Wood Bridge Design Based on the Ontario Wood Highway Bridge Reference Guide

OVERVIEW OF TOPICS

Reference Materials

Wood Materials

Bridge Systems and Sample Bridges

Durability Considerations

Design Example
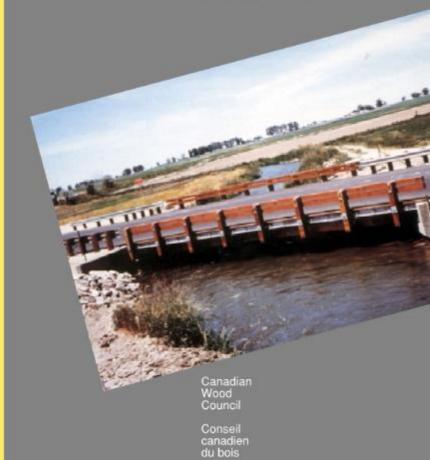
Reference Materials

- Wood Highway Bridges (CWC)

- Canadian Highway Bridge Design Code 2014
  (Code canadien sur le calcul des ponts routiers)

- CAN/CSA 086

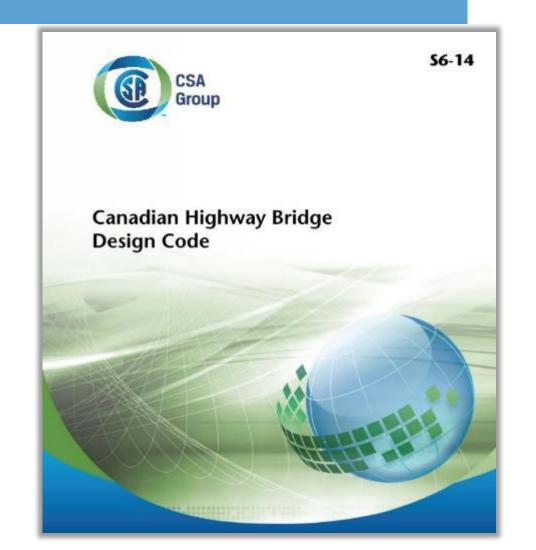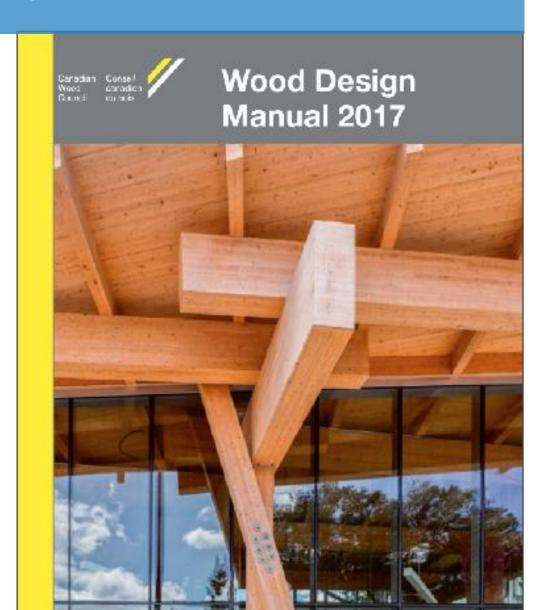- Ontario Wood Bridge Reference Guide

1992 – Wood Highway Bridges (CWC)

Wood Text Series

2

Wood Highway Bridges

Decks
Beams & Girders
Loads & Analysis

Detailing
Specifications
Trends

Canadian
Wood
Council

Conseil
canadien
du bois

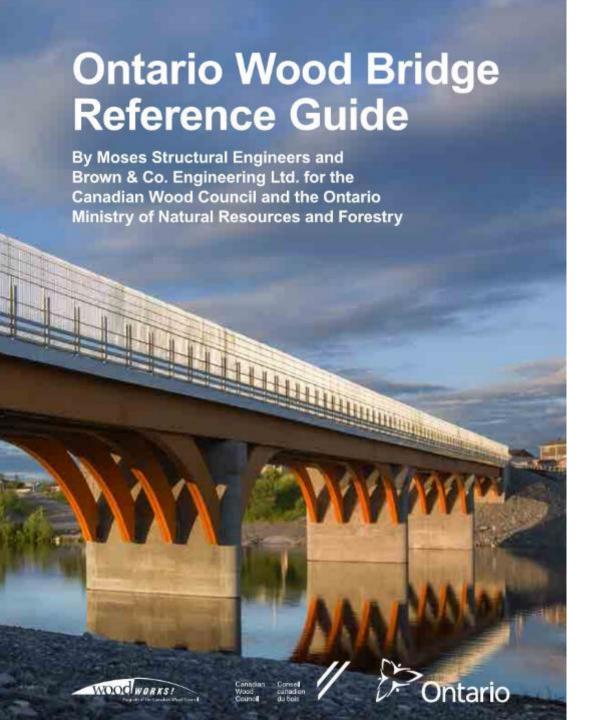**2014 – Canadian Highway Bridge Design Code**
2014 - Code canadien sur le calcul des ponts routiers

2014 – CAN/CSA O86-14

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

Canadian Wood Council — Conseil canadien du bois

wood WORKS!
Program of the Canadian Wood Council

Ontario
Ministry of Natural Resources and Forestry

OVERVIEW OF TOPICS

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Wood Materials

Bridge Systems and Sample Bridges

Durability Considerations

Design Example

Wood Materials

- Solid Sawn

- Glued-laminated timber (glulam)

- Laminated veneer lumber (LVL)

- Parallel Strand Lumber (PSL)

- Composites

OVERVIEW OF TOPICS

Reference Materials

Wood Materials

Bridge Systems and Sample Bridges

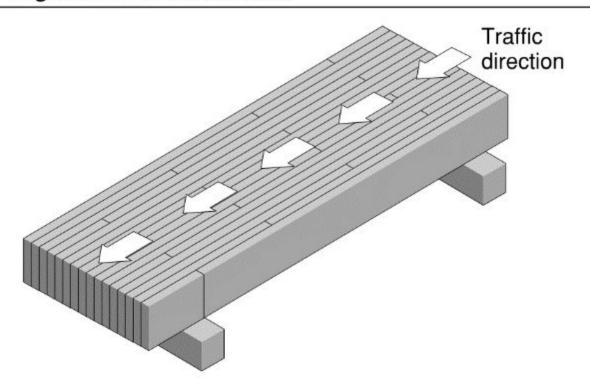Durability Considerations

Design Example

Bridge Systems and Sample Bridges

- Decks

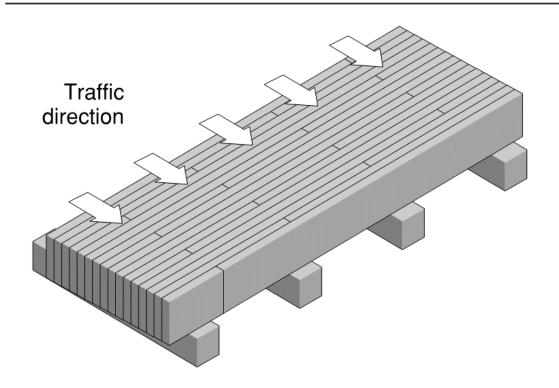- Superstructures

- Substructures

1992 – Wood Highway Bridges (CWC)

1992 – Wood Highway Bridges (CWC)

Longitudinal laminated deck

Traffic direction

1992 – Wood Highway Bridges (CWC)

Transverse laminated deck

Traffic direction

1992 – Wood Highway Bridges (CWC)

Figure 2.8
Cross section of longitudinal stress-laminated deck

- Wood post and flex-beam guide rail
- Wood curb
- Asphalt wearing surface
- Wood laminations
- High strength post-tensioning bar installed in predrilled holes
- Steel channel (post-tensioning bulkhead)

1992 – Wood Highway Bridges (CWC)
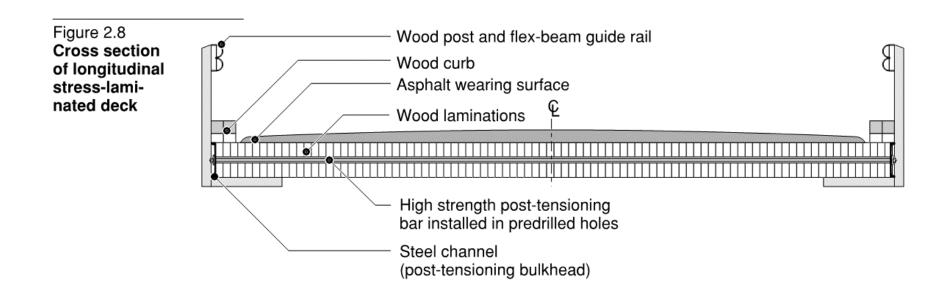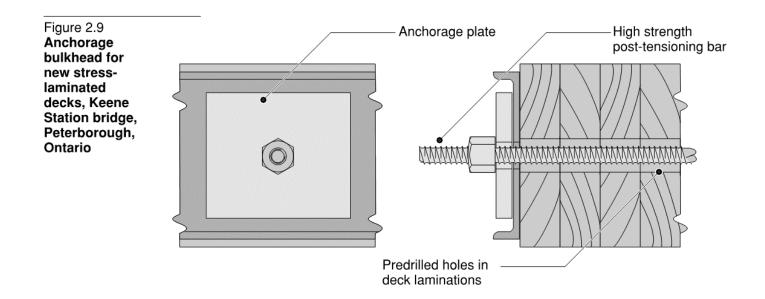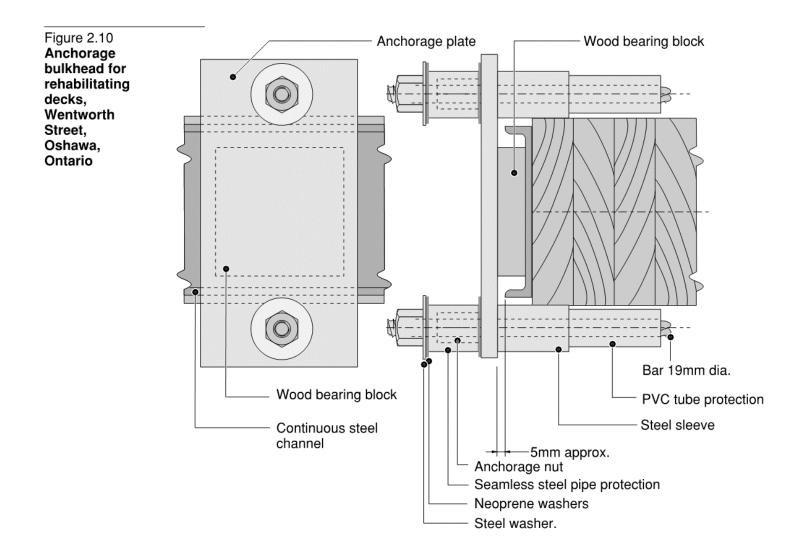
Figure 2.12
Segmental TSL deck on Aquasabon River, Terrace Bay, Ontario

1992 – Wood Highway Bridges (CWC)
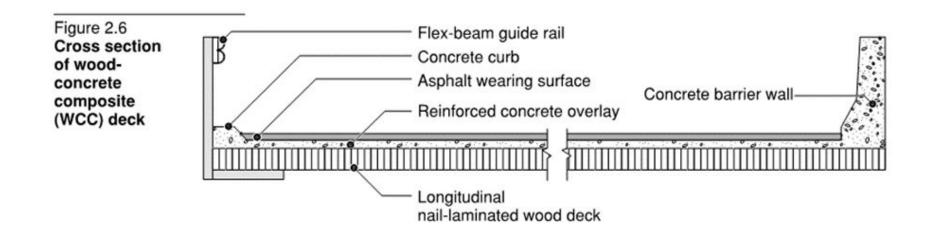
Figure 2.7
Rehabilitation of an LNL deck by post-tensioning, Kabaigon River, Atikokan, Ontario

1992 – Wood Highway Bridges (CWC)

Figure 2.9
Anchorage bulkhead for new stress-laminated decks, Keene Station bridge, Peterborough, Ontario

Anchorage plate

High strength post-tensioning bar

Predrilled holes in deck laminations

1992 – Wood Highway Bridges (CWC)

Figure 2.10
Anchorage bulkhead for rehabilitating decks, Wentworth Street, Oshawa, Ontario

Anchorage plate

Wood bearing block

Wood bearing block

Continuous steel channel

Bar 19mm dia.

PVC tube protection

Steel sleeve

5mm approx.
Anchorage nut
Seamless steel pipe protection
Neoprene washers
Steel washer.

1992 – Wood Highway Bridges (CWC)

Figure 2.6
Cross section of wood-concrete composite (WCC) deck

Flex-beam guide rail
Concrete curb
Asphalt wearing surface
Reinforced concrete overlay
Concrete barrier wall
Longitudinal nail-laminated wood deck

2017 – Ontario Wood Bridge Reference Guide

1992 – Wood Highway Bridges (CWC)

Figure 2.18
Double plank surface, beam and plank deck

BC Ministry of Forests typically provides additional untreated running surface to reduce wear

Longitudinal planks 63.5mm to 140mm thick

Heavy steel or wood girders

Transverse wood beams spaced 200mm to 400mm apart

Girder spacing 1m to 2.5m

Bridge Systems and Sample Bridges

- Decks

- **Superstructures**

- Substructures

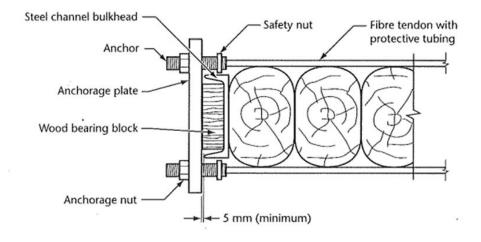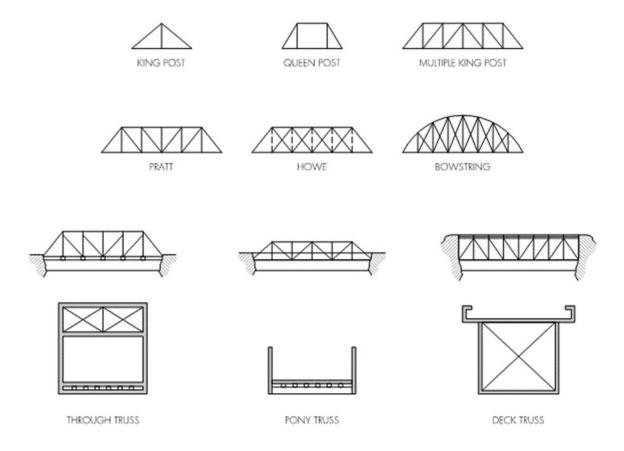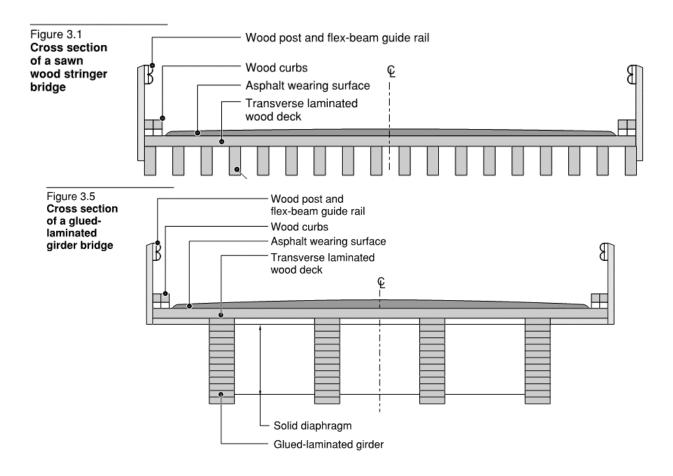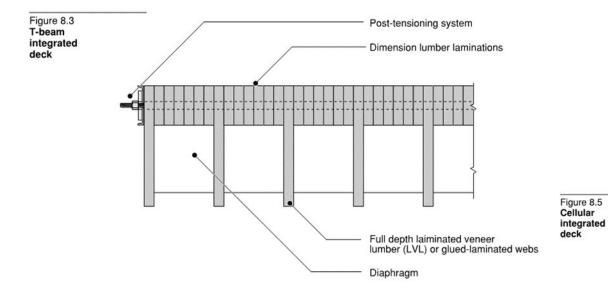## CAN/CSA S6-14 – CANDIAN HIGHWAY Bridge Design Code (CHBDC)

Figure 16.9
Post-tensioning system for stressed log bridges
(See Clause 16.9.3.)

2017 – Ontario Wood Bridge Reference Guide
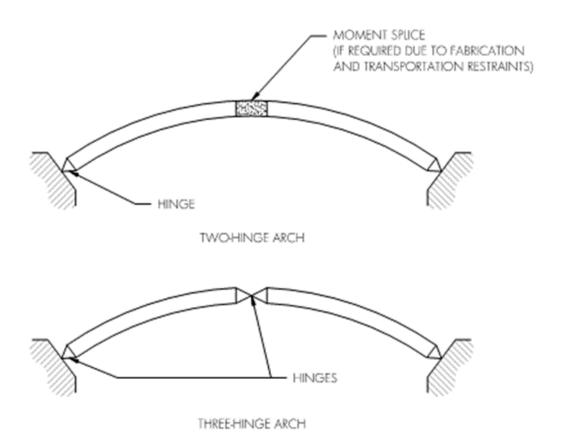

KING POST

QUEEN POST

MULTIPLE KING POST

PRATT

HOWE

BOWSTRING

THROUGH TRUSS

PONY TRUSS

DECK TRUSS

TRUSS BRIDGE TYPES

1992 – Wood Highway Bridges (CWC)

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

Figure 3.5
Cross section of a glued-laminated girder bridge
- Wood post and flex-beam guide rail
- Wood curbs
- Asphalt wearing surface
- Transverse laminated wood deck
- Solid diaphragm
- Glued-laminated girder

1992 – Wood Highway Bridges (CWC)

Figure 8.3
T-beam integrated deck

- Post-tensioning system
- Dimension lumber laminations
- Full depth laiminated veneer lumber (LVL) or glued-laminated webs
- Diaphragm

Figure 8.5
Cellular integrated deck

- Spacer deck elements are regular dimension lumber
- Full depth laminated veneer lumber (LVL) or glued-laminated webs
- Post-tensioning rods

2017 – Ontario Wood Bridge Reference Guide

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

HINGE
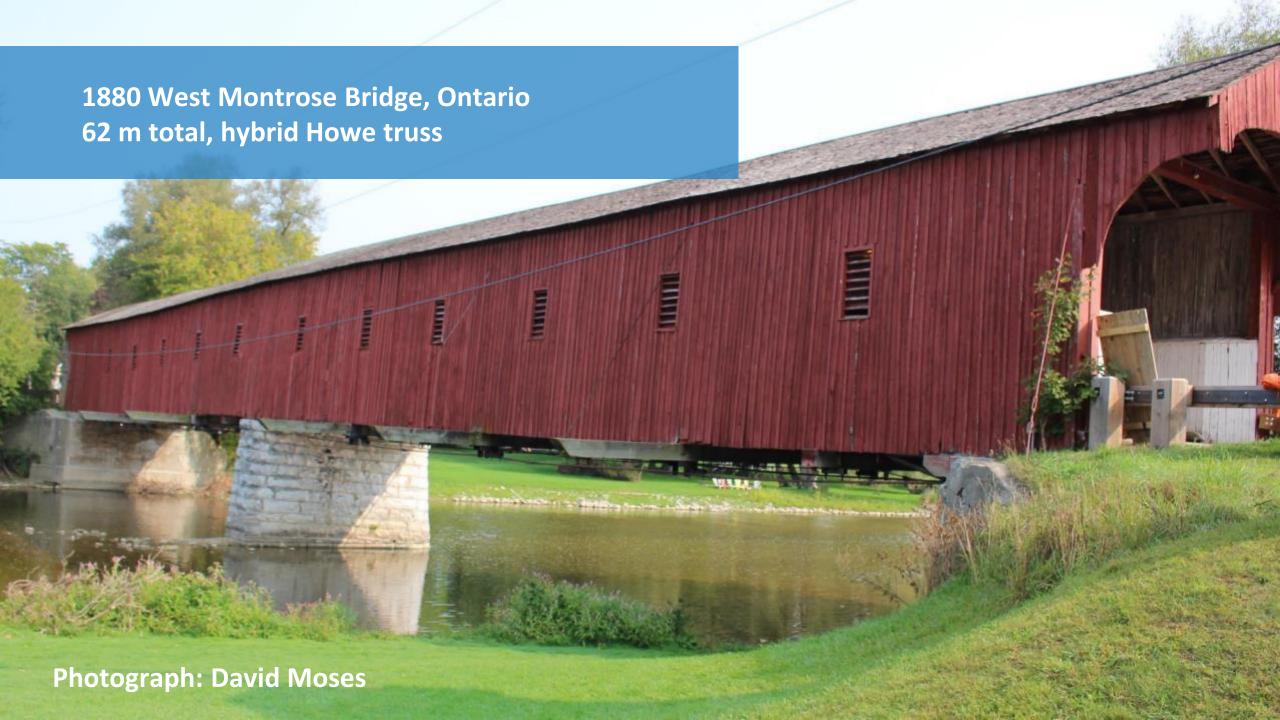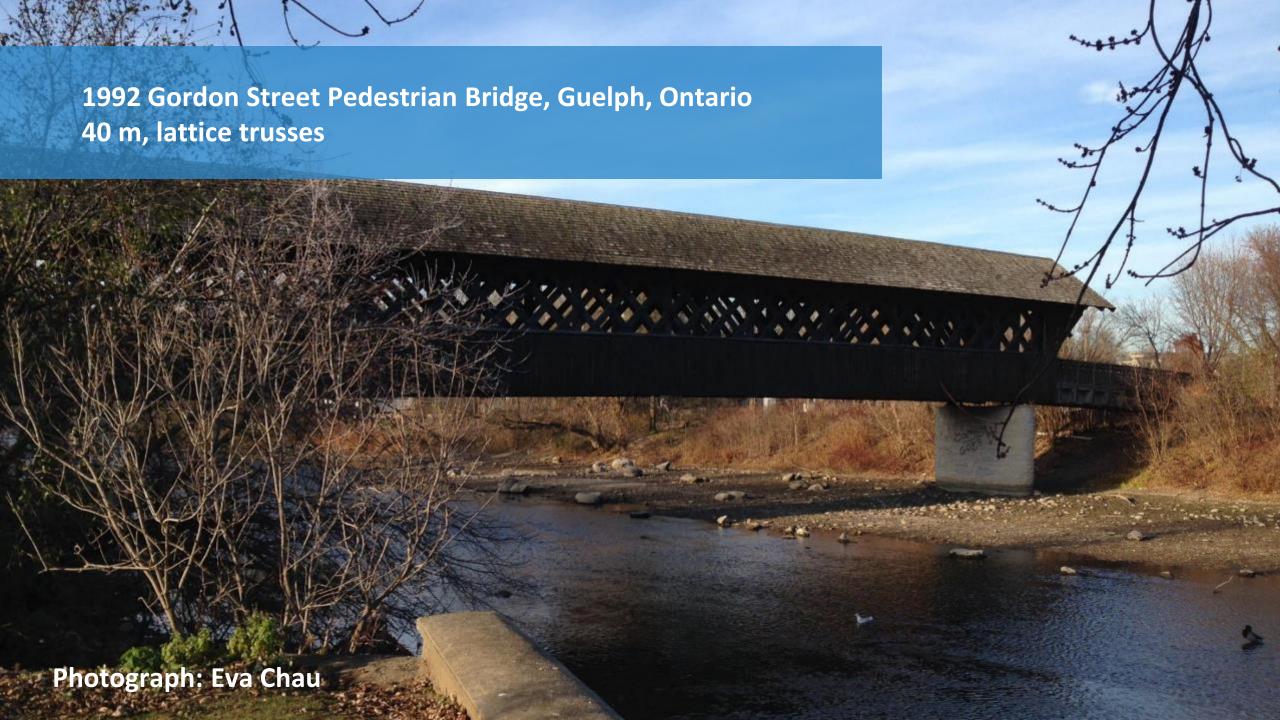
TWO-HINGE ARCH

HINGES

THREE-HINGE ARCH

Bridge Systems and Sample Bridges

- Decks

- Superstructures

- **Substructures**

Bridge Systems and Sample Bridges

- Abutments

  - Most commonly made of concrete

  - Timber can be used (must be pressure treated)

- Bents

  - For intermediate support between abutments
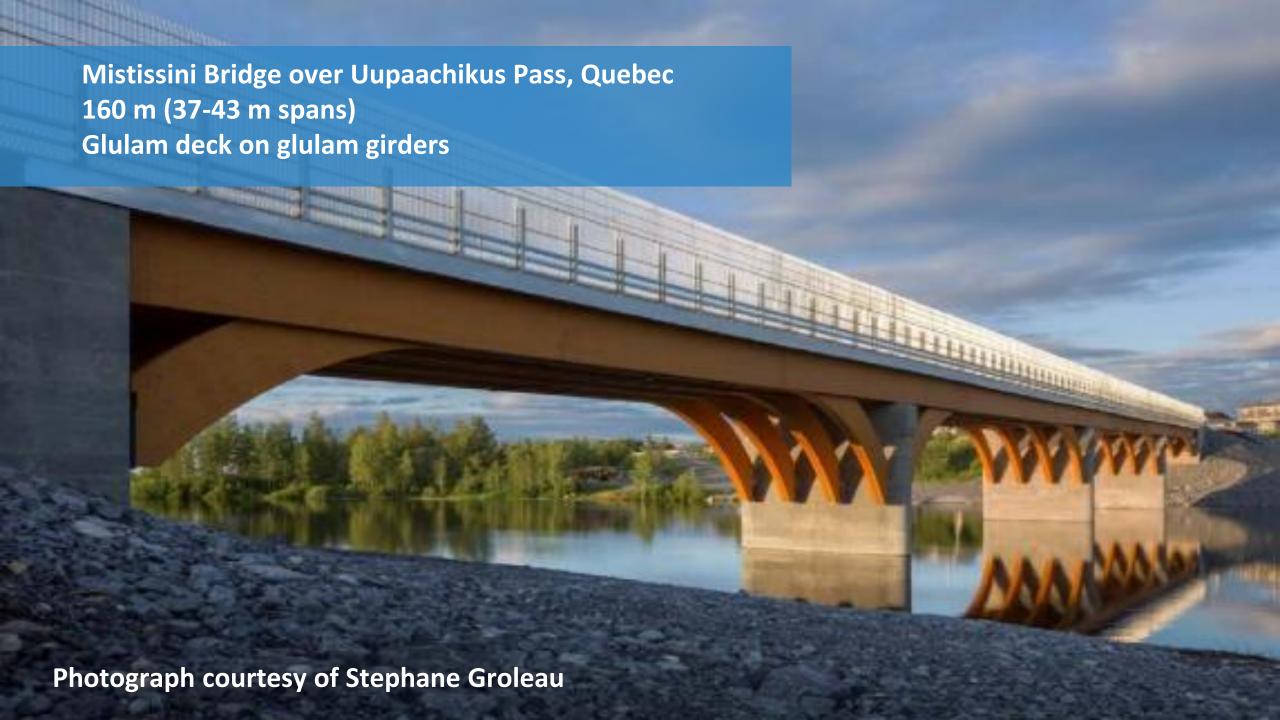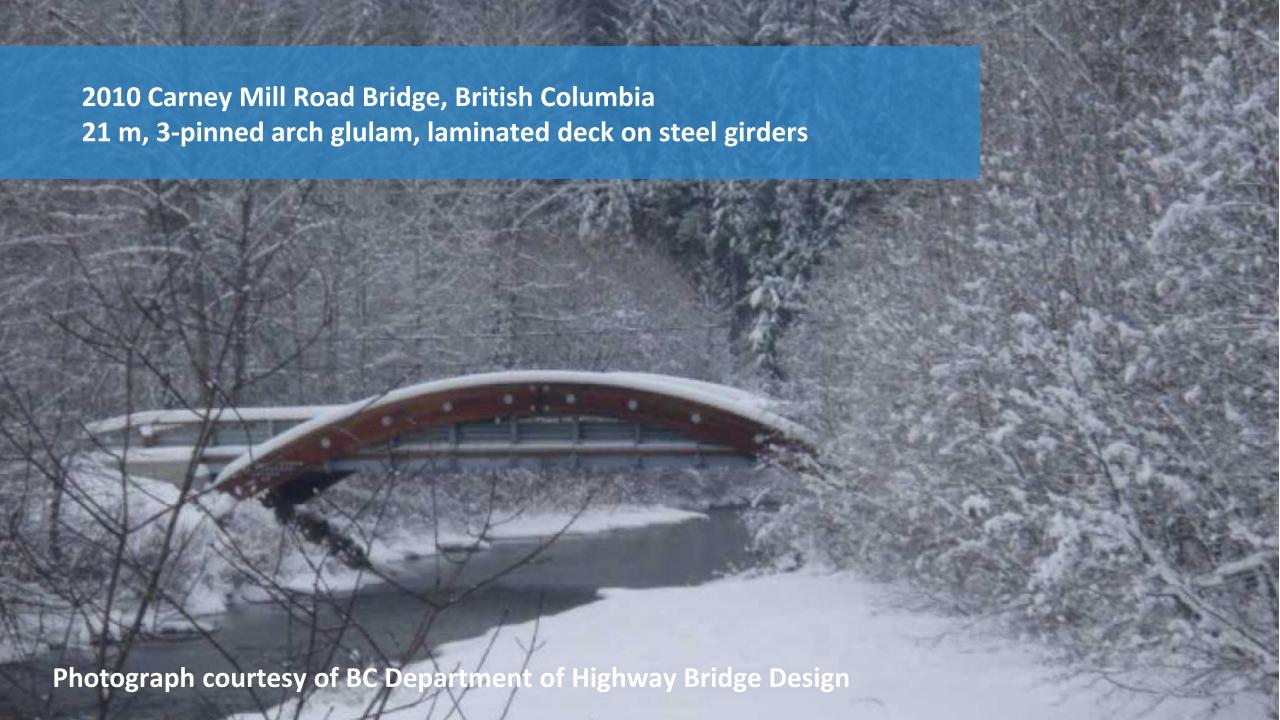
  - Timber piles or frames

1880 West Montrose Bridge, Ontario
62 m total, hybrid Howe truss

Photograph: David Moses

1992 Gordon Street Pedestrian Bridge, Guelph, Ontario
40 m, lattice trusses

Photograph: Eva Chau

1993 North Pagwachuan River Bridge, Ontario
50 m steel girders, stress-lam.

Photograph courtesy of MTO

1994 Hoiles Creek Bridge, Ontario
30 m, steel girders, stress-lam.

Photograph courtesy of MTO

2006 Fibreglass Reinforced Wood Deck, Ontario

Photograph courtesy of Guardian Bridge Rapid Construction Inc.

2008 Mather Creek Bridge, Ontario
23 m, PSL slab composite precast concrete deck

Photograph courtesy of MTO

2009 Nestor Falls Bridge, Ontario
16 m, PSL with composite precast concrete deck

DANGER
Fast Water - Keep
Courant rapide
Ne pas approcher

◇ DAM AHEAD – KEEP AWAY ◇

Photograph courtesy of MTO

2009 Waterfront Wave Decks, Toronto, Ontario

Photograph courtesy of Ontario WoodWORKS!

2012 Pennock Creek Culvert, Ontario
8 m, laminated PSL on steel sheet pile abutments

Photograph courtesy of MTO

2013 Providence Road Bridge, Ontario
Glulam deck on glulam girders

Photograph courtesy of Timber Restoration Systems

2013 Dickinson Road Bridge, Ontario
Glulam deck on glulam girders

Photograph courtesy of Timber Restoration Systems

1912/2013 Snake Road Bridge, Ontario
Glulam deck on glulam girders

Photograph courtesy of Timber Restoration Systems

2014 Silver Falls Creek Bridge, Ontario
8 m, PSL slab on steel sheet piles

Photograph courtesy of MTO

Mistissini Bridge over Uupaachikus Pass, Quebec
160 m (37-43 m spans)
Glulam deck on glulam girders

Photograph courtesy of Stephane Groleau

1946 Lillooet Bridge, Lillooet, BC
103 m, Steel truss retrofit, pedestrian only now

2003 McCulloch Trestle , Penticton, BC

Photograph: David Moses

2010 Carney Mill Road Bridge, British Columbia
21 m, 3-pinned arch glulam, laminated deck on steel girders

Photograph courtesy of BC Department of Highway Bridge Design

2010 Answer Creek Bridge, BC
Concrete deck on glulam girders (non-composite)

Photograph courtesy of BC Department of Highway Bridge Design

2010 Large Creek Bridge, BC
Concrete deck on glulam girders (non-composite)

Photograph courtesy of BC Department of Highway Bridge Design

1948 – Loon Lake, Oregon (glulam)

Photograph courtesy of Mike Ritter

1966

2010

Keystone Wye Interchange, South Dakota (glulam)

Lower Burnett Road Bridge, Washington
Glulam deck

Photograph courtesy of Western Wood Structures Inc. Tualatin, Oregon

Overpeck Park Bridges, New Jersey
2 x 42 m spans, glulam deck with glulam arches

Photograph courtesy of Western Wood Structures Inc. Tualatin, Oregon

2001 Highway bridge on A89, France

Photograph courtesy of Arboresence SARL

2001 Road bridge over the Crest River, France

Photograph courtesy of Arboresence SARL

1996 Evenstad Bridge, Norway
5 x 36 m, glulam truss, stress-lam. glulam deck

Photograph courtesy of FPInnovations

2003 Filsa Bridge, Norway
70 m max, glulam truss, stress-lam. glulam deck

Photograph courtesy of FPInnovations

2005 Kjollsaeter Bridge, Norway
145 m (45 m max), 100 tons (>800 kN), glulam truss, conc deck

Photograph courtesy of FPInnovations

2007 Skogsrud Bridge, Norway
37 m, glulam arch with stress-lam. glulam deck

Photograph courtesy of FPInnovations

2012 Tretten Bridge, Norway
70 m longest span, glulam truss, stress-lam. glulam deck

Photograph courtesy of FPInnovations

OVERVIEW OF TOPICS

Reference Materials

Wood Materials

Bridge Systems and Sample Bridges

Durability Considerations

Design Example

Durability Considerations

- Protection by design

- Preservative treatment

- Replaceable elements

Durability

Protection by design:

- With a roof
- Under a deck
- Attention to snow accumulation

Durability

- Limit contact to water, allow drying, drainage
- Orient connectors vertically to allow drainage
- Do not expose end grain to water
- Raise bases
- Use metal flashings (correctly)

Durability

Detailing for durability is also very important

- Position bearings on high pedestals above bearing seat

- Detail to allow for air flow around members

- Detail connections to minimize the potential for trapped moisture

- Use metal flashings to protect critical components

- All steel hot-dipped galvanized or stainless

Durability

Moisture content 26%-28% starts decay

- Glulam manufactured to ≤15% moisture content
- Moisture content for wood bridges in service measured between 15%-19%

Glulam members must be preservative treated

- Oil-borne preservatives preferred over water-borne preservatives
- Incising necessary to increase depth of penetration
- All fabrication to be completed before treatment
- Excess preservative to be avoided to prevent leaching and interaction with waterproofing/paving

OVERVIEW OF TOPICS

Reference Materials

Wood Materials

Bridge Systems and Sample Bridges

Durability Considerations

Design Example

Design Example

- Design Considerations

- Proposed Concept

- Material Properties

- Loading

- Glulam Deck Panels

- Glulam Girder Beams

Design Example

- Stiffener Beams

- Glued-laminated timber diaphragms

- Connection Design

Design Considerations

- CHBDC

- MTO Ontario Provincial Standard Specification (OPSS)

- Ontario Hertiage Bridge Guidelines (OHBG)

- Prefabrication

- Prestressing

- Service Life and Durability

CHBDC Relevant Sections

- CAN/CSA S6 – Section 3 – Loads
  - Dynamic load allowance 30% reduction for wood decks
  - Shrinkage and swelling

- CAN/CSA S6 – Section 5 – Methods of Analysis
  - 5.5.8 Transverse wood deck
  - 5.6 Simplified methods for longitudinal load effects
  - 5.7 Analysis of decks

CHBDC Relevant Sections

- CAN/CSA S6 – Section 9 – Wood Structures
  - Clause 9.4 Limit States Design
  - Clause 9.5 General Design
  - Clause 9.6 Flexure
  - Clause 9.7 Shear
  - Clause 9.11 Solid Wood
  - Clause 9.12 Glulam Timber
  - Clause 9.15 Connections
  - Clause 9.17 Durability
  - Clause 9.21 Nail-laminated wood decks

CHBDC Relevant Sections

- CAN/CSA S6 – Section 9 – Wood Structures
  - Clause 9.22 Wood-concrete composite decks
  - Clause 9.23 Stress-laminated wood decks
  - Clause 9.24 Glued-laminated decks

CHBDC Relevant Sections

- CAN/CSA S6 – Section 10 – Steel Structures
- CAN/CSA S6 – Section 14 – Evaluation
- CAN/CSA S6 – Section 15 – Rehabilitation and Repair
- CAN/CSA S6 – Section 16 – Fibre-Reinforced Structures
- CAN/CSA S6 – Section 17 – Aluminum Structures

Design Example

- Design Considerations

- **Proposed Concept**

- Material Properties

- Loading

- Glulam Deck Panels

- Glulam Girder Beams

DESIGN EXAMPLE OVERVIEW
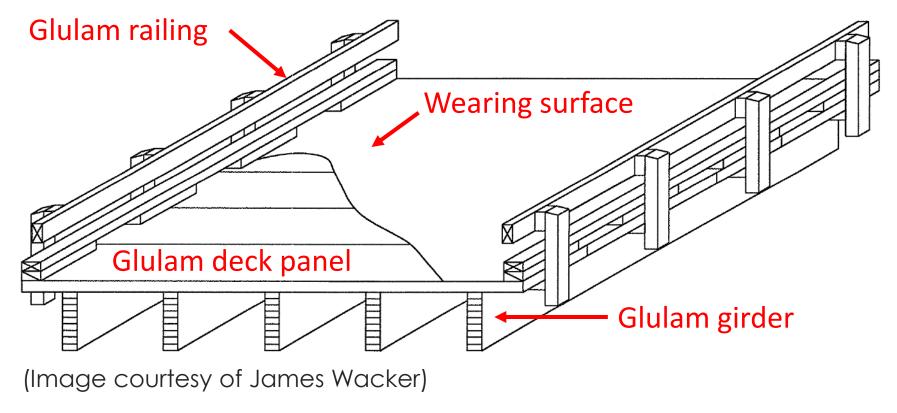
(Photograph courtesy of Laminated Concepts Inc.)

- Glued-laminated timber (glulam) deck panels supported by glued-laminated timber girders
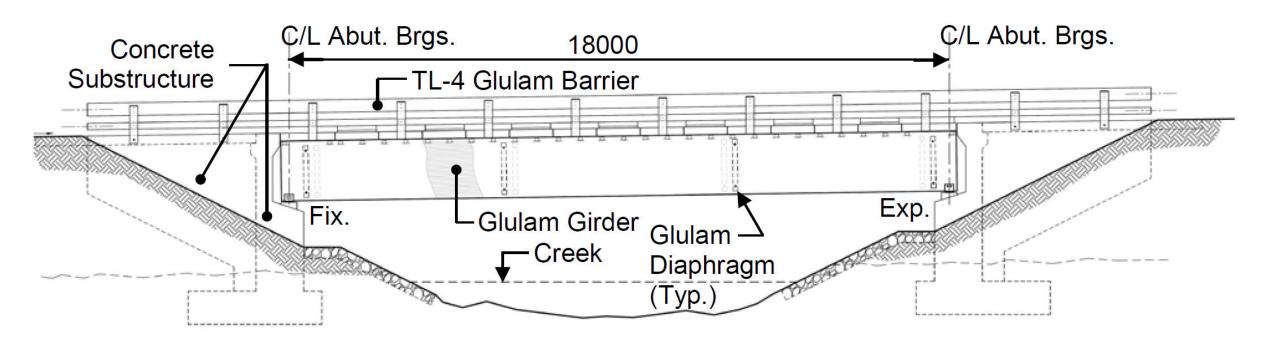
DESIGN EXAMPLE OVERVIEW

- Design Codes:

  - CAN/CSA-S6-19 Canadian Highway Bridge Design Code (CHBDC)

  - CAN/CSA-O86-14 Engineering Design in Wood (O86)

  - Other relevant technical literature

- Superstructure design only

DESIGN EXAMPLE OVERVIEW

Glulam railing

Wearing surface

Glulam deck panel

Glulam girder

(Image courtesy of James Wacker)
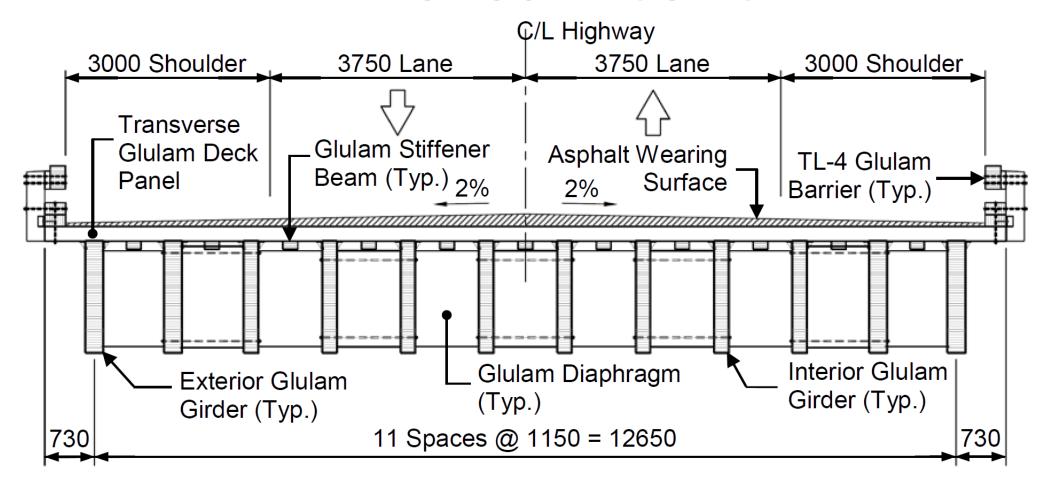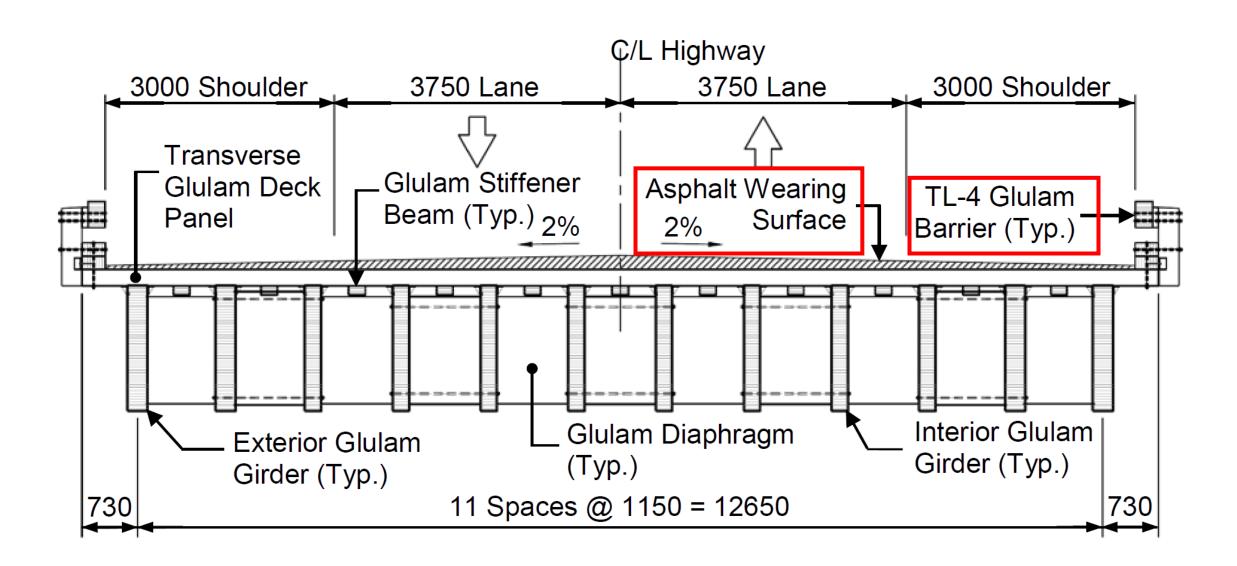
- Transverse glulam deck panels on glulam girders

- Popular system in US, but rarely built in Canada

- 2019 CHBDC now addresses glulam deck panels

THE PROPOSED CONCEPT
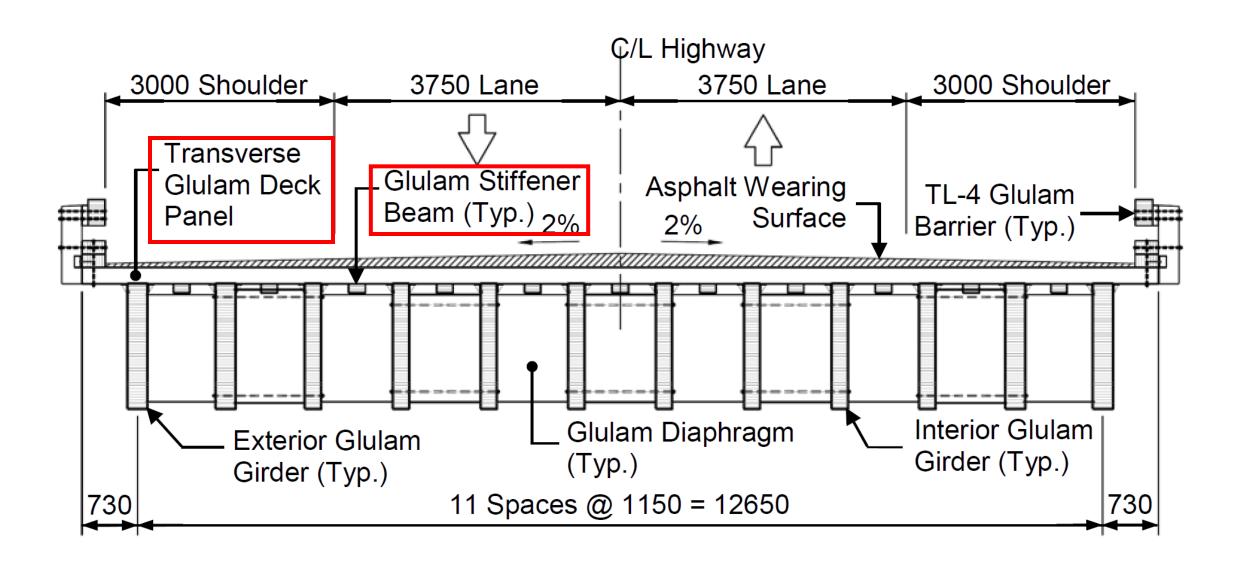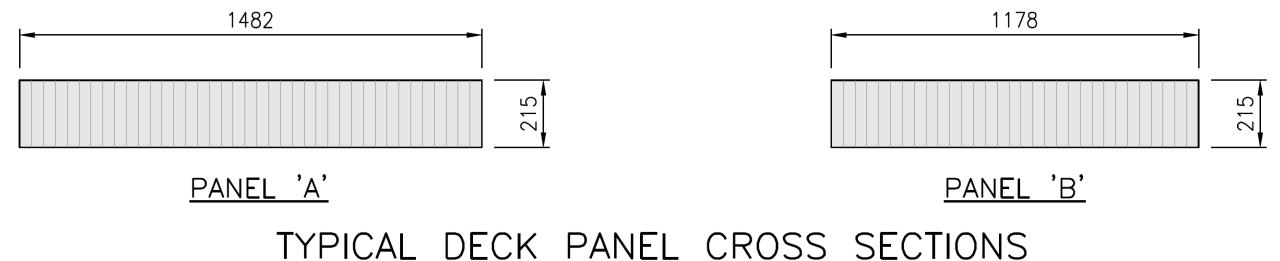
- Transverse glulam deck panels on glulam girders

- 18 m single span vehicular bridge

THE PROPOSED CONCEPT

- Geometric cross-section valid for undivided arterial road with 110 km/h speed limit (i.e. Trans-Canada Highway)
- Shoulder width maintained across bridge

THE PROPOSED CONCEPT

THE PROPOSED CONCEPT

GLULAM DECK PANELS

1482

215

PANEL 'A'

1178

215

PANEL 'B'
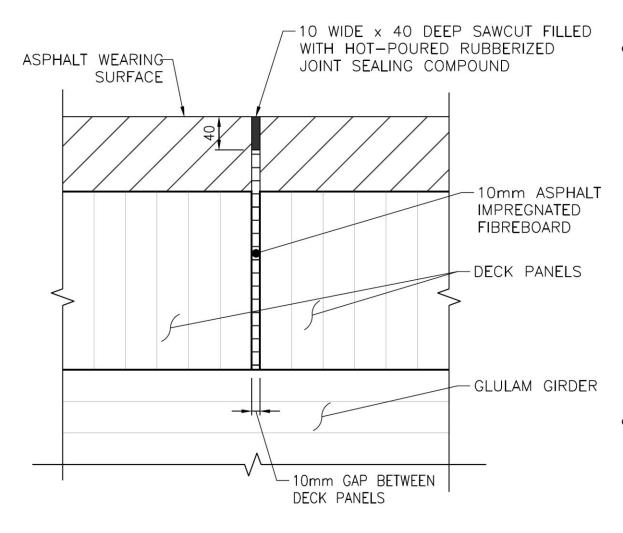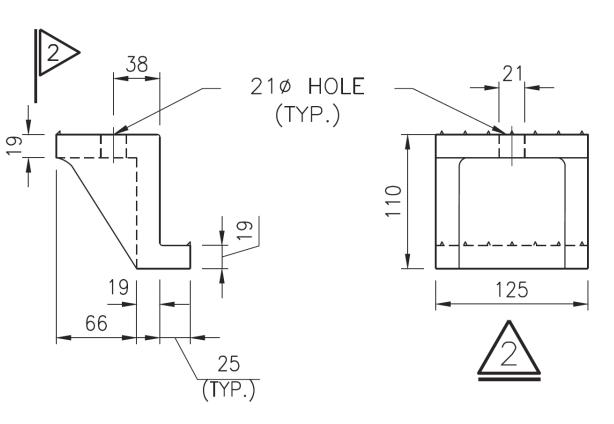
TYPICAL DECK PANEL CROSS SECTIONS
1:20

- Deck panels are essentially glulam beams turned on their sides
- Interior panels typically ~1200 mm wide
- Vary end panel widths to suit bridge length

GLULAM DECK PANELS

10 WIDE x 40 DEEP SAWCUT FILLED WITH HOT-POURED RUBBERIZED JOINT SEALING COMPOUND

ASPHALT WEARING SURFACE

40

10mm ASPHALT IMPREGNATED FIBREBOARD

DECK PANELS

GLULAM GIRDER

10mm GAP BETWEEN DECK PANELS

TYPCAL JOINT DETAIL BETWEEN DECK PANELS
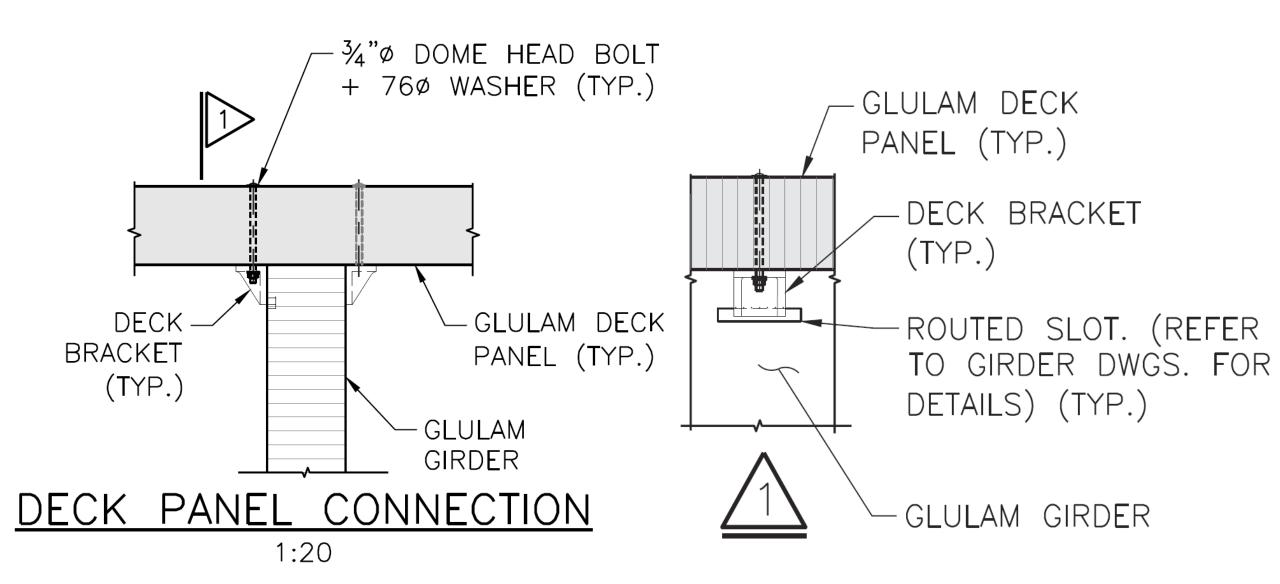
- Gap between adjacent panels necessary to accommodate swelling of panels in service

  - Glulam manufactured to ≤15% moisture content

  - Moisture content for wood bridges in service measured between 15%-19%

- Proposed joint detail similar to expansion detail at ends of approaches slabs for steel and concrete bridges

GLULAM DECK PANELS

21⌀ HOLE (TYP.)

38

19

19

19

66

25 (TYP.)

21

110

125

⟨2⟩

◇F DECK BRACKET DETAIL
(MATERIAL: CAST ALUMINUM ALLOY)

(Photograph courtesy of Laminated Concepts Inc.)

GLULAM DECK PANELS

¾"∅ DOME HEAD BOLT + 76∅ WASHER (TYP.)

DECK BRACKET (TYP.)

GLULAM DECK PANEL (TYP.)

GLULAM GIRDER

DECK PANEL CONNECTION
1:20

GLULAM DECK PANEL (TYP.)

DECK BRACKET (TYP.)

ROUTED SLOT. (REFER TO GIRDER DWGS. FOR DETAILS) (TYP.)

GLULAM GIRDER

GLULAM STIFFENER BEAMS

TYPICAL DECK–TO–STIFFENER BEAM CONNECTION
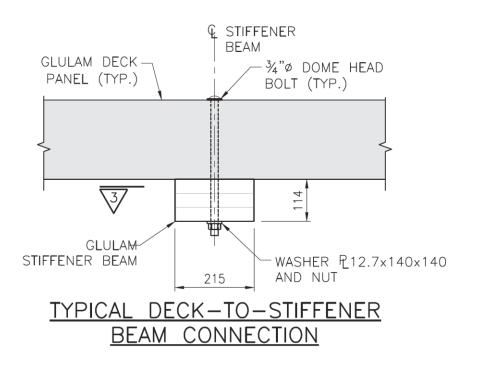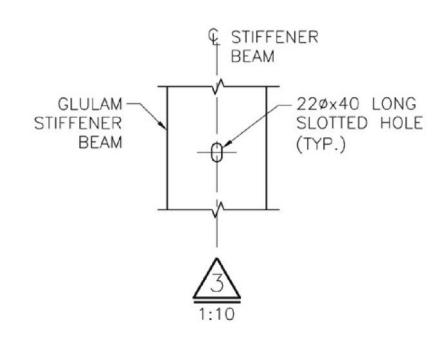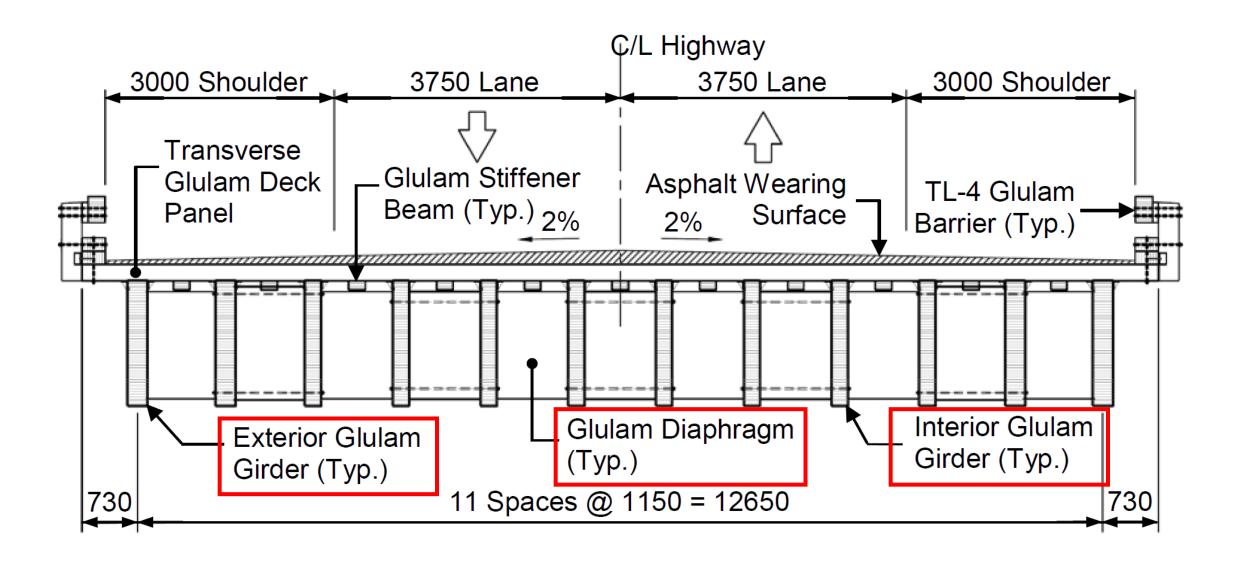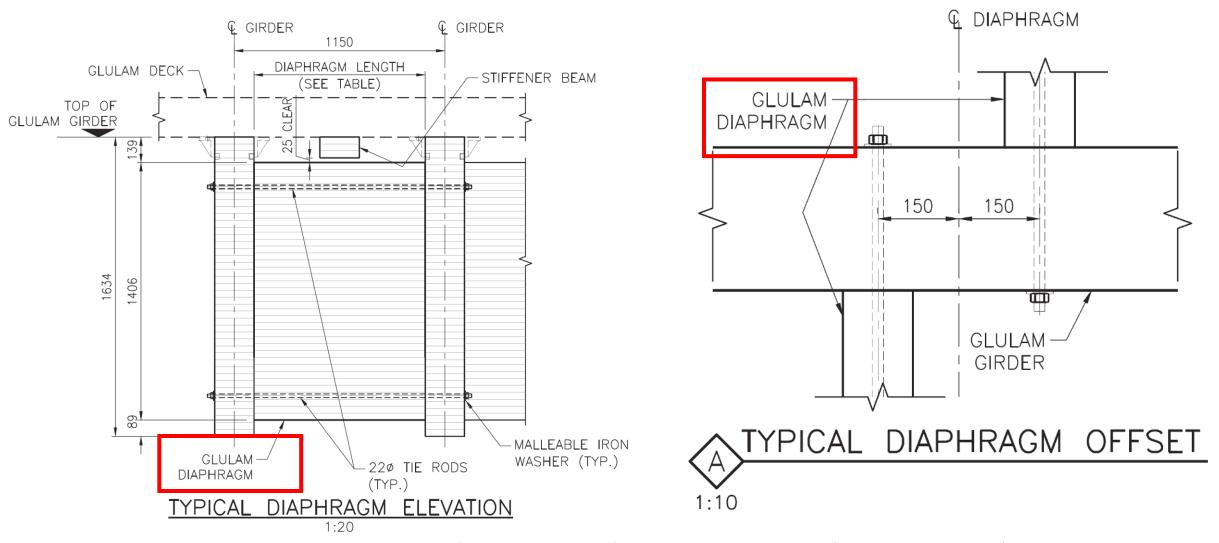
- Stiffener beams behave like external dowels

- Limit relative deflections between adjacent deck panels

- Slot holes in continuous stiffener beams to prevent development of restraint forces

GLULAM DIAPHRAGMS

TYPICAL DIAPHRAGM ELEVATION
1:20

TYPICAL DIAPHRAGM OFFSET
A 1:10
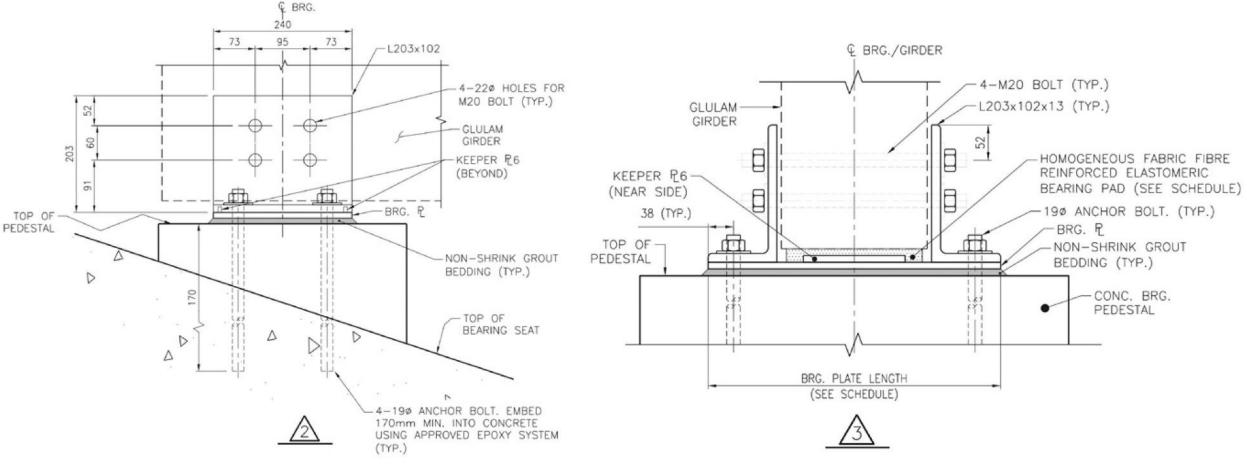
- Brace girders against lateral-torsional buckling
- Share lateral loads amongst girders

BEARING ASSEMBLIES

- Transmit vertical loads to substructure and allow rotation of girders
- Restrain longitudinal and transverse movements as necessary
- Elevate girders above bearing seat

Design Example

- Design Considerations

- Proposed Concept

- **Material Properties**

- Loading

- Glulam Deck Panels

- Glulam Girder Beams

MATERIALS

- <u>Spruce-Lodgepole Pine-Jack Pine</u> glulam properties assumed for **girders, diaphragms, and stiffener beams**

• 20f-E glulam stress grade	f_b	=	25.6	MPa	bending moment (positive)
	f_b	=	19.2	MPa	bending moment (negative)
• 2019 CHBDC will features these properties (not previously included)	f_v	=	1.3	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

* Longitudinal shear updated to match CSA S6-19

MATERIALS

- Service condition
  - Wet service assumed for deck panels
  - Semi-wet service assumed for girders, diaphragms, and stiffener beams
  - Connections (always) designed for wet service
  - 2019 CHBDC now provides designers the opportunity to directly apply service condition factor instead of embedding it in the material properties

- Treatment factor taken as unity
  - Incising not considered to reduce strength unless laminations are thin

MATERIALS

- **Deck panels** designed as built-up system of No. 2 grade sawn lumber

- S-P-F No. 2 grade structural joist and plank properties assumed per CHBDC Table 9.16

- Better material properties could be obtained by specifying glulam layup

- American glulam industry has numerous stress grades created specifically for glulam deck panels

f_b	=	8.4	MPa	bending moment
f_v	=	1.2	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

* Longitudinal shear updated to match CSA S6-19

MATERIALS
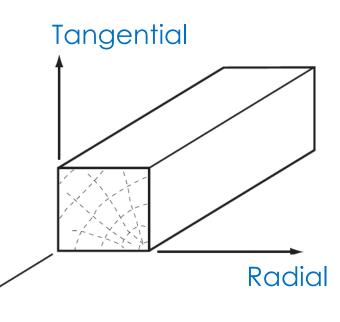
- Wood is orthotropic (i.e. distinct material properties in three orthogonal directions – longitudinal, radial, and tangential)
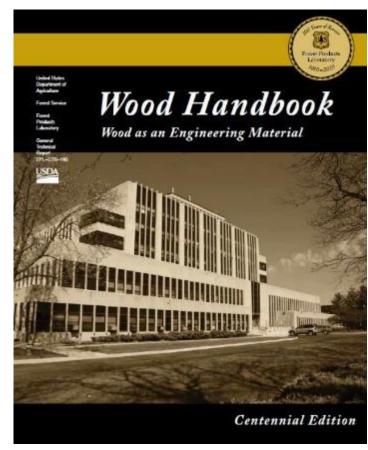
- Obtain additional material properties (as necessary) for 3D computer structural analysis from the "Wood Handbook – Wood As An Engineering Material"

- Elastic moduli, shear moduli, and Poisson's ratios necessary for computer structural analysis

Tangential

Radial

Longitudinal

Design Example

- Design Considerations

- Proposed Concept

- Material Properties

- **Loading**

- Glulam Deck Panels

- Stiffener beams

LOADS & IMPOSED DEFORMATIONS

- Self-Weight (γ_{wood} = 6.0 kN/m³)

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

LOADS & IMPOSED DEFORMATIONS

- Self-Weight (γ_{wood} = 6.0 kN/m³)

- Superimposed Dead Load
  - Wearing surface ($\gamma_{asphalt}$ = 23.5 kN/m³)
    - Asphalt varies from 185 mm at the crown to 50 mm thickness at the face of the timber railings (2% crossfall).
    - With girder spacing at 1150 mm. The average thickness for either the two interior girders nearest the crown is

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

    - 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

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

LOADS & IMPOSED DEFORMATIONS

- Self-Weight $\quad\quad\quad (\gamma_{wood} = 6.0 \text{ kN/m}^3)$

- Superimposed Dead Load
  - Wearing surface $\quad (\gamma_{asphalt} = 23.5 \text{ kN/m}^3)$
  - Barriers $\quad\quad\quad (\omega_{barrier} = 1.3 \text{ kN/m})$

LOADS & IMPOSED DEFORMATIONS

- Self-Weight $\qquad$ ($\gamma_{wood} = 6.0$ kN/m^3)

- Superimposed Dead Load
  - Wearing surface $\qquad$ ($\gamma_{asphalt} = 23.5$ kN/m^3)
  - Barriers $\qquad$ ($\omega_{barrier} = 1.3$ kN/m)

- Live Load
  - Vertical CL-625-ONT loading
  - Braking forces

Vertical CL-625-ONT Loading
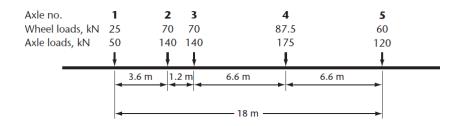
Figure A3.4.1
CL-625-ONT Truck
(See Clause A3.4.1.)
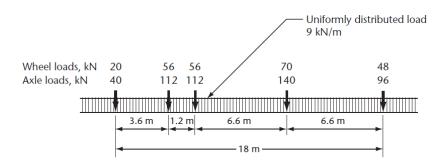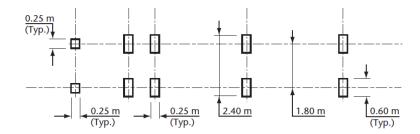
Figure A3.4.2
CL-625-ONT Lane load
(See Clause A3.4.1.)
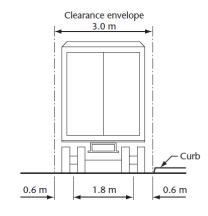
Figure 3.2
CL-W Truck
(See Clause 3.8.3.2.)

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)

DYNAMIC LOAD ALLOWANCE

- Dynamic Load Allowance
  - Accounts for interaction between vehicles and structure
  - Precludes dynamic analysis
  - Allows designer to convert dynamic load effects to an equivalent static load
  - Dependent on number of axles
  - <u>30% reduction for wood components!!!</u>

    e.g. DLA for four axles = 1 + 0.25 x 70% = 1.175 (versus 1.25)

Vertical Live Load	Dynamic Load Allowance
one axle of the CL-625-ONT truck	0.40
any two axles or axles 1-3 of the CL-625-ONT truck	0.30
any three of more axles of the CL-625-ONT truck, except for axles 1-3	0.25

BRAKING FORCE

- Apply 15% of wheel load laterally as braking force

- Ignore CHBDC braking force component caused by lane load UDL

  - Represents smaller vehicles braking elsewhere on structure

  - Deck panels are narrow, so can be ignored

- Multi-lane reduction factors not necessary

  - Already built into formula

- More research is necessary to validate this approach

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.
  - Bridge span = 18.000 m
  - Girders overhang the centre line of abutment bearings = 0.272 m
  - Total deck length = 18.544
  - Braking force is thus:

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

- The heaviest wheel load of the CL-625-ONT truck occurs at Axle 4 = 87.5 kN
  - Max discrete wheel braking load is therefore:

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

LOADS & IMPOSED DEFORMATIONS

- Self-Weight $\qquad (\gamma_{wood} = 6.0 \text{ kN/m}^3)$

- Superimposed Dead Load
  - Wearing surface $\qquad (\gamma_{asphalt} = 23.5 \text{ kN/m}^3)$
  - Barriers $\qquad (\omega_{barrier} = 1.3 \text{ kN/m})$

- Live Load
  - Vertical CL-625-ONT loading
  - Braking forces

- Wind Loads
  - Vertical wind on structure
  - Horizontal wind on structure
  - Horizontal wind on live load

Vertical Wind Load

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

- Considered to act both upwards and downwards
- Two applications considered for both upward and downward wind
  - Uniform load acting over the entire bridge plan area
  - Eccentric wind load with the centroid acting at the windward quarter-point

Horizontal Wind Load

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_v	=	1860	Pa	horizontal wind load per unit exposed plan area

- Resulting unfactored uniformly distributed load due wind acting on the railing:

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

Horizontal Wind Load

- The unfactored uniformly distributed load due to wind acting on the deck is

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

- The resulting unfactored uniformly distributed load due to wind acting on an exterior girder is

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

Horizontal Wind Load on Live Load

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	=	1.2		horizontal wind load coefficient
F_v	=	1116	Pa	horizontal wind load per unit exposed plan area

- Total exposed area for wind on live load (excluding exposed frontal area of a timber railing) is:

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

- Unfactored uniformly distributed load to wind on live load is:

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

LOADS & IMPOSED DEFORMATIONS

- Self-Weight $(\gamma_{wood} = 6.0 \text{ kN/m}^3)$

- Superimposed Dead Load
  - Wearing surface $(\gamma_{asphalt} = 23.5 \text{ kN/m}^3)$
  - Barriers $(\omega_{barrier} = 1.3 \text{ kN/m})$

- Live Load
  - Vertical CL-625-ONT loading
  - Braking forces

- Wind Loads
  - Vertical wind on structure
  - Horizontal wind on structure
  - Horizontal wind on live load

- Imposed Deformations
  - Thermal Effects
  - Shrinkage/Swelling due to changes in moisture content

LOAD COMBINATIONS

Loads	Permanent loads			Transitory loads						Exceptional loads			
	D	E	P	L^*	K	W	V	S	EQ	F	A	H
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	Table 3.2	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	Table 3.2	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	Table 3.2	1.00	0.45§	0.45	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.40§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.75§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

(Image courtesy of CSA Group)

LOAD COMBINATIONS

- SLS 1
  - Deflections
- SLS 2
  - Vibrations
- ULS 1
  - Worst case vertical loads
- ULS 3 & 4
  - Lateral loads and uplift due to wind
- FLS 1
  - Check metal connectors
  - No need to check wood components for fatigue
- Check other load combinations as necessary
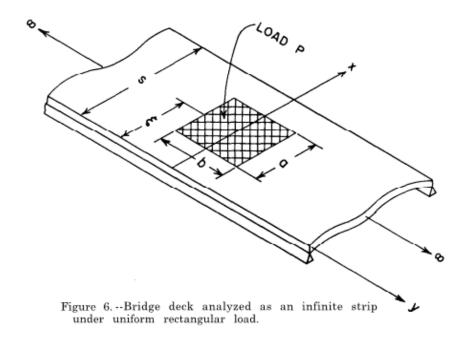 - Earthquakes (ULS 5), stream/ice pressure (ULS 6), etc.

Design Example

- Design Considerations

- Proposed Concept

- Material Properties

- Loading

- **Glulam Deck Panels**

- Glulam Girder Beams

STRUCTURAL ANALYSIS – DECK PANELS

- Uniform loads (i.e. self-weight, SDL, & wind) are easy to address

- <u>Entire deck panel width not effective in resisting discrete wheel loads</u>

- Original research by McCutcheon & Tuomi at US FPL in 1970's yielded design equations for live load shear and moment per metre deck width

Figure 6.--Bridge deck analyzed as an infinite strip under uniform rectangular load.

Table 1.—Deck design equations [3]
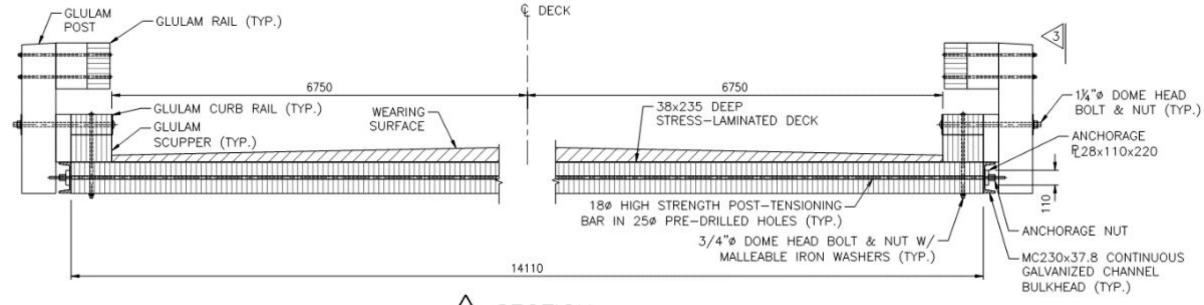
Unit moments, shears, and deflection	Computation equations for AASHO truck loads		
	H10	H15	H20
Primary moment, M_x (in.-lb/in.)	$P[(0.51 \log s) - 0.44]$	$P[(0.51 \log s) - 0.47]$	$P[(0.51 \log s) - 0.51]$
Primary shear, R_x (lb/in.)	0.034P		

(Left image from "Procedure for Design of Glued-Laminated Orthotropic Bridge Decks", 1973)

(Above image from "Simplified Design Procedure for Glued-Laminated Bridge Decks", 1974)

STRUCTURAL ANALYSIS – DECK PANELS

- MTO research in 1970's focused on the development of "stress-laminated timber decks"

- No research on glulam deck panels

STRUCTURAL ANALYSIS – DECK PANELS

- MTO research by Bakht in late 1980's validated 1970's research, but found it unconservative for wheels placed near deck panel edges
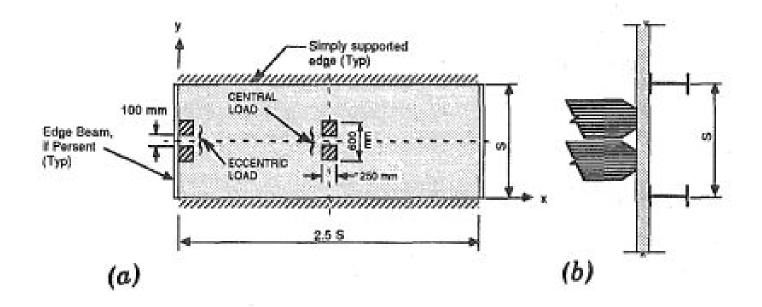
FIG. 16. Idealization of Prestressed or Glued Laminated Decks: (a) Plan of Idealized Orthotropic Plate; (b) Cross Section of Actual Bridge
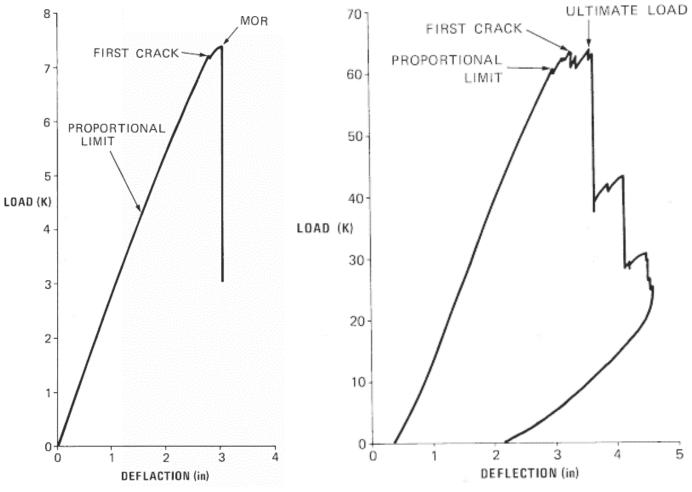
1566

(Image from "Load Distribution in Laminated Timber Decks" by Baidar Bakht, 1988)

STRUCTURAL ANALYSIS – DECK PANELS

- Bakht developed design curves to address his findings

- Bakht's curves have been developed into the equations presented in CHBDC clause 5.7.3.2

  Effective Width = 0.30 m + 0.14 x girder spacing   (*no edge stiffening*)

- CHBDC equations equally applicable to glulam decks as they are stress-laminated decks

STRUCTURAL ANALYSIS – DECK PANELS

- AASHTO LFRD Bridge Design Specifications (AASHTO) introduced equations in 1994 for same purpose

- Bakht's and AASHTO's equations yield significantly different results

- Why the big difference?

- AASHTO's equations based on findings of Sexsmith et al. (1979)

STRUCTURAL ANALYSIS – DECK PANELS

Single Lamination

Laminated Deck

- Sexsmith et al. found that wood softens slightly before failure, resulting in load sharing for systems of adjacent members (i.e. plastic redistribution)

- CHBDC & O86 load-sharing factor accounts for Sexsmith's findings

- AASHTO does not have a load-sharing factor

(Images from "Load Sharing in Vertically Laminated Post-Tensioned Bridge Decking" by Sexsmith et al., 1979)

STRUCTURAL ANALYSIS – DECK PANELS

- AASHTO accounts for plasticity in analysis (i.e. demand)

- Canadian codes account for plasticity in resistance equations

- Potentially unconservative to use AASHTO effective deck panel width equations with CHBDC & O86 resistances

- Use Bakht's CHBDC analysis equations with CHBDC resistances

STRUCTURAL ANALYSIS – DECK PANELS

- Deck panels analyzed as continuous beams on rigid vertical supports

- Apply permanent loads and wind as distributed loads

- Apply wheel loads as short UDLs (less conservative than point loads)

- Move wheel loads along deck panels while respecting truck envelopes and design lane widths

- Don't forget to check for uplift (wood is light!)

Frame element representing deck panel

Rigid vertical support representing girder

MEMBER DESIGN – GLULAM DECK PANELS

- Limit states for design:

  - SLS 1
    - Live load deflection without DLA < span/400
      - CHBDC clause 9.4.2

    - Differential deflections < 1.3 mm (0.05 in)
      - Limits chance of asphalt cracking
      - Good practice, but not a code check

  - ULS
    - Bending strength
      - CHBDC clause 9.6.1
    - Design for shear not necessary per CHBDC clause 9.7.5

MEMBER DESIGN – GLULAM DECK PANELS

- Effective strip width of a deck panel:
$$b_{eff} = 0.30 + 0.14S = 0.30 + 0.14 \times 1.150 \, m = 0.461 \, m$$

- Unfactored self weight:
$$\omega_{deck} = 0.215 \, m \times 0.461 \, m \times 6 \frac{kN}{m^3} = 0.59 \, kN/m$$

- Min and max ULS load factors for the deck are 0.90 and 1.20

- Unfactored trapezoidal superimposed dead load from wearing surface
$$\omega_{ws,min} = 0.050 \, m \times 0.461 \, m \times 23.5 \frac{kN}{m^3} = 0.55 \frac{kN}{m}$$

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

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

MEMBER DESIGN – GLULAM DECK PANELS

- Railing unfactored point load:

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

- Min and max ULS load factors for the timber railings are 0.90 and 1.20
- Unfactored uniformly distributed wind load:

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

- Eccentric wind load pressures acting in a trapezoidal distribution are:

$$\pm 2.5 \times F_v = \pm 2.5 \times 930 \, Pa \times \frac{kPa}{1000 \, Pa} \times 0.461 \, m = \pm 1.08 \frac{kN}{m}$$

$$\pm 0.5 \times F_v = \pm 0.5 \times 930 \, Pa \times \frac{kPa}{1000 \, Pa} \times 0.461 \, m = \pm 0.22 \frac{kN}{m}$$

MEMBER DESIGN – GLULAM DECK PANELS

- Live load applied to deck design strip for the heaviest transverse line of wheels (axle 4)
- Wheel loads positioned transversely within the design lanes for worst load effect
- Both two and three lanes are considered
- Two lane configuration
  - Each single load lane
  - Both loaded lanes
- Multi-lane reduction factors
  - 1.0 for one loaded lane
  - 0.9 for two loaded lanes
  - 0.8 for 3 loaded lanes
- Analysis completed using computer structural analysis software

MEMBER DESIGN – GLULAM DECK PANELS

- Wheel loads amplified by the dynamic load allowance of 0.4. This values is reduced by 30% for dynamic qualities of wood bridges per CHBDC clause 3.8.4.5.4.

- Dynamic load allowance is:

  - $1 + 0.40 \times 0.70 = 1.28$

- All described load effects are applied to a continuous beam of 461 mm width that spans the full 14110 mm deck width

  - Girders assumed to act as rigid vertical supports

FLEXURAL DESIGN – GLULAM DECK PANELS

- Summary of factored bending moments on the deck design strip

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 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

$$M_{r,deck} = \phi F_b S K_L K_{Zb}$$

Where $F_b = f_b K_D K_{Sb} K_T K_m$

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

FLEXURAL DESIGN – GLULAM DECK PANELS

$$M_{r,deck} = \phi F_b S K_L K_{Zb}$$

Where $F_b = f_b K_D K_{Sb} K_T K_m$

- K_D per CHBDC clause 9.5.3 = 1.0
- K_{sb} per CHBDC Table 9.2 = 0.84
- K_T per CHBDC Table 9.6 = 0.85
- K_m per CHBDC clause 9.5.6 = 1.36
- f_b per CHBDC Table 9.16 = 11.8 MPa

$$F_b = 11.8 \; MPa \; x \; 1.0 \; x \; 0.84 \; x \; 0.85 \; x \; 1.36 = 11.5 \; MPa$$

FLEXURAL DESIGN – GLULAM DECK PANELS

$$M_{r,deck} = \phi F_b S K_L K_{Zb}$$

- K_L per CHBDC clause 9.6.3 = 1.0
- K_{Zb} per CHBDC Table 9.7 = 1.14
- $S = \dfrac{bd^2}{6} = \dfrac{461\ mm \times (215\ mm)^2}{6} = 3552 \times 10^3\ mm^3$

- $M_{r,deck} = 0.9\ x\ 11.5\ MPa\ x\ 3552 \times 10^3\ mm^3 x\ 1.0\ x\ 1.14 = 41\ MPa$

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

LIVE LOAD DEFLECTIONS – GLULAM DECK PANELS

- Live load under SLS 1 load combination limited to a maximum of 1/400th the span per CHBDC clause 9.4.2

- Limiting values 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 are:

$$\Delta_{SLS,ext} = 1.3 \; mm < \Delta_{max,ext} = 1.8 \; mm$$
$$\Delta_{SLS,int} = 1.1 \; mm < \Delta_{max,int} = 2.9 \; mm$$
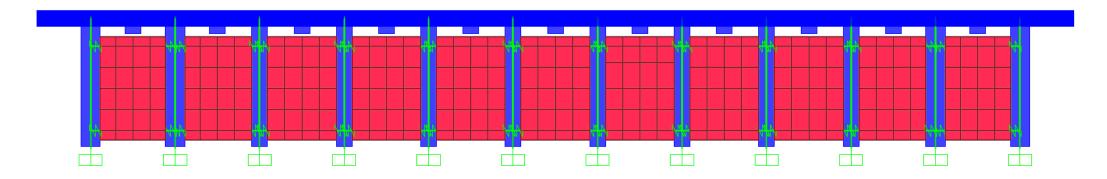
Design Example

- Design Considerations

- Proposed Concept

- Material Properties

- Loading

- Glulam Deck Panels

- **Glulam Girder Beams**
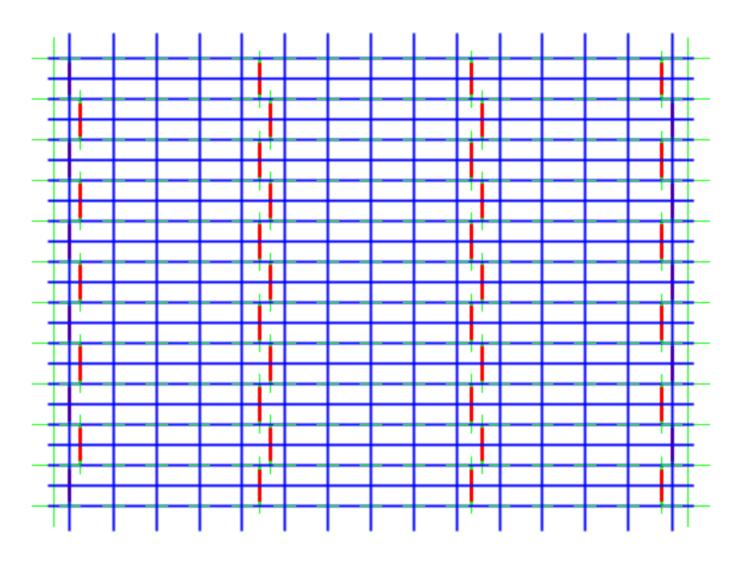
STRUCTURAL ANALYSIS - GIRDERS

- Girders can be analyzed by CHBDC simplified method or by computer structural analysis

- Both simplified method and computer structural analysis used in design example for comparison purposes

- Simplified method generally more conservative

  - Useful for preliminary design

  - Computer structural analysis preferred for detailed design

STRUCTURAL ANALYSIS
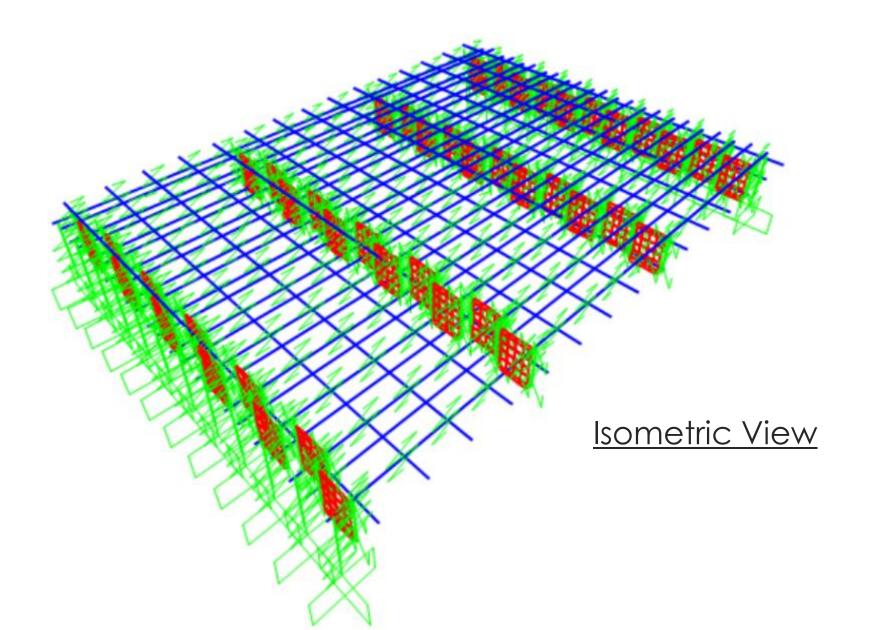
- Computer structural analysis was performed using CSi Bridge

- Girders, stiffener beams, and deck panels modeled as frame elements

- Diaphragms modeled as shell elements

- Connections and bearings modeled as link elements

Cross-Section View

STRUCTURAL ANALYSIS

Plan View

STRUCTURAL ANALYSIS

Isometric View

STRUCTURAL ANALYSIS

Self-Weight Deflection

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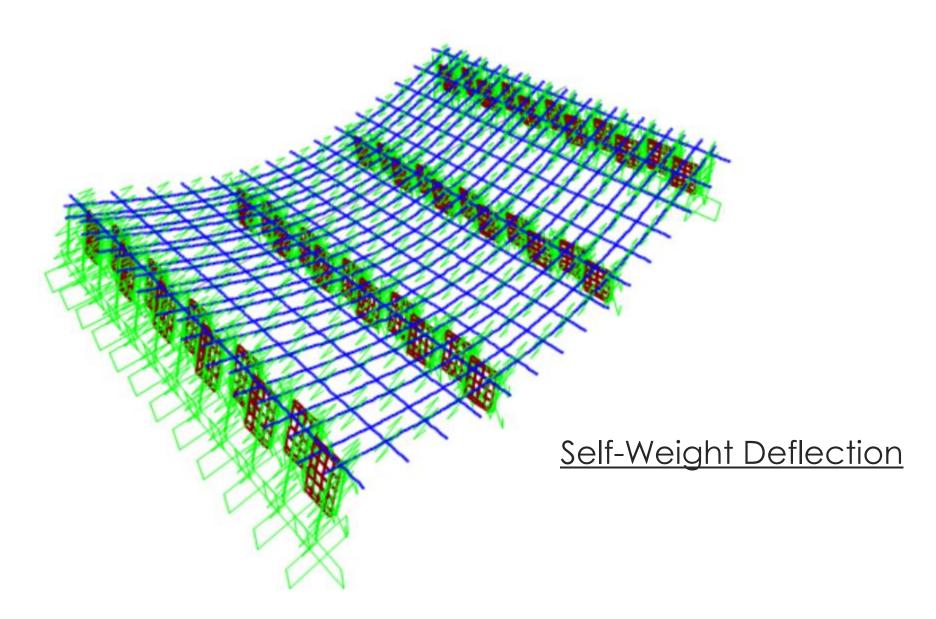
- The interior and exterior girders have a depth of 1634 mm, a width of 215 mm, and a unit weight of 6 kN/m3, resulting in unfactored linear weights of

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

- 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.

SIMPLIFIED METHOD- GIRDERS

- 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

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

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

SIMPLIFIED METHOD- GIRDERS

- The use of the CHBDC simplified method of analysis relied upon satisfying the follow 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

- Therefore, is is acceptable to use the simplified method for analysis of the girders

SIMPLIFIED METHOD- GIRDERS

- From Table 3.5 of the CHBDC both 2 and 3 lane design lanes are to be considered for the travelled width of 13.500 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.

SIMPLIFIED METHOD- GIRDERS

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 =12.

$$V_L = F_T F_S V_T$$

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

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

SIMPLIFIED METHOD- GIRDERS

Recall that the truck fraction for bending moment at FLS can be used to approximate the SLS live load deflection in a girder.

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

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

SIMPLIFIED METHOD- GIRDERS

- 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 :

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

SIMPLIFIED METHOD- GIRDERS

- 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:

$$M_T = 1505\ kNm \times (1 + 0.25 \times 0.70) = 1505\ kNm \times 1.175 = 1769\ kNm$$
$$V_T = 394\ kN \times (1 + 0.25 \times 0.70) = 394\ kN \times 1.175 = 463\ kN$$
$$\Delta_T = 72\ mm \times (1 + 0.25 \times 0.70) = 72\ mm \times 1.175 = 85\ mm$$

SIMPLIFIED METHOD- GIRDERS

- The resulting maximum bending moment of interior and exterior girders due to CL-625-ONT live loading, $M_{L,int}$ and $M_{L,ext}$, respectively, are:

$$M_{L,int} = 0.340 \times 1.0 \times 1769 \ kNm = 602 \ kNm$$
$$M_{L,ext} = 0.363 \times 1.0 \times 1769 \ kNm = 642 \ kNm$$

- The resulting maximum shear force of interior and exterior girders due to CL-625-ONT live loading, $V_{L,int}$ and $V_{L,ext}$, respectively, are:

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

SIMPLIFIED METHOD- GIRDERS

- The maximum live load deflection at interior and exterior girders due to CL-625-ONT live loading, $\Delta_{L,int}$ and $\Delta_{L,ext}$, respectively, are:

$$\Delta_{L,int} = 0.320 \times 1.0 \times 72 \; mm = 23 \; mm$$
$$\Delta_{L,ext} = 0.282 \times 1.0 \times 72 \; mm = 21 \; mm$$

STRUCTURAL ANALYSIS - GIRDERS

- Comparison of simplified method and computer structural analysis 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

STRUCTURAL ANALYSIS - GIRDERS

- Comparison of simplified method and computer structural analysis results:

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Δ_{LL}	16 mm	13 mm	18 mm	14 mm
$\Delta_{PERMANENT}$	11 mm	13 mm	17 mm	17 mm

- Exterior girder bending moments similar for both analysis methods

STRUCTURAL ANALYSIS - GIRDERS

- Comparison of simplified method and computer structural analysis 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

- Interior girder bending moments overestimated by simplified method

STRUCTURAL ANALYSIS - GIRDERS

- Comparison of simplified method and computer structural analysis 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

- Shear forces and live load deflections overestimated by simplified method

- Computer structural analysis preferred for detailed design

SHEAR LOAD

- Shear failures in glulam follow a horizontal slip plane

- Shear resistance is a function of member volume and loading pattern (aka "shear load")

- CHBDC requires that factored shear resistance exceed the "shear load"

$$V_f = 0.82 \left[\frac{1}{L} \int_0^L |V(x)|^5 \, dx \right]^{0.2}$$

- All loads must be applied simultaneously (i.e. no superposition)

- Critical live load position for maximum shear load is generally not the same as for maximum vertical shear

SHEAR LOAD

- Shear load calculation best performed on isolated girder as follows:

  1. Apply permanent loads at discrete locations, say 10th-points

  2. Determine the truck fraction for the girder (i.e. the percentage of one lane of CL-625 loading carried by that girder)

  3. Use multi-step live load analysis to move live load along girder at one metre increments

  4. Calculate the shear load for each live load increment

  5. The shear load for design is the maximum shear load

- Benefit to applying only point loads is the shear load integral reduces to a summation of a step-wise shear force diagram

SHEAR LOAD

- The calculated shear loads were determined to be as follows:

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

- Although cumbersome, calculating the factored shear load provides substantial design economy relative to using the factored vertical shear

MEMBER DESIGN – GLULAM GIRDERS

- Lateral stability factor accounts for lateral-torsional buckling
  - Unbraced length taken as centre-to-centre spacing of diaphragms

- Load sharing factor taken as unity
  - Some designers treat as sawn timber stringer for this calculation
    - Likely unconservative benefit because sawn timbers have more defects than glulam, and thus more to gain from presence of multiple members in a system

- Reduction of section modulus due to butt joints not necessary
  - Individual laminations are finger-jointed and glued together to form continuous lamination

FLEXURAL DESIGN – GLULAM GIRDERS

- The flexural resistance of a glued-laminated girder, $M_{r,girder}$, per CHBDC clause 9.6.1, is the lesser of:

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

- $\phi = 0.9$
- $K_d = 1.0$.

FLEXURAL DESIGN – GLULAM GIRDERS

- k_{ls}, requires explicit calculation because the depth-to-width ratio of the girders exceeds 1.0. The slenderness factor is:

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

- 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, where:

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

FLEXURAL DESIGN – GLULAM GIRDERS

- Accordingly, the lateral stability factor is determined from CHBDC Table 9.5:

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

- 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.

FLEXURAL DESIGN – GLULAM GIRDERS

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

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

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

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

- The section modulus of the girder is

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

FLEXURAL DESIGN – GLULAM GIRDERS

- The resulting factored flexural resistance of an exterior girder 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

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

$$M_{r,girder,ext} = 0.9 \times 1.0 \times 1.0 \times 0.79 \times 23.0 \ MPa \times 117923 \times 10^3 \ mm^3$$
$$= 1928 \ kNm$$

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

FLEXURAL DESIGN – GLULAM GIRDERS

- 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.

SHEAR DESIGN – GLULAM GIRDERS

- The shear resistance of a girder, $V_{r,girder}$, per CHBDC clause 9.6.1:

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

- Shear resistance of 265 mm x 1 634 mm exterior girder.

- $\emptyset = 0.9$

- $K_d = 1.0$ and

- $K_m = 1.0$

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

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

SHEAR DESIGN – GLULAM GIRDERS

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

- The resulting factored shear resistance is:

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

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

- The factored shear load is:
$$V_{f,girder,ext} = 168\ kNm < V_{r,girder,ext} = 211\ kN$$
$$V_{f,girder,int} = 168\ kNm < V_{r,girder,int} = 178\ kN$$
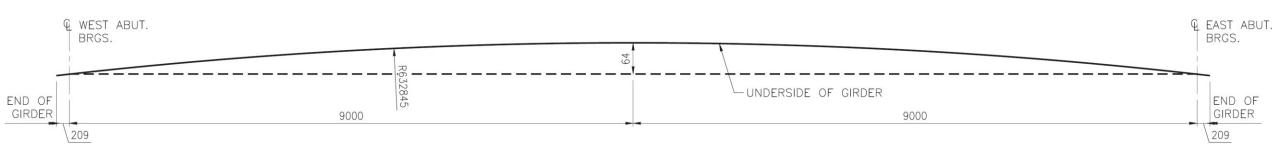
- Therefore, the girders have sufficient shear capacity.

MEMBER DESIGN – GLULAM GIRDERS

- Limit states for design:

  - SLS 1
    - Live load deflection without DLA < span/400
      - CHBDC clause 9.4.2
      - Deflection limit = 18000 mm/400 = 45 mm
      - Exterior girder deflection = 12.7 mm < 45 mm
      - Interior girder deflection = 13.3 mm < 45 mm
      - Therefore, superstructure deflections under live load have been addressed

  - SLS 2
    - Vibration check
      - CHBDC clause 3.4.4

MEMBER DESIGN – GLULAM GIRDERS

- Camber girders for 1/600th the span + twice the unfactored permanent load deflection

  - CHBDC clause 9.12.4
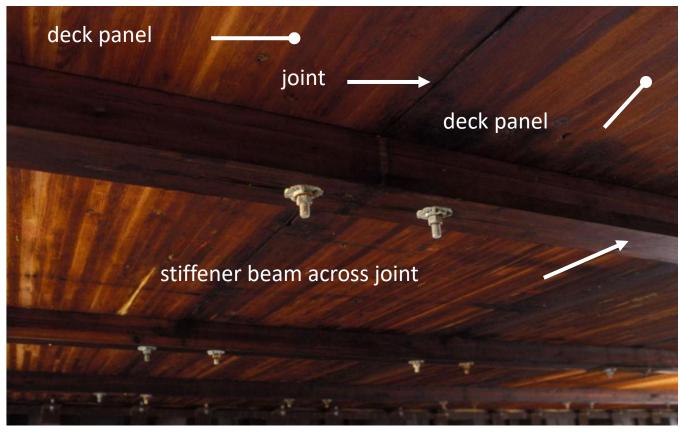
  - Accounts for long-term creep deflection

TYPICAL GIRDER CAMBER DIAGRAM
NTS

Design Example

- **Stiffener Beams**

- Glued-laminated timber diaphragms

- Connection Design

STRUCTURAL ANALYSIS – STIFFENER BEAMS

- Deck panels span predominantly one-way between girders

- There does exist some two-way action under wheel loads

- Stiffener beams transfer the secondary moment and shear caused by two-way action between adjacent deck panels

- Secondary moment and shear can be found using the "Simplified Design Procedure for Glued-Laminated Bridge Decks" document by McCutcheon & Tuomi (1974)

(Photograph courtesy of Mike Ritter)

STRUCTURAL ANALYSIS – STIFFENER BEAMS

- The factored moment, $M_{f,stiffener}$, and the factored shear, $V_{f,stiffener}$, that are to be transmitted by the stiffener beam are:

$$M_{f,stiffener} = 4.8 \ kNm$$
$$V_{f,stiffener} = 51.7 \ kN$$

- The maximum fastener axial force, T_f, is:

$$T_f = 55 \ kN$$

- The shear load in the stiffener beam is:

$$V_f = 33 \ kN$$

MEMBER DESIGN– STIFFENER BEAMS

- The factored moment resistance, and factored shear resistance of a stiffener beam are:

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

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

- Therefore, the stiffener beams have adequate capacity.

Design Example

- Stiffener Beams

- **Glued-laminated timber diaphragms**

- Connection Design

STRUCTURAL ANALYSIS – GLULAM DIAPHRAGMS

- Glulam timber diaphragms brace the girders against lateral-torsional buckling

- Maintain the relative spacing of the girders

- Transmit lateral load between girders
  - at midspan for spans < 12 m and
  - Third-points for spans >= 12 m

- Abutment diaphragms in the design are offset 600 mm forward from the bearings to not interfere with the bearing assemblies

- Diaphragms should be as deep as possible (per CHBDC clause 9.20.2)
  - Design example detailed to provide a 25 mm gap between diaphragms and stiffener beams for air circulation

- Tie roads are located a 222 mm from the underside of the girder

STRUCTURAL ANALYSIS – GLULAM DIAPHRAGMS

- 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

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

- The maximum factored force experienced by a tie rod due to external loads is 46 kN in tension. The maximum factored tie rod force, considering both forces arising from external loads and bracing of the girders is

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

STRUCTURAL ANALYSIS – GLULAM DIAPHRAGMS

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

MEMBER DESIGN– GLULAM DIAPHRAGMS

- Similar to the flexural resistance for girder design, the flexural resistance of the diaphragms is taken as the lesser of

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

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

Where:

- $\emptyset = 0.9$

- $k_d = 1.0$

- $k_m = 1.0$

- $\dfrac{d}{b} = \dfrac{1406 \, mm}{130 \, mm} = 10.82 < 10$

MEMBER DESIGN– GLULAM DIAPHRAGMS

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

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

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

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

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

MEMBER DESIGN– GLULAM DIAPHRAGMS

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

$$\sigma_{r,diaphragm} = 0.9 \times 1.0 \times 0.97 \times 1.0 \times 23.0 \; MPa = 20.1 \; MPa$$

and

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

$$\sigma_{r,diaphragm} = 0.9 \times 1.0 \times 1.0 \times 1.15 \times 23.0 \; MPa = 23.8 \; MPa$$

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

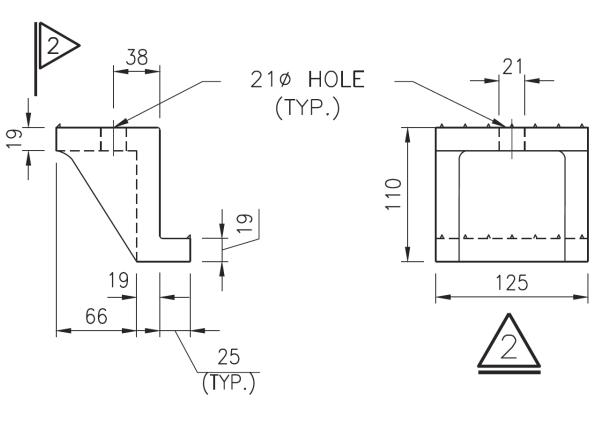- Therefore, the diaphragms have sufficient capacity.

Design Example

- Stiffener Beams

- Glued-laminated timber diaphragms

- **Connection Design**

CONNECTION DESIGN

- 4 Major connections for this bridge design
  - Deck to Girder connection
  - Deck to Stiffener beam connection
  - Diaphragm connection
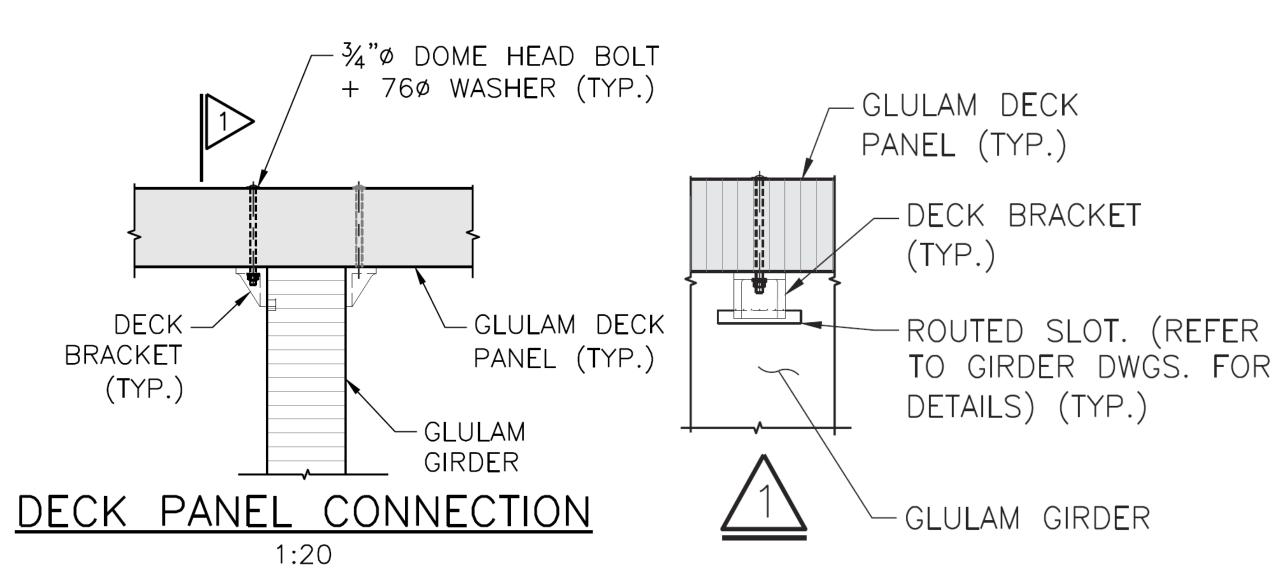 - Girder bearing connection

CONNECTION DESIGN – DECK-TO-GIRDER

(Photograph courtesy of Laminated Concepts Inc.)

CONNECTION DESIGN – DECK-TO-GIRDER

¾"∅ DOME HEAD BOLT + 76∅ WASHER (TYP.)

DECK BRACKET (TYP.)

GLULAM DECK PANEL (TYP.)

GLULAM GIRDER

DECK PANEL CONNECTION
1:20

GLULAM DECK PANEL (TYP.)

DECK BRACKET (TYP.)

ROUTED SLOT. (REFER TO GIRDER DWGS. FOR DETAILS) (TYP.)

GLULAM GIRDER

CONNECTION DESIGN – DECK-TO-GIRDER

- Downward forces are resisted by direct bearing between deck panels and tops of girders

- Upward forces are resisted by tension in the through-bolts and direct bearing of aluminum deck clips

- Transverse forces resisted by shear in the through bolts and direct bearing of the aluminum deck clips

- Longitudinal loads are resisted by shear in the lag screws

CONNECTION DESIGN – DECK-TO-GIRDER

- Only longitudinal force is the braking force

- Recall $F_{br} = 87.5\ kN \times \dfrac{180\ kN}{2 \times 625\ kN} = 87.5\ kN \times 0.144 = 12.6\ kN$

- Multiply by 1.7 for ULS 1 $= 12.6\ kNx1.7 = 21.4\ kN$

- See next slide

CONNECTION DESIGN – DECK-TO-GIRDER

Fx (main perp, side parallel) Qr3

- Use 8-19Ø Lag screws @150 OC

SPF

- $P_r = 7.63 \; x \; 7.0 \; x \; 0.67 = 24.8kN$
- $Q_r = 6.21 \; x \; 7.0 \; x \; 0.67 = 29.1kN > F_x = 21.4 \; kN$ (74%) [note typo p.130]
- $A_m = 1200 \; x \; 215 = 258,00mm^2$
- $A_s = 1558 \; x \; 215 = 334,970mm^2$
- $\frac{A_m}{A_s} = 0.77$
- $n_{Fe} = \frac{1}{2}(6.8 + 7.28) = 7.0$

Note: this ignores friction between deck and girder

CONNECTION DESIGN – DECK-TO-GIRDER

Uplift on lag screws deck-to-girder connection

- From analysis, deck uplift is Fz=34 kN per glulam deck plank over each girder
  - Use 19Ø Lag screws with 150 mm embedment down into the girder
  - Lag screw length = 215 +150 = 365 mm

- $P'_{rw} = 84 \; x \; 141 \; x \; 5 \; lags \; x \; (1.0 x 0.67 x 1.0 x 1.0) \; x \; 10^{-3} = 39.6 kN > 34 kN \; (F_z \; 86\%)$

- $L_t = 152 - 11.1 = 141 mm$

CONNECTION DESIGN – DECK-TO-GIRDER

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

4-19Ø Rods A307

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

75Ø Washers

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

Clip in bolted slot

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

Slot Shear

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

CONNECTION DESIGN – DECK-TO-GIRDER

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

F$_x$DOWN

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

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

$Q_r = 0.8 \ x \ 5.8 \ x \ (1.0 \ x \ 0.67 \ x \ 1.0) x (215 \ x \ 1200) x 1.0 x 1.0 x 10^{-3} = 732 kN > T_f = 8 \ kN$ (1%)

CONNECTION DESIGN – DECK-TO-GIRDER

Where,

- $\phi = 0.8$

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

- $f_{cp} = 5.8 MPa$

- $K_D = 1.0$

- $K_{S_{cp}} = 0.67 \ (Wet \ service \ condition \ at \ connection)$

- $K_T = 1.0$

- $K_B = 1.0$

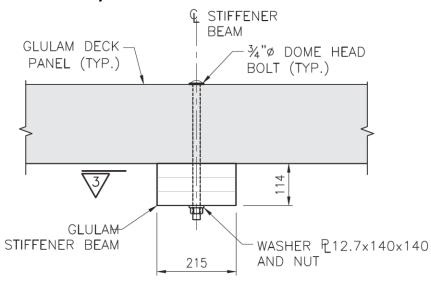- $K_{z_{cp}} = 1.0$

CONNECTION DESIGN – DECK-TO-STIFFENER

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

- 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

- Stiffener beam moments and shears determined per "Simplified Design Procedure for Glued-Laminated Bridge Decks" by McCatcheon and Taomi

- Strength design per the CHBDC

TYPICAL DECK–TO–STIFFENER BEAM CONNECTION

CONNECTION DESIGN – DECK-TO-STIFFENER

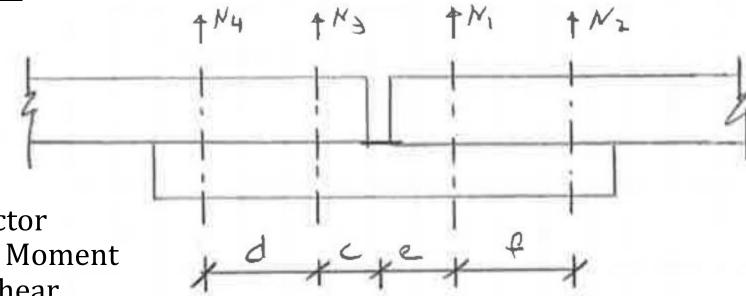## 1) Longitudinal Moment and Shear to be transferred by Stiffener Beams

$$P = 87.5kN = 19646lb \ (Axle \ 4 \ wheel \ load)$$
$$s = 1150mm = 45.3in \ (girder \ spacing)$$

$$R_y = \frac{6Ps}{1000} = 5337lb = 23.8kN \ (Live \ load \ longitudinal \ shear \ to \ be \ transferred)$$

$$M_y = \frac{Ps}{1600}(s - 10) = 19611lb.in$$

$$= 2.2kNm \ (Live \ load \ longitudinal \ moment \ to \ be \ transfered)$$

CONNECTION DESIGN – DECK-TO-STIFFENER

2) Stiffener Beam Forces

αLL=1.7 ULS Live load factor
Mf=$\alpha LL My$=3.8kNm ULS Moment
Vf=$\alpha LL Ry$=40.4kN ULS Shear
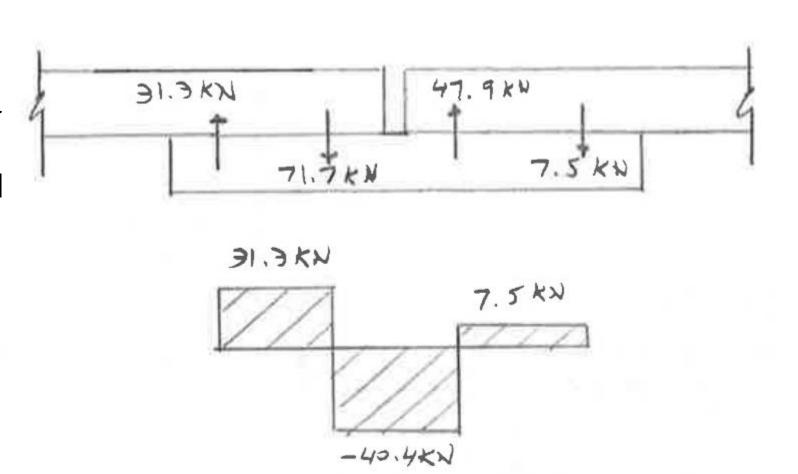
d=0.317m
c=0.152m
e=0.152m
f=0.317m

CONNECTION DESIGN – DECK-TO-STIFFENER

3) Shear in Stiffener beam

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

- $d + c + e + f = 0.938m$

CONNECTION DESIGN – DECK-TO-STIFFENER

3) Shear in Stiffener Beam

$$V_f = 0.82 \left[\frac{1}{0.938m} x \{|31.3|^5 x 0.317 + |-40.4|^5 x 0.304 + |7.5kN|^5 x 0.317 \} \right]^{0.2}$$
$$= 28kN$$

$$V_r = \phi k_d k_m k_{sv} f_{vu} A / 1.5$$
$$V_r = 51.3kN > V_f = 28kN \therefore OK$$

Where,

- $\phi = 0.90$
- $k_d = 1.0$
- $k_m = 1.0$
- $k_{sv} = V^{-0.18} = (0.215 x 0.114 x 0.938)^{-0.18} = 1.97$
- $f_{vu} = 1.18 \, MPa$
- $A = 215 x 114 = 24510 mm^2$
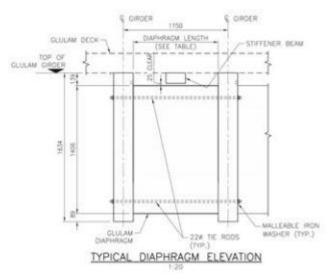
CONNECTION DESIGN – DIAPHRAGM-TO-GIRDER

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.

Rods

$$T_f = 73kN$$

Use 22Ø A307 rod with threaded ends

$$T_r = 80.7kN > T_f = 73kN \ OK \ (91\%)$$
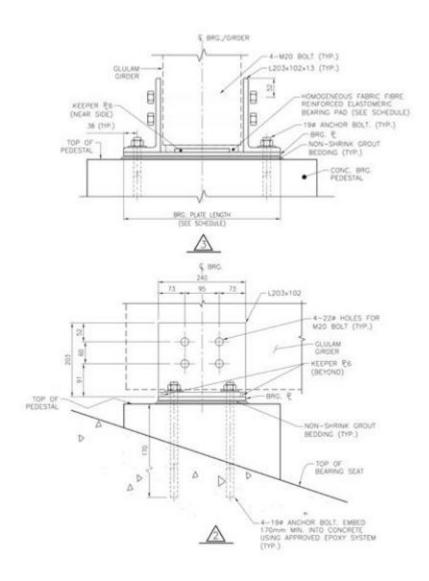
Washer: 170x170x12.7

$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$
$$Q_r = 0.8x5.8x(1.0x0.67x1.0)x(170x170)x1.0x1.0x10^{-3}$$
$$= 90kN > T_f = 73kN \ OK \ (82\%)$$

GIRDER
150
GIRDER

GLULAM DECK
DIAPHRAGM LENGTH
(SEE TABLE)
STIFFENER BEAM

TOP OF
GLULAM GIRDER

GLULAM
DIAPHRAGM
22∅ TIE RODS
(TYP.)
MALLEABLE IRON
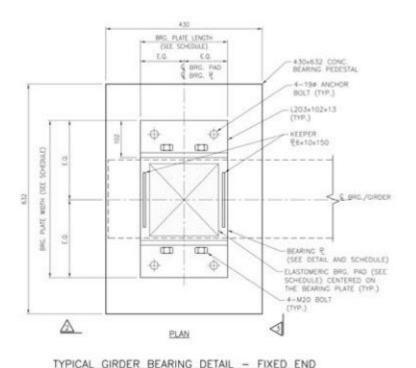WASHER (TYP.)

TYPICAL DIAPHRAGM ELEVATION
1:20

CONNECTION DESIGN – DIAPHRAGM-TO-GIRDER

Where,

- $\phi = 0.8$

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

- $f_{cp} = 5.8 MPa$

- $K_D = 1.0$

- $K_{S_{cp}} = 0.67 \ (Wet \ service \ condition \ at \ connection)$

- $K_T = 1.0$

- $K_B = 1.0$

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

NOTE: Washer plate thickness shall be checked.

CONNECTION DESIGN – GIRDER-TO-BEARING

TYPICAL GIRDER BEARING DETAIL – FIXED END

CONNECTION DESIGN – GIRDER-TO-BEARING

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,

- $\phi = 0.8$

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

- $f_{cp} = 5.8MPa$

- $K_D = K_T = K_B = K_{Z_{cp}} = 1.0$

- $K_{S_{cp}} = 0.67 \; (Wet \; service \; condition \; at \; connection)$

CONNECTION DESIGN – GIRDER-TO-BEARING

Bearing (Clause 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,

- $\phi = 0.8$

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

- $f_{cp} = 5.8 MPa$

- $K_D = K_T = K_B = 1.0$

- $K_{S_{cp}} = 0.67 \ (Wet \ service \ condition \ at \ connection)$

- $K_{z_{cp}} = 1.15 \ (ratio = \dfrac{215}{38} = 5.65 > 2)$

CONNECTION DESIGN – GIRDER-TO-BEARING

Bolts in Glued-laminated Timber (Clause 10 O86-01)

- $P_f = 87kN$

- $Q_f = 9kN$

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

- $K_{SF} = 0.67 \ (Wet \ service)$

- $K_{SV} = 0.87$

- $K_{st} = 0.75$

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

CONNECTION DESIGN – GIRDER-TO-BEARING

Bolts in Glued-laminated Timber (Clause 10 O86-01)

S-W-S (STEEL SIDE PLATES)

TRY 2 ROWS OF 2 BOLTS, 19∅, A307:

- $P_r = 104kN > F_x = 87kN \ (Brittle \ resistance) \ OK$
- $Q_{RS} = 17kN > F_z^{UP} = 9kN \ (Splitting \ resistance) \ OK$
- $N_r = 114kN > N_f^{5.9°} = 87.5kN \ (Yeilding \ resistance) \ OK$
- $N_r = 103kN > N_f = 87.5kN \ (Brittle \ resistance) \ OK$

CONNECTION DESIGN – GIRDER-TO-BEARING

Anchor bolts (Clause 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:

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

CONNECTION DESIGN – GIRDER-TO-BEARING

<u>Anchor bolts (Clause 10 O86-01)</u>

Eccentricity of F_X on anchors:

- $M_f = 87x0.14m = 12.2kN.m$

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

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

∴ Use larger bolt: Try 19 ∅

- $T_r = 55x0.91x0.88 = 44kN/BOLT > T_f = \dfrac{9}{4BOLTS} + 30 = 32kN/BOLT$

- $V_r = 107x0.91x0.56 = 54.5kN/BOLT > V_f = \dfrac{87}{4BOLTS} = 22kN/BOLT$

Unity Check:

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

CONNECTION DESIGN – GIRDER-TO-BEARING

Anchor bolts (Clause 10 O86-01)

 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: Check bearing plate thickness for gravity and uplift forces

CONNECTION DESIGN – GIRDER-TO-BEARING

Upper clips (Clause 10 O86-01)

Estimate side force:

- $M_f = 1308 kN.m$

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

- say 2% is torsional components for restraint:
$$C_f^b = 0.02 x 886 = 17.7 kN$$

- assume 200x100 bearing area on steel angle (6.5.9.2 O86-01):
$$Q_r = \phi F_{cp} A_b K_B K_{Z_{cp}}$$
$$Q_r = 0.8 x 5.8 x (1 x 0.67 x 1.0) x (200 x 100) x 1.0 x 1.0 x 10^{-3} = 62 kN > 17.7 kN \ OK$$

CONNECTION DESIGN – GIRDER-TO-BEARING

Where,

- $\phi = 0.8$

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

- $f_{cp} = 5.8 MPa$

- $K_D = K_T = K_B = K_{z_{cp}} = 1.0$

- $K_{S_{cp}} = 0.67 \ (Wet \ service \ condition \ at \ connection)$

Note: Diaphragms near support will also provide restraint. Additional checks for angles for bending & prying on anchor bolts required.

CONNECTION DESIGN – GIRDER-TO-BEARING

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,

- $\phi = 0.8$

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

- $f_{cp} = 5.8MPa$

- $K_D = K_T = K_B = K_{z_{cp}} = 1.0$

- $K_{S_{cp}} = 0.67 \ (Wet \ service \ condition \ at \ connection)$

Note: Check plate bending.

FABRICATION & CONSTRUCTION

- Glulam members fabricated in controlled environment

- Lightweight members are easy to transport to site

- Superstructure construction limited to erection of glulam members, fastening of connections, and waterproofing/paving deck

- Small cranes or excavators sufficient for erection of glulam members

- Minimal on-site construction time

NEXT STEPS

- Simple spans currently limited to ~20 metres

- Girder continuity and/or strengthening with post-tensioning or FRPs should afford increased span possibilities

- Design examples indicate topics that require more research

- 2019 CHBDC will include new clauses to address glulam deck panels