBED 170mm MIN. INTO CONCRETE USING APPROVED EPOXY SYSTEM (TYP.)

Detail 4:
- ℄ BRG.
- 240 / 73 / 95 / 73
- L203x102
- 2-22∅x40 SLOTS FOR M20 BOLT (TYP.)
- GLULAM GIRDER
- KEEPER ℝ6 (BEYOND)
- 52 / 151 / 203
- BRG. ℄
- NON-SHRINK GROUT BEDDING (TYP.)
- TOP OF BEARING SEAT
- TOP OF PEDESTAL
- 170
- 4-19∅ ANCHOR BOLT. EMBED 170mm MIN. INTO CONCRETE USING APPROVED EPOXY SYSTEM (TYP.)

BEARING PLATE DETAIL 1:5
- BRG. PLATE LENGTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG. ℝ
- 38 TYP.
- BRG. PLATE WIDTH (SEE SCHEDULE)
- 22∅ HOLES (TYP. 4 LOCATIONS)
- KEEPER ℝ6x10x150
- ℄ BRG. PLATE
- E.Q. / E.Q.
- 150
- 212

PLAN — TYPICAL GIRDER BEARING DETAIL — FIXED END 1:5
- 430
- BRG. PLATE LENGTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG. PAD / ℄ BRG. ℝ
- 430x632 CONC. BEARING PEDESTAL
- 4-19∅ ANCHOR BOLT (TYP.)
- L203x102x13 (TYP.)
- KEEPER ℝ6x10x150
- ℄ BRG./GIRDER
- BEARING ℝ (SEE DETAIL AND SCHEDULE)
- ELASTOMERIC BRG. PAD (SEE SCHEDULE) CENTERED ON THE BRG. PLATE (TYP.)
- 4-M20 BOLT (TYP.)
- 102
- 632
- BRG. PLATE WIDTH (SEE SCHEDULE)

PLAN — TYPICAL GIRDER BEARING DETAIL — EXP. END 1:5
- 430
- BRG. PLATE LENGTH (SEE SCHEDULE)
- E.Q. / E.Q.
- ℄ BRG. PAD / ℄ BRG. ℝ
- 430x632 CONC. BEARING PEDESTAL
- 4-19∅ ANCHOR BOLT (TYP.)
- L203x102x13 (TYP.)
- KEEPER ℝ6x10x150
- ℄ BRG./GIRDER
- BEARING ℝ (SEE DETAIL AND SCHEDULE)
- ELASTOMERIC BRG. PAD (SEE SCHEDULE) CENTERED ON THE BEARING PLATE (TYP.)
- 2-M20 BOLT (TYP.)
- 102
- 632
- BRG. PLATE WIDTH (SEE SCHEDULE)

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

DECK LAYOUT PLAN
1:75

NORTH FOR CONSTRUCTION

¢ WEST ABUT. BRGS.
¢ EAST ABUT. BRGS.

18544
18000
21 SPACES AT 840 = 17640 (POST-TENSIONING BARS)
272 / 180 / 180 / 272

MC230 (TYP.)
18ø HIGH STRENGTH POST-TENSIONING BAR IN 25ø PRE-DRILLED HOLES (TYP.)
FACE OF CURB
16ø DOME HEAD BOLTS (TYP.)
L127x89x13 LSV
WP#1
¢ HWY
DIRECTION OF TRAFFIC
WP#2
L127x89x13 LSV
FACE OF CURB
235mm DEEP STRESS-LAMINATED DECK (TYP.)
MC230 (TYP.)

6750
6750
14110
305
305

SECTION (2)
1:5

16ø DOME HEAD BOLT (TYP.) @ 500 O.C.
6mm DEEP RECESS FOR BOLT HEAD
RECESS DECK END AS REQUIRED
L127x89x13 LSV ARMOURING ANGLE
LAG SCREW @ 500 O.C.
60 / 25
60
CONCRETE BALLAST WALL
STRESS-LAMINATED DECK

NOTE:
ALL DIMENSION SHOWN ARE IN METRES AND/OR MILLIMETRES UNLESS NOTED OTHERWISE.

NOTES:
1. ALL HARDWARE, FASTENERS, AND STRUCTURAL STEEL SHALL BE HOT-DIP GALVANIZED IN ACCORDANCE WITH ASTM A123/A123M. DAMAGED GALVANIZING SHALL BE REPAIRED USING TWO COATS OF ZINC-RICH PAINT.
2. DOME HEAD BOLTS SHALL BE GALVANIZED ASTM A307 GRADE A. BOLT THREADS TO BE EXCLUDED FROM SHEAR PLANES.
3. BOLTS SHALL BE TIGHTENED TO A SNUG-TIGHT CONDITION.
4. MALLEABLE IRON WASHERS SHALL BE GALVANIZED ASTM A47 GRADE 32510.
5. ALL GIRDERS AND DIAPHRAGMS ARE TO BE ERECTED, AND THEIR CONNECTIONS FASTENED, BEFORE CONSTRUCTING THE STRESS-LAMINATED DECK.
6. DECK LIVE LOAD DEFLECTION HAS BEEN LIMITED TO A MAXIMUM OF 7.5mm.
7. POST-TENSIONING STEEL SHALL BE HIGH-STRENGTH BARS IN ACCORDANCE WITH CSA G279, GRADE 150.
8. POST-TENSIONING ANCHORAGES SHALL ABLE TO WITHSTAND UP TO SIX RESTRESSING OPERATIONS WITHOUT LOSS OF CAPACITY OR EFFECTIVENESS.
9. THE POST-TENSIONING FORCE IN EACH BAR SHALL BE 180 kN AT JACKING, WITH AN ALLOWABLE VARIATION OF +/- 5%.
10. THE POST-TENSIONING BARS SHALL BE STRESSED USING HYDRAULIC JACKS ACCORDING THE FOLLOWING SEQUENCE:
   a. INITIAL STRESSING - THE INITIAL STRESSING SHALL CONSIST OF TWO STRESSING OPERATIONS NOT LESS THAN 12 HOURS APART
   b. FIRST RESTRESSING - THE FIRST RESTRESSING SHALL CONSIST OF A SINGLE STRESSING OPERATION NOT LESS THAN TWO WEEKS AFTER THE INITIAL STRESSING.
   c. SECOND RESTRESSING - THE SECOND RESTRESSING SHALL CONSIST OF A SINGLE STRESSING OPERATION NOT LESS THAN FOUR WEEKS AFTER THE INITIAL STRESSING IS COMPLETE.
11. THE DECK SHALL NOT BE FASTENED TO THE FLOOR BEAMS UNTIL AFTER THE FIRST RESTRESSING OPERATION IS COMPLETE.
12. THE DECK IS TO BE RESTRAINED AGAINST BUCKLING AT ITS MID-WIDTH DURING ALL STRESSING OPERATIONS. THE FACTORED RESTRAINT FORCE IS EQUAL TO 4.6 kN/m.

TYPICAL BUTT JOINT LAYOUT
N.T.S.

400 MIN. / 800 MAX.
1000 (TYP.)
BUTT JOINT (1 IN 4 FREQUENCY)
END OF LAMINATES
TIMBER LAMINATES
5 GAP AT JOINT (TYP.)

TYPICAL NAILING PATTERN
N.T.S.

25ø HOLES FOR POST-TENSIONING BAR
500 TYP. / 500 MAX.
BUTT JOINT
100mm LONG NAILS (TYP.)
TIMBER LAMINATES
100

SECTION (1)
1:25

GLULAM POST
GLULAM RAIL (TYP.)
¢ DECK
GLULAM CURB RAIL (TYP.)
WEARING SURFACE
GLULAM SCUPPER (TYP.)
6750 / 6750
38x235 DEEP STRESS-LAMINATED DECK
1¼"ø DOME HEAD BOLT & NUT (TYP.)
ANCHORAGE ℞28x110x220
18ø HIGH STRENGTH POST-TENSIONING BAR IN 25ø PRE-DRILLED HOLES. COAT WITH ANTI-CORROSION PASTE (TYP.)
PROTECTIVE PLASTIC CAP PACKED WITH ANTI-CORROSION PASTE
ANCHORAGE NUT
3/4"ø DOME HEAD BOLT & NUT W/ MALLEABLE IRON WASHERS (TYP.)
MC230x37.8 CONTINUOUS GALVANIZED CHANNEL BULKHEAD (TYP.)
14110
110

ELEVATION (3)
1:10

GLULAM POST
¢ GLULAM POST
WASHER ℞6x152x152 W/ 35ø HOLE CENTERED
1¼"ø DOME HEAD BOLT & NUT (TYP.)
GLULAM RAIL
GLULAM CURB
103 / 152 / 152 / 203 / 203 / 152 / 152 / 103
POST ANGLE WITH LAG SCREW (SEE DETAIL ON DWG. SWA-09)
GLULAM SCUPPER (1219 Lg)
STRESS-LAMINATED DECK
6-3/4"ø DOME HEAD BOLTS & NUTS W/ MALLEABLE IRON WASHERS (TYP.)
103x165x203 GLULAM BLOCK (BEHIND)
18ø POST TENSIONING BAR
MC230
ANCHORAGE ℞28 (TYP)
ANCHORAGE NUT (TYP)
18ø POST TENSIONING BAR
420 / 420
840

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

BROWN | CO

PREPARED FOR:
MOSES STRUCTURAL ENGINEERS

WOOD BRIDGE DESIGN EXAMPLE
STRESS-LAMINATED DECK OPTION
STRESS-LAMINATED TIMBER DECK

|					
DES	A.L.	NOV2017	FILE NAME:	JOB	DRAWING	REV
DRW	G.D.	NOV2017		2017-075	SWA-07	0
CHK	S.B.	NOV2017	SCALE: AS INDICATED		

REV / DATE / DESCRIPTION / BY

ELEVATION
(EXTERIOR FACE SHOWN)
1:40

ANCHORAGE FOR APPROACH GUIDERAIL TRANSITION. (TYP. AT ENDS OF TIMBER BARRIERS)

₵ WEST ABUT. BRGS.

₵ EAST ABUT. BRGS.

18000
5500
1060
600
16 EQUAL SPACES @ 1680 = 26880 (GLULAM POSTS)
5500
1060
600

GLULAM CURB (TYP.)
GLULAM RAIL (TYP.)
GLULAM POST (TYP.)
STRESS-LAMINATED DECK (TYP.)
GLULAM SCUPPER (TYP.)

PARTIAL ELEVATION – CONCRETE WINGWALL ANCHORAGE
(EXTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ₵
WASHER ℞6x152x152 W/ 35ø CENTERED
6-3/4"ø DOME HEAD BOLTS & NUTS W/ MALLEABLE IRON WASHERS (TYP.)
POST ANGLE
4"ø SPLIT RINGS OR BACK TO BACK SHEAR PLATES. (TYP)
TOP OF CONCRETE WINGWALL
151 151
152 152 203
GLULAM BLOCK (BEHIND)
1¼"ø DOME HEAD BOLT (TYP)

PARTIAL ELEVATION – CONCRETE WINGWALL ANCHORAGE
(INTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ₵
2-DOME HEAD BOLTS & NUTS, ¾"ø, W/ MALLEABLE IRON WASHER (TYP.)
89 165
WASHER ℞6x152x152 W/ 35ø CENTERED
1¼"ø DOME HEAD BOLT & NUT (TYP.)

(TIMBER BARRIER ON CONCRETE WINGWALL)
2
1:10

660
254 235 171
13
89 165 343
203x254x1124 GLULAM POST
235x343x203 GLULAM BLOCK
GLULAM RAIL
305
E.Q. E.Q.
152x305 GLULAM CURB RAIL (CONT.)
152 203
16ø280 Lg. LAG SCREW W/ WASHER
POST ANGLE
300
120
600
114x165x203 GLULAM BLOCK
CONCRETE WINGWALL
DRILL AND BOND 1¼"ø BOLT IN 35ø HOLE USING APPROVED EPOXY GROUT FROM DSM
DRILL AND BOND ¾"ø BOLT IN 22ø HOLES USING APPROVED EPOXY GROUT FROM DSM
1124 1111 889

PARTIAL ELEVATION – GLULAM DECK ANCHORAGE
(EXTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ₵
WASHER ℞6x152x152 W/ 35ø HOLE CENTERED
6-3/4"ø DOME HEAD BOLTS & NUTS W/ MALLEABLE IRON WASHERS (TYP.)
POST ANGLE
GLULAM BLOCK (BEHIND)
4"ø SPLIT RINGS OR BACK TO BACK SHEAR PLATES. (TYP)
151 151

PARTIAL ELEVATION – GLULAM DECK ANCHORAGE
(INTERIOR FACE SHOWN)
1:10

SYMMETRICAL ABOUT ₵
2-DOME HEAD BOLTS & NUTS, ¾"ø, W/ MALLEABLE IRON WASHER (TYP.)
89 165
WASHER ℞6x152x152 W/ 35ø HOLE CENTERED
1¼"ø DOME HEAD BOLT & NUT (TYP.)
152 152 203

(TIMBER BARRIER ON GLULAM DECK)
1
1:10

660
254 235 171
13
89 165 343
235x343x203 GLULAM BLOCK
GLULAM RAIL
114x355x457 GLULAM BLOCK
305
E.Q. E.Q.
152x305 GLULAM CURB RAIL (CONT.)
152 203
POST ANGLE
203x254x1124 GLULAM POST
16ø280 Lg. LAG SCREW W/ WASHER
103x165x203 GLULAM BLOCK
38x89 SAWN WOOD RETAINER BLOCK (BEYOND)
203x305x1219 GLULAM SCUPPER
WEARING SURFACE
235 THK STRESS-LAMINATED DECK
1124 1111 165

POST ANGLE DETAIL
1:5

L102x102x9.5
19ø
127
52
ø HOLE

NOTES:

1. STRUCTURAL STEEL FOR PLATES SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04 GRADE 300W. STRUCTURAL STEEL FOR ROLLED SECTIONS SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.20-04/G40.21-04 GRADE 350W.

2. GALVANIZE ALL STEEL COMPONENTS. GALVANIZING TO COMFORM TO ASTM A123/A123M WITH A MINIMUM COATING OF 600 g/m2. TOUCH UP ALL DAMAGED OR REMOVED GALVANIZING WITH TWO COATS OF ZINC RICH PAINT.

3. ALL WELDING TO CONFORM TO CAN/CSA-W59.1 AND IS TO BE UNDERTAKEN BY A FABRICATOR QUALIFIED IN ACCORDANCE WITH CAN/CSA-W47.1.

4. DOME HEAD BOLTS SHALL BE GALVANIZED ASTM A307 GRADE A. BOLT THREADS TO BE EXCLUDED FROM THE SHEAR PLANES.

5. BOLTS SHALL BE TIGHTENED TO A SNUG-TIGHT CONDITION.

6. MALLEABLE IRON WASHERS SHALL BE GALVANIZED ASTM A47 GRADE 32510.

7. SPLIT RINGS SHALL BE GALVANIZED.

8. LAG SCREWS SHALL BE GALVANIZED AND CONFORM TO CAN/CSA-B34.

9. TIMBER RAILINGS TO BE PRE-ASSEMBLED IN THE SHOP WITH DECK PANELS TO ENSURE PROPER FIT-UP.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REV | DATE | DESCRIPTION | BY

CONSULTANT:
BROWN | CO

PREPARED FOR:
MOSES STRUCTURAL ENGINEERS

WOOD BRIDGE DESIGN EXAMPLE
STRESS-LAMINATED DECK OPTION
GLUED-LAMINATED TIMBER BARRIER I

DES	A.L.	NOV2017	FILE NAME:	JOB	DRAWING	REV
DRW	G.D.	NOV2017		2017-075	SWA-08	0
CHK	S.B.	NOV2017	SCALE: AS INDICATED			

NOTES:

1. THIS DRAWING TO BE READ IN CONJUNCTION WITH DRAWING SWA-08.

SYMMETRICAL ABOUT ℄

127 127 51

6mm MAX. GAP

UPPER RAIL

89
165

POST

OUTLINE OF SPLICE ℄

369

△1
1:10

SYMMETRICAL ABOUT ℄

SPLICE ℄13x343x737

4-18ø HOLES
8-34ø HOLES

89
165
343

127 127 51
737

STEEL SPLICE PLATE

SYMMETRICAL ABOUT ℄

GLULAM POST

SEE STEEL SPLICE PLATE DETAIL

GLULAM BLOCK

UPPER RAIL

E.Q. E.Q.

16ø DOME HEAD BOLTS & NUTS W/ MALLEABLE IRON WASHER (2 EACH SIDE)

6mm MAX. GAP (TYP.)

8-32ø DOME HEAD BOLT & NUT W/ WASHER

△3

PLAN

GLULAM RAIL SPLICE
1:10

76   3@152 = 456

E.Q. E.Q.

△2
1:10

608

℄ GLULAM CURB

19ø DOME HEAD BOLT & NUT W/ WASHER (TYP)

△3

PLAN

GLULAM CURB SPLICE
1:10

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

Wood *WORKS!* is a program of the Canadian Wood Council

Ontario Wood *WORKS!* 1-866-886-3574