Design Description—Silver Spoon Bridges

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Abstract

This report describes the design of the replacement bridge that is planned for the Silver Spoon trails in Deep River over the first crossing of Kennedy Creek on the "M" Loop. This bridge will be used for pedestrian traffic as well as for equipment necessary for trail maintenance. For ease of assembly and longevity, structural components are made of aluminum and are joined using stainless steel bolts. The deck is made of preserved wood.

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Design Description—Silver Spoon Bridges

1. BACKGROUND AND GENERAL REQUIREMENTS

As identified in red in Figure 1, the Silver Spoon “X” and “M” trails cross Kennedy Creek in four locations. The replacement bridge design described in this document is suitable for all four locations, with some adjustments of footings. However, this document applies specifically to the replacement for the existing Bishop’s Bridge at the M1 location, shown in Figure 2. While this is the only bridge that is expected to be required to support heavy equipment required for trail maintenance, it has not yet been decided whether to create a different design for the other locations or whether it would be simpler to use the same design for all four locations.

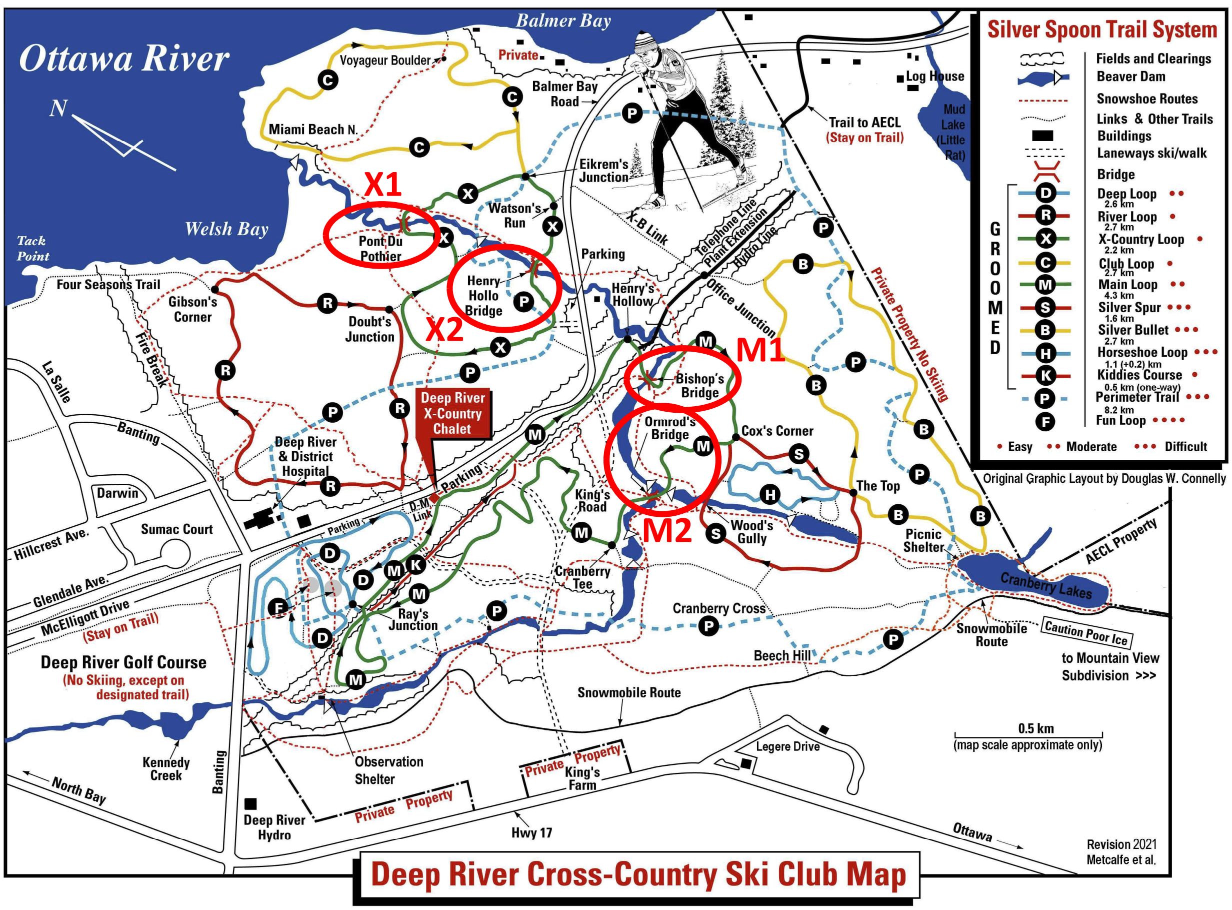


Figure 1. Four major bridges on the Silver Spoon Trails

Figure 2. M1 location: existing Bishop's Bridge as viewed from downstream (left photo); and from the north end (right photo)

2. DESIGN LOADS

- Span between footings: 35'2" ft or less.
- Self-weight: nominally 4,980 lb distributed uniformly along the length, roughly 41% aluminum and 54% wood.
- Snow and rain load: 56.4 lb/ft^2
- Winter live load: 900 lb snowmobile towing grooming equipment + driver
- Special summer live load: 7,000 lb tractor towing a 11,500 lb tandem-axle trailer, or a 10,000 lb tractor alone. This would occur only irregularly and would be subject to very controlled conditions.
- Maximum eccentricity of vehicle loading from the centreline of the bridge is 5".
- The location of ski trail grooming equipment and personnel are not restricted.
- Tractors considered were:
  - Massey Ferguson 1533 (wheel track 50" front, 55" rear) with a 1525 front loader. Wheelbase is 69", and total weight approximately 5,000 lb with empty bucket.
  - Kioti NX4510 HST (wheel track 53.2" front, 52.1" rear), wheelbase 75.6", weight approximately 6,400 lb, including cab, front loader and salt-filled tires.
- The tandem axle trailer considered has a wheel track of 46-1/8", with 33.5" between axles, and 10'6" between the rear axle of the Massey Ferguson 1533 tractor and the front axle of the trailer.
- Compact excavators considered were:
  - John Deere 17G, weighing 3,790 lb, has 9" wide adjustable tracks, which, when fully extended, are spaced 41" to their centrelines, and have a contact length of 48".

    - John Deere 26G, which weighs 6,110 lb, has 12” wide tracks spaced 47” to their centrelines, with a contact length of 60”.
- A 7,500 lb full-size pickup truck with 72” wheel track, 8” wide tires and 6,000 axle loading has also been considered. Compact pickup trucks with narrower wheel track are also acceptable.
- Bounding wheel load has been taken as 4,000 lb for some calculations.
- Generic live load used in design of pedestrian bridges: 90 psf. (Some codes specify 85 psf.)
- Fatigue is not explicitly considered because of limited high-load use. The primary concern with fatigue on this design is believed to be potential loosening of the fasteners in the wooden deck.
- Clearance above summer water level: approximately 18”. Required flow clearance has been assessed previously. Side loads due to water impact are not explicitly considered as there will be sufficient clearance for anticipated off-normal flows.
- Wind loads are not considered as the bridges are relatively close to the ground.

3. METHODOLOGY AND REFERENCE DOCUMENTS

Overall design was performed according to the allowable stress design (ASD) methodology described in *Aluminum Design Manual—Specifications and Guidelines for Aluminum Structures*, The Aluminum Association, 2005 (588 pages).

A companion book was also used to provide alternative explanations of the methodology described in the *Aluminum Design Manual*: J.R. Kissell and R.L. Ferry, *Aluminum Structures—A Guide to their Specification and Design*, Second Edition, 2002, John Wiley & Sons (546 pages).

General bridge design guidelines were obtained from *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway Transportation Officials, 2012. However, instead of the load and resistance factor design (LRFD) methodology used in this specification for assessing the acceptability of combined load cases, the simpler allowable stress design (ASD) method was used.

Basic beam stress formulas were obtained from *Roark’s Formulas for Stress and Strain*, Warren C Young and Richard G. Budynas, McGraw-Hill, Seventh Edition, 2002. Another good reference that includes more complex configurations is “Beam Formulas with Shear and Moment Diagrams,” The American Wood Council, 2007.

Some section properties and multi-support stresses were checked using online calculators:
https://webstructural.com/beam-designer.html#
https://calcresource.com/cross-section-angle.html

Beam bracing requirements were determined using the methodology described in J.A. Yura, “Fundamentals of Beam Bracing,” Engineering Journal, AISC, Vol. 38, No. 1, pp 11-26.

Wood design properties were obtained from the chapter entitled "Properties of Lumber Products," contained in *Marks' Standard Handbook for Mechanical Engineers*, 11th edition, 2007, pp6-101 to 6-127.

Snow and rain load was obtained from the *National Building Code of Canada* and *The Ontario Building Code* http://www.buildingcode.online/1249.html.

"Snowmobile Bridge Guidelines," Ontario Ministry of Natural Resources Policy PL 10.08.00, Number RA 3-3, 1992 August was consulted for guidance on design loading and railings.

Considerations on whether to include railings on a trail bridge are discussed in "Trail Bridge Rail Systems," US Technology & Development Program, United States Department of Agriculture Forest Service, 0723-2339, MTDC, 2007.

Footing design principles were based on American Concrete Institute Standard ACI 318-19, Building Code Requirements for Structural Concrete. Again, rather than using the LRFD methodology for designing concrete reinforcement, the simpler and more conservative ASD method was used.

4. OVERALL DESIGN CONCEPTS

There are two types of loads that govern the design of these bridges:

- distributed load (such as self-weight and snow load) and
- localized loads (people or vehicles).

For the distributed loads, the deck does not have to be especially strong, but, because these loads are integrated over the whole bridge, distributed loads are very significant for the main beams. Localized loads control the design of both the deck and the main beams.

Bridges such as these generally have the following components:

- Two main beams, on each side of the bridge. Often these beams are quite tall and form a guardrail.
- Cross beams that run between the two main beams. Cross beams transmit the deck loading to the main beams. They are spaced many feet apart and are often made with rectangular tubing.
- Stringers that support the wooden deck planks. The stringers are spaced quite closely, generally 8 to 12 inches apart if any type of vehicle loading has been considered. This close spacing is required for two reasons.
    - Wood planks cannot span very far when loaded locally by a vehicle.
    - Close spacing enhances rigidity. Flexibility is irritating to users and can cause failure of fasteners.
- Wooden deck planks running across the bridge.

- Sometimes longitudinal wooden planks in line with wheel tracks (“runners”) are used on bridges that must withstand heavier loads. These spread the wheel load to more than one deck plank and reduce fastener loosening.
- Diagonal bracing is used to prevent the bridge from skewing under unusual loads.

These are illustrated in the pedestrian bridge shown in Figure 3.

Figure 3. Underside of a pedestrian bridge at Victoria Park, Kitchener, ON

Our bridges have several simplifications, primarily to minimize cost and facilitate assembly:

- The structural elements are made of aluminum, which is lighter than steel, to allow in-situ assembly and minimize future maintenance due to its corrosion resistance.
- Bolts are used instead of welding.
- The bridge deck has no curvature. (Often bridges have an arch shape for esthetics and to maximize clearance below the bridge.)
- The main beams are aluminum structural I-beams that do not protrude above the deck. (Pedestrian bridges often use truss-type construction, which is much stiffer but also more complex.)
- Because aluminum is more flexible than steel and because an I-beam profile was chosen for the main beams rather than a truss, bridge deflection is relatively large for snow and heavy vehicle loads. For the normal live loads, however, deflection is small.

Our bridges are long enough to allow footings to be placed safely back from the water's edge. Soil conditions are different at the four locations. Footing options will be metal helical piles, direct attachment to bedrock, or poured concrete.

One design objective was to make these bridges long-lasting, with a design lifetime of perhaps 100 years, except for wood components that would need to be replaced more frequently.

5. ALLOWABLE STRESSES

5.1 Aluminum

Aluminum components are made of grade 6061-T6, which is the standard structural aluminum. Yield stress in either tension or compression is:

$$\sigma_{ty} = \sigma_{cy} = 35{,}000 \text{ psi}$$

Ultimate stresses in tension and shear are:

$$\sigma_u = 38{,}000 \text{ psi (for extrusions)}$$

$$\tau_u = 24{,}000 \text{ psi}$$

For bridge-type structures designed using the ASD method, the appropriate factors of safety, are n_y=1.85 on yield and n_u=2.2 on ultimate. Because there is much less plastic deformation in 6061-T6 before failure as compared to steel, the allowable stress is generally governed by the ultimate stress criteria:

$$\sigma_a = \sigma_u/2.2 = 17{,}273 \text{ psi}$$

$$\tau_a = \tau_u/2.2 = 10{,}909 \text{ psi}$$

For combined bending and shear, stress utilization is:

$(\sigma/\sigma_a)^2 + (\tau/\tau_a)^2 \leq 1.0$

5.2 Wood

The following properties are for S-P-F No. 1 or No. 2 loaded in the flat direction. The allowable bending stress is obtained by multiplying the base value by a series of adjustment factors, as shown below.

The size factor depends on plank width. For a 2x8 it is 1.2. The repetitive use factor would be 1.15 when three or more members share the load, but it is 1.0 for our deck. The flat-use factor increases the allowable stress for 2x6 or 2x8 planks used in this orientation. The wet-use factor would be 0.85 for the strongest grades, but otherwise is 1.0.

E	1.40E+06	psi
Base design value for bending	**875**	psi
Adjustment factors		
C_F = size factor (No. 1, 2 or 3 Grade)	1.2	
C_r = repetitive member factor	1.0	
C_{FU} = Flat-use factor	1.15	
C_M = Wet-use factor	1.0	
Allowable bending stress	1,208	psi

Allowable tension or compression parallel to the grain is roughly half the allowable stress for bending, but the current design does not use wood tension or compression members. Bearing stress for S-P-F wood perpendicular to the grain is 425 psi. This is not limiting for the current design.

Conventional pressure-treated wood causes corrosion and therefore cannot be installed in contact with aluminum, so a membrane is typically used to maintain a separation.

A new type of pressure-treated wood, MicroPro Sienna, claims to embed the copper inside cells, reducing release to the environment and not causing as much damage to contacting aluminum.

5.3 Bolts

For corrosion resistance and compatibility with both aluminum and pressure-treated lumber, stainless steel fasteners have been chosen. The design calculations initially assumed that bolts would be either Grade 316 or Grade 304, which have a nominal ultimate tensile strength of 75,000 psi. (Grade 18-8 is weaker.) Subsequently, it was discovered that stronger cold worked bolts (designated “CW”) made to ASTM F593C are easily available and so these were used. F593C bolts have a minimum ultimate tensile strength of 100,000 psi. The expected strength of the actual bolts that are being used is roughly 140,000 psi.

Bolt Strength					
Tensile strength of stainless steel bolt	100,000	psi			
minimum shear strength = 60% x tensile strength	60,000	psi			
safety factor for stainless steel bolts	3.0				
reduction factor for shear on threaded section	75%				
Shear					
Bolt size:	3/8"	1/2"	5/8"	3/4"	
nominal diameter	0.375	0.5	0.625	0.75	in
nominal area	0.1104	0.1963	0.3068	0.4418	in^2
minimum shear strength = 60% x tensile strength	60,000	60,000	60,000	60,000	psi
small bolt thread area factor	0.9	1.0	1.0	1.0	
allowable shear force (no threads)	2,209	3,927	6,136	8,836	lb
allowable shear force (on threaded section)	1,491	2,945	4,602	6,627	lb
Tension					
stress area for tension	0.0775	0.1419	0.226	0.334	in^2
allowable tensile force	2,583	4,730	7,533	11,133	lb

Combined tensile and shear forces are acceptable if:

$$(F_t/F_{ta})^2 + (F_s/F_{sa})^2 \leq 1.0$$

where subscript t refers to tension, s to shear, and a to allowable.

Standard bolt holes shall be no larger than 1/16” greater in diameter than the nominal bolt size.

All bolted joints in this bridge design use bearing-type connections rather than friction-type (which would require high-strength bolts). The bearing strength of the aluminum in contact with the bolt must be considered. This is generally quite a high value because some yielding is allowed. Allowable bearing stress for 6061-T6 extruded material is:

$$\text{Allowable bearing stress} = 2\sigma_u/n_u = 2 \times 38{,}000 \text{ psi}/2.2 = 34{,}545 \text{ psi}$$

For our design, this is generally not a limiting factor. Allowable bearing stress is reduced by a factor of 1.5 for slotted holes.

To avoid downgrading the connection, edge distance needs to be considered. Provided the centre of the bolt hole is no closer than 2 × bolt diameter from the edge of the material, measured in the direction of the load, the full load can be used. Allowable bearing stress is reduced by 25% if edge distance is reduced to 1.5 × bolt diameter. Particularly if multiple bolts are used close to an edge, pull-out strength also needs to be assessed.

Allowable bearing force for extruded 6061-T6 with common size bolts in standard or slotted holes is tabulated below:

Bearing strength of 6061-T6 Extrusion					
Bolt size:	3/8"	1/2"	5/8"	3/4"	
Allowable bearing stress for standard holes	34,545	34,545	34,545	34,545	psi
Allowable bearing force - 1/8" thick material	1,619	2,159	2,699	3,239	lb
Allowable bearing force - 3/16" thick material	2,429	3,239	4,048	4,858	lb
Allowable bearing force - 1/4" thick material	3,239	4,318	5,398	6,477	lb
Allowable bearing force - 5/16" thick material	4,048	5,398	6,747	8,097	lb
Allowable bearing force - 3/8" thick material	4,858	6,477	8,097	9,716	lb
allowable bearing stress (with slot or oversize hole) $F = 2Ft_u/(1.5n_u)$					
Allowable bearing stress for slotted holes	23,030	23,030	23,030	23,030	psi
Allowable bearing force - 1/8" thick material	1,080	1,439	1,799	2,159	lb
Allowable bearing force - 3/16" thick material	1,619	2,159	2,699	3,239	lb
Allowable bearing force - 1/4" thick material	2,159	2,879	3,598	4,318	lb
Allowable bearing force - 5/16" thick material	2,699	3,598	4,498	5,398	lb
Allowable bearing force - 3/8" thick material	3,239	4,318	5,398	6,477	lb

6. DESIGN

6.1 Parts List and Assembly Drawings

Aluminum 6061-T6 components:

1. Main I-beam, I 12×14.3
2. Cross Beam, 6×4×¼” tube, square corners
3. Stringer, 4×4×1/8” tube, square corners
4. Diagonal Brace, 2×2×3/16” angle, one brace oriented with leg up, the other leg down
5. A & B, Cross Beam Bracket, 5×3×5/16” angle
6. A & B, Stiffener, 3×3×3/8” angle
8. Stringer Bracket A, 2.5×2.5×¼” angle, slotted hole on upper end
   Stringer Bracket B, 3×3×3/8” angle, round hole on upper end (used at mid-span only)
9. Deck Bracket, 3×3×3/8” angle (not shown)
10. Diagonal Brace Bracket, 3×3×3/8” angle
11. Spacer for Diagonal Brace, 1”OD × ½” ID tube (not shown, at intersection point)
12. Stringer Splice, 3.5×3.5×3/16” tube (not shown, fits into adjoining ends of pieces of stringer to extend across the whole bridge)

Wood components:

13. Deck Plank, 2×8×96” pressure treated wood (or 2×10 if needed to fit end location)
14. Runner, pressure treated wood, A (2×10) & B (2×4)

Ramp components:

18. Ramp Hinge, 3.5×3.5×¼” tube
19. End Piece, 6×4×½” angle

Ramp stringers (3), deck planks (13) and runners (14) are the same as for the main bridge span, but stringers and runners are only 2’ long.

Figure 4. Proposed DRXC bridges shown with deck installed (top) and without deck (bottom) showing aluminum frame

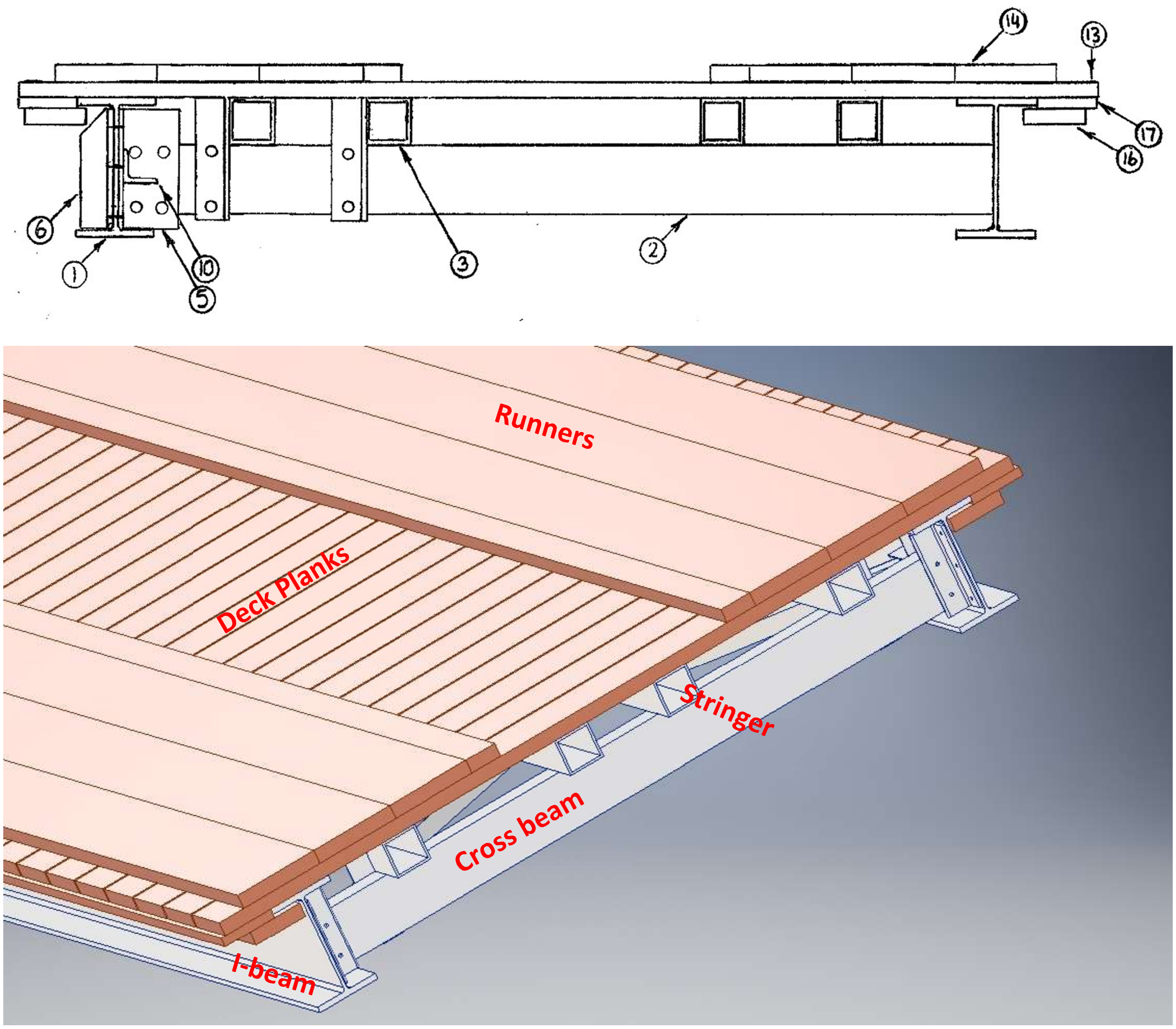


Figure 5. Transverse cross-section

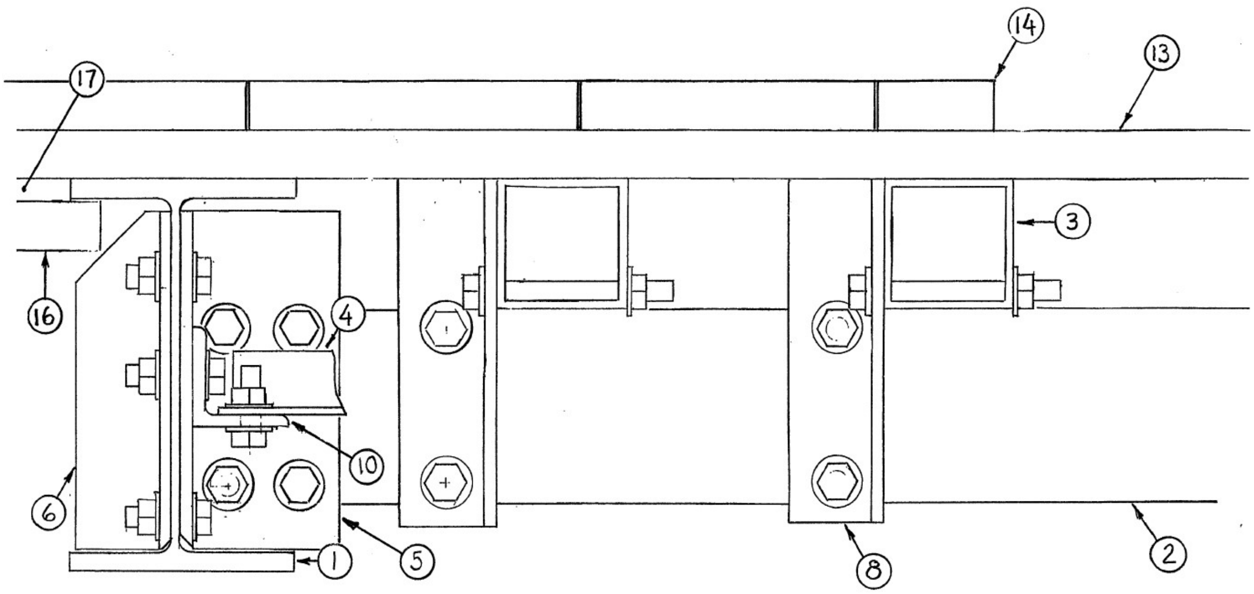


Figure 6. Bolting details

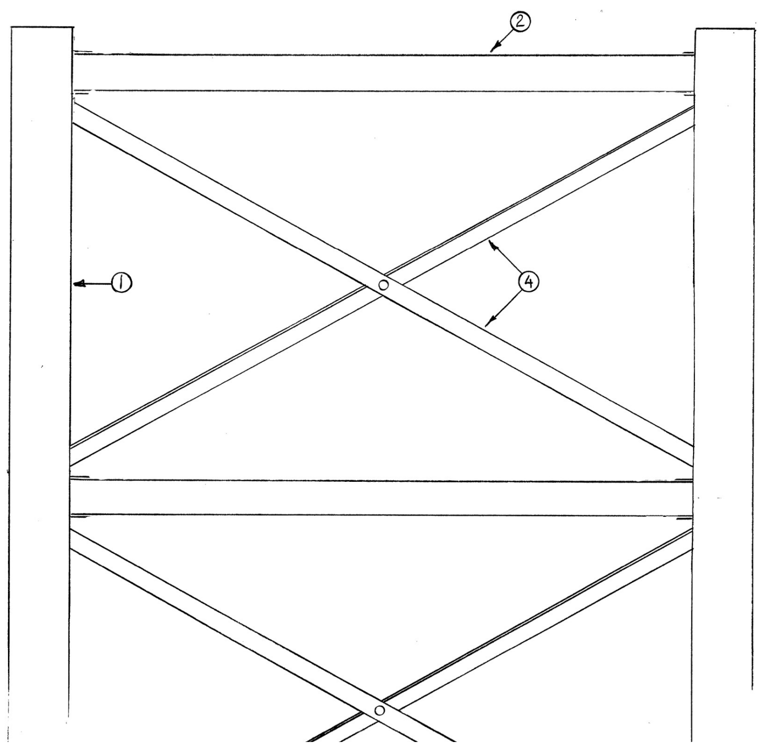


Figure 7. Plan view of diagonal braces (without deck or stringers)

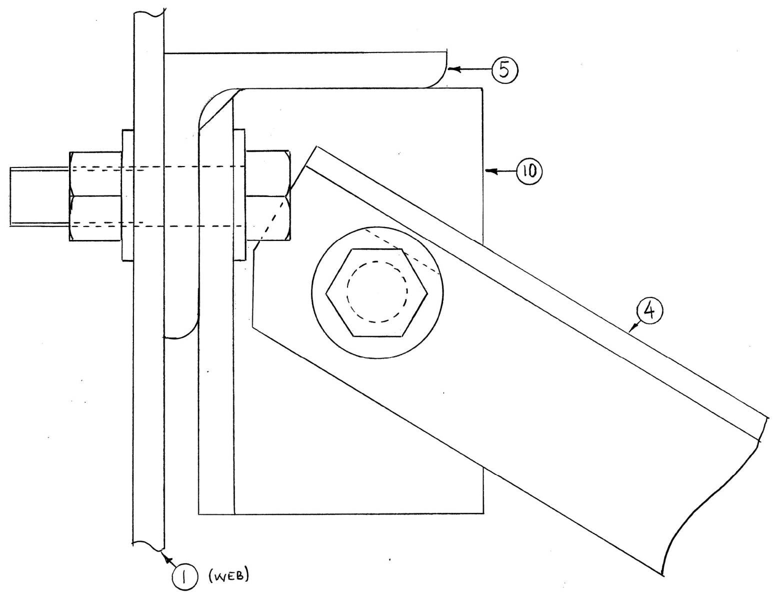


Figure 8. Connection of diagonal braces to web of I-beam

(Stiffener (6) is not shown.)

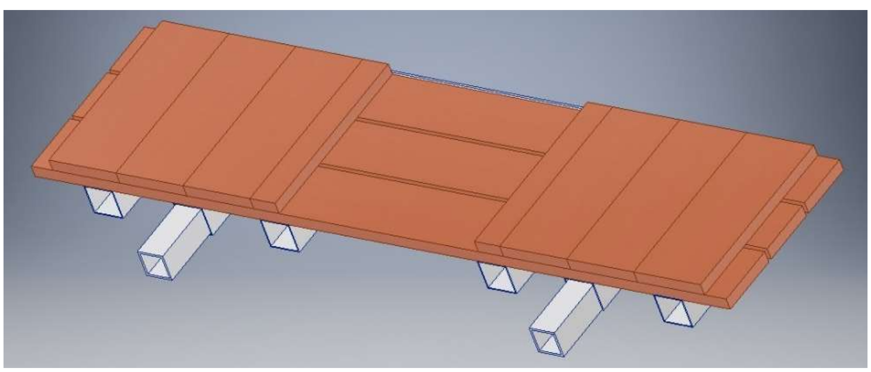


Figure 9. Ramp

6.2 Main Beams

The chosen section is Aluminum Association Standard I-Beam I 12 × 14.3. Flange thickness is uniform, as opposed to tapered towards the edges. Properties are as follows:

Height × Flange width:	12 in × 7 in
Weight per foot	14.292 lb/ft
Cross-sectional area	12.153 in^2
Flange thickness	0.62 in
Web thickness	0.31 in
Fillet radius (flange-to-web)	0.4 in
I_x	317.33 in^4
I_y	35.48 in^4

6.2.1 Stress and Deflection

Self-weight and snow load are treated as a uniform load along the length of the beam. Vehicle weight is treated as point loads at axle locations. This second assumption is reasonable, but not exactly correct because the cross beams transfer part of the vehicle load to the main beams at each of their locations, independent of the exact location of the vehicle axles. However, the net result is no more severe than the assumed loading because if an axle is not directly lined up with a cross beam, its load will be shared by more than one cross beam and the load will be distributed to the I-beam in a similar manner.

The governing vehicle loading (on one beam) was defined as follows by "driving" over the bridge with a tractor and trailer until stress was maximized:

Distance along bridge from support location (in)	Force (lb)
44	1,700
113	2,300
239	2,896
272.5	2,896

These loads were increased by 13% to account for the assumed eccentric loading, meaning that one of the beams would be loaded more than the other, which is equivalent to the vehicle driving 5" to one side from the centreline.

Calculated stress and deflection for the bounding bridge span are as follows:

Load Case	Bending	Shear	Stress	Deflection
Span = 35'2"	Stress	Stress	Utilization	(in)
	(ksi)	(ksi)		
Single Loads				
Self-weight, SW	3.3	0.14	19%	1.0
10,000 lb tractor	9.3	0.24	54%	2.4
7,000 lb Tractor & trailer	14.0	0.50	81%	3.9
Groomer & driver	2.1	0.09	12%	0.5
Snow & rain load	7.6	0.33	44%	2.3
90 psf	12.1	0.53	70%	3.7
Load Combinations				
Summer: SW + 10,000 lb Tractor	12.6	0.4	73%	3.4
Summer: SW + 7,000 lb Tractor&Trailer	17.3	0.65	100%	5.0
Winter: SW + Groomer + Snow	13.0	0.57	75%	3.9
Generic: SW + 90 psf	15.4	0.68	89%	4.8

Stresses are acceptable.

Deflection for point loads is a cubic function of span (and a fourth power for distributed loads), so any reductions of span from the maximum that has been analyzed will result in significantly reduced deflections.

The bridge support points will be installed as close together as feasible. For the M1 bridge, which will be the one subject to tractor loading, the span will be roughly 30 feet and the resulting stresses and deflections will be reduced as shown below:

Load Case	Bending	Shear	Stress	Deflection
Span = 30'0"	Stress	Stress	Utilization	(in)
	(ksi)	(ksi)		
Single Loads				
Self-weight, SW	2.8	0.14	16%	0.6
10,000 lb tractor	7.4	0.26	43%	1.5
7,000 lb Tractor & trailer	11.0	0.53	64%	2.2
Groomer & driver	1.8	0.09	10%	0.3
Snow & rain load	5.5	0.33	32%	1.2
90 psf	8.8	0.53	51%	2.0
Load Combinations				
Summer: SW + 10,000 lb Tractor	10.2	0.4	59%	2.1
Summer: SW + 7,000 lb Tractor&Trailer	13.8	0.67	80%	2.8
Winter: SW + Groomer + Snow	10.1	0.57	59%	2.2
Generic: SW + 90 psf	11.6	0.68	68%	2.6

Deflections due to pedestrian or groomer loading are quite small. The ½" maximum deflection caused by a groomer driving directly over one beam represents a ratio of deflection to span of slightly greater than 1/800. This happens to equal the general guideline for vehicular loading in the AASHTO LRFD Bridge Design Specifications. (It should also be noted that this specification states that deflection guidelines are optional, as long as deck integrity has been considered.)

Maximum deflection under either snow load or tractor loading are much larger, but these deflections result from either static loads or the special case of a tractor crossing the bridge. In either case, they will not affect the performance or structural integrity of the bridge.

6.2.2 I-Beam Stability

Another aspect that was analyzed was the ability of the I-beams to resist sideways twisting instability when subjected to large vertical load. Because the stiffness of the I-beams to bending in the horizontal plane (i.e., about a vertical axis) is an order of magnitude lower than the desired orientation of bending in the vertical direction, the beams tend to roll sideways when the bridge is loaded. A major purpose of the cross beams is to prevent this instability.

A moment connection is required between the I-beams and the cross beams, so a very rigid connection is required. This connection also ensures that the relatively thin web of the I-beam does not flex. Without stiffening the web at the connection points, the web flexibility would overwhelm the benefit of the stiff cross beam.

The moment connection is strong enough to withstand both the bridge loading and the restoring moment required to prevent instability.

6.3 Cross Beams

The cross beams transfer bridge loads to the main I-beams, and, as described above, provide torsional stability to the I-beams. The chosen section is 6×4×¼”. To ensure that I-beams do not buckle and to provide good support for stringers, a 47” separation between cross beams was selected. Bridge length may be shortened by reducing the separation at one or more locations (preferentially end locations to improve stringer stiffness).

While the ends of the cross beams are rigidly attached to the I-beams, the I-beams themselves are not fixed rigidly to anything else. Thus, the I-beams cannot be considered to provide a moment support to the cross beams, and so the cross beams are modelled with pinned ends for purposes of evaluating stress and deflection.

The joint between cross beams and I-beams is highly stressed and must be quite stiff to provide the required torsional bracing to the I-beams. Angles are used to connect each end of a cross beam to the web of the I-beam. These angles sandwich the web of the I-beam. This stiffens the web, which otherwise would allow the I-beam to twist sideways when the bridge is loaded. To evaluate the connection strength, it was necessary to consider the relative stiffness of the cross beam compared to the torsional stiffness of the I-beam because twisting of the I-beam partially relieves stress in the joint.

Stress and deflection of a cross beam under bounding loads of vehicle loading and self-weight is shown below.

Bending stress	9.2 ksi
Shear stress	0.8 ksi
Stress utilization	63%
Deflection at centre	0.19 in
Deflection at wheel location	0.12 in

These deflections are slightly conservative because they assume no moment support at the ends of the cross beam.

The 5×3×5/16” and 3×3×3/8” angle chosen for attachment brackets provides compatible stiffness and sufficient room for bolts.

6.4 Stringers

The primary purpose of the stringers is to provide intermediate support for the deck planks. They also provide additional stiffness to the bridge. A square tube section has been chosen for two main reasons:

- Square sections are not susceptible to twisting instability under bending loads because their vertical and sideways area moments of inertia are identical.
- The square section reduces the effective span of the deck planks, as compared to a narrower beam.

The chosen section is 4×4×1/8".

Stress in these beams due to distributed loading (such as snow load) is very small. The governing load is vehicle wheel loading. For assessment of stringer performance, it is assumed that the design wheel load is applied as a point load aligned with a single stringer mid-span between two cross beams.

An important design consideration is the attachment of stringers to cross beams. The two main issues are:

- Brackets provide downwards force on stringers when the bridge is loaded elsewhere so they are pulled down to match the overall "sag" of the main I-beams and cross beams. All stringers must follow the overall deflection of the bridge, or the deck will be disrupted.
- Stringers need to be free to expand lengthwise relative to the cross beams when the bridge deflects. The required connection brackets to provide this flexibility have short slots aligned with the direction of the stringers to allow this expansion under load.

Stress and deflection of a stringer under bounding loads of vehicle loading plus self-weight is shown below.

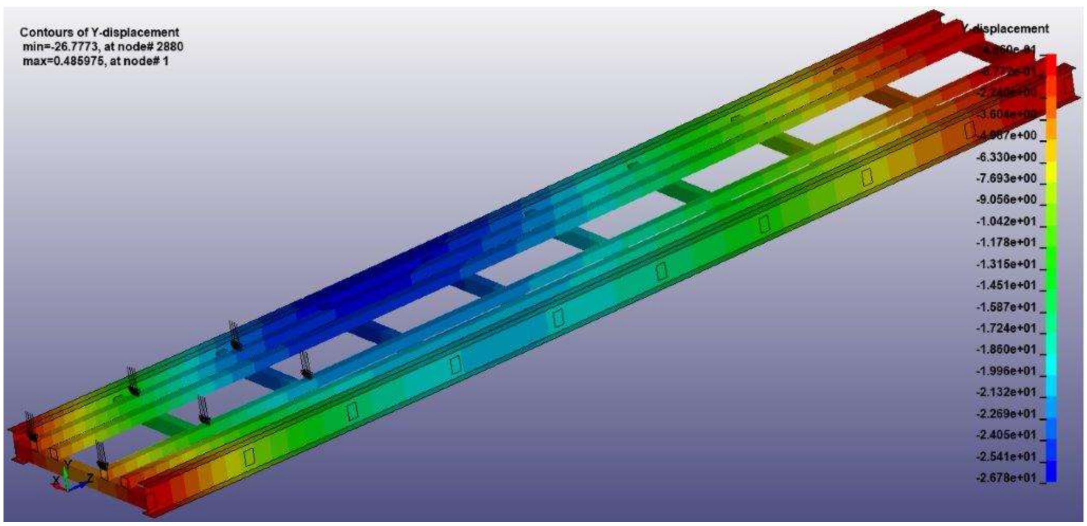
Bending stress	16.4 ksi
Shear stress	1.3 ksi
Stress utilization	96%
Deflection at mid-span under wheel load	0.13 in

The deflection in the above table is probably slightly conservative because of the multi-span beam formulas that were used. Not included in the above table is the deflection of the stringers to follow the corresponding deflection of the cross beam, discussed in the previous section. Deck stiffness was not credited for these calculations.

To achieve a continuous stringer over the length of the bridge, stringer sections will be joined using 3.5×3.5×3/16" tubing inserted into the ends of mating pieces (with shims to take up the clearance). Since these connection pieces are stronger than the stringer, the only restriction on the location of joints is avoiding bolt interference.

6.5 Finite Element Analysis

Calculations of stresses and deflections have all been done by hand using beam theory, applying appropriate loads to individual components. To verify gross results, a finite element analysis was performed. It verified the hand calculations and allowed further optimization of the design.



Preliminary finite element model of bridge substructure without deck or diagonal braces

6.6 Wooden Deck

The deck has been designed with the following assumptions:

- The maximum wheel load is assumed to be 4,000 lb.
- Load is assumed to be spread by at least one runner to at least two deck planks. Thus, the design load for a single deck plank is 2,000 lb, which is assumed to be spread uniformly over the width of a runner (conservatively assumed to be 7.25", although runners are actually 9.25" wide). This assumption has been checked analytically.
- It is assumed that this load is applied in the worst location within the defined track.

With these assumptions, the maximum allowable simply-supported span is 10". Contacts with the stringers and I-beams are modelled as follows:

- Contact with the stringers is assumed to be on their edges. Since the stringers are 4" wide, the support is assumed to be 2" from their centreline. Pinned support is assumed (which is slightly conservative).
- For the contact of a plank with the upper flange of the I-beam, not all planks have the same support because of the torsional flexibility of the I-beam. The worst position is half-way between cross beams because the torsional flexibility of the I-beam is greatest at this point. Contact with the upper flange of the main I-beam is assumed to be across the whole flange, and the torsional flexibility of the I-beam is included in the analysis. For our geometry, it works out that the effective contact point is only about 1" from the web (even though the flange protrudes 3.5" on each side of the beam).

Within possible wheel track locations, the centre-to-centre span between longitudinal beams has been chosen as 12", which means that the effective span of planks is 8" between stringers and 9" between stringer and I-beam.

The deck has also been analyzed for a 6" cantilever overhang beyond the beam centreline, considering the flexibility of the I-beam, as described above.

Between possible wheel track locations, there is a 27.5" zone without runners. Planks span 26" between the edges of the middle two stringers. The maximum allowable point load mid-span on a single plank in this zone would be 500 lb, which is more than adequate for any anticipated use.

The deck needs to be able to slide axially a small amount relative to the I-beams and stringers. The differential thermal expansion between aluminum and wood could result in about 3/8" of relative motion between the deck and support structure at each end of the bridge (assuming a temperature range of –40°C to +40°C). When the bridge is highly loaded, there could be a similar elongation of the lower surface of the deck at each end of the bridge relative to the I-beams and 1/8" relative to the stringers. To cope with this relative motion, the deck is only bolted rigidly to the aluminum substructure at mid-span. At other cross beam locations, the deck is bolted through slotted brackets that ensure that the deck conforms to the overall sag of the bridge under load and restrains it in high winds while allowing some axial squirm.

To minimize fastener loosening in the deck, large head fasteners are specified.

6.7 Diagonal Braces

To stiffen the bridge for general use and especially for parallelogram-type deflection caused by unusual off-design lateral loads, diagonal braces are installed between cross beams. Two braces forming an "X" (rather than a single diagonal brace) are used so the two braces provide mutual support at their intersection point, which means that they can be effective in both tension and compression. This doubles the effectiveness of each individual diagonal brace, which means that an "X" configuration is four times as effective as a single diagonal. When the bridge is subjected to a lateral load, one diagonal brace in each “X” will be in tension and the other in compression. The two diagonals are connected at their intersection by a single bolt. The diagonal in tension is tight enough when loaded that it can be considered rigid for purposes of providing buckling support for the diagonal in compression.

It is sometimes advantageous to position the diagonal braces close to the neutral axis of the I-beams, because if they were to be positioned too far below the neutral axis, they would be subject to large tensile stresses when the bridge is loaded, which would result in excessively large connection loads in braces in the middle sections of the bridge.

For our design, the diagonal braces have been attached to the middle bolt attaching the cross beams brackets to the web of the I-beam. This bolt is not loaded as heavily as the others because it does not restrain the cross beam for moment. An angle bracket connects each brace to this bolt in such a way that the centroid of the 2×2×3/16” angle used for the braces lines up roughly with the bolt, thereby preventing additional misalignment forces on the bolt.

The 2” width of angle permits use of a 5/8” bolt (but requires that one edge of the washer be ground off a bit so as not to interfere with the fillet of the angle). Bolt strength is the limiting factor for the strength of the brace. The 3/16” thickness was chosen to provide sufficient bearing strength for the bolts and for suitable robustness.

The slenderness ratio, kL/r, determines the allowable stress in a compression member. For our situation, k=1, L is the unsupported length (i.e., the half-length for our diagonal braces), and r is the radius of gyration of the section. For the chosen 2×2×3/16” section, the allowable compressive force is less than the shear strength of a 5/8” bolt, so, although both tension and compression arms of the brace are effective, buckling does limit the capability of the compression diagonal brace somewhat.

The primary purposes of the diagonal braces are to ensure that the bridge structure remains square and to enhance the general stiffness of the structure. While these requirements cannot easily be quantified because potential off-design side loads and skewing loads are ill-defined, the strength of the bracing system to such hypothetical loading can be assessed. With diagonal braces in each of the nine sections, the bridge structure itself could resist a side load of roughly 8 kips applied mid-span, or ~20 kips skewing load applied axially to one of the I-beams. (While these values are conservative because of the factors of safety, it must be stressed that these are hypothetical off-design loading conditions, and the footings have not been designed for this type of horizontal load.)

6.8 Natural Frequency

In general, it is advantageous that the first natural frequency of a bridge is as high as possible, so it is not close to potential excitation frequencies that might risk exciting a resonance. The first natural frequency of the bridge is 4.6 Hz for a 34 ft span between footings and somewhat higher for the expected 30 ft span of the M1 bridge. This first mode is a vertical oscillation of the centre of the bridge with a node point at each footing. Since the bridge is much stiffer in the horizontal direction, horizontal modes do not come into play until higher frequencies.

For our bridges, the primary excitation source will be from pedestrians. The range of excitation frequencies by pedestrians depend on their activity: walking (1.2 – 2.4 Hz), running (2.0 – 3.5 Hz), jumping (1.8 – 3.4 Hz). Counterbalancing the concerns with resonance between the bridge and pedestrian loading is the fact that wood and bolted joints have higher damping than welded construction typically used in steel bridges, which may mitigate the issue somewhat; 5% damping is typically assumed for metal structures with bolted connections. Assuming a 5% damping ratio with a worst-case 3.5 Hz forcing function, the input would be amplified by a factor of 1.5, which is not significant. With lower-frequency forcing functions, the amplification would be much lower. For example, at 2.4 Hz, the amplification factor is 0.5, and no resonance occurs.

With winter snow load, the natural frequency of the bridge might drop, but it is more likely that the extra stiffness provided by the packed snow and ice together with the extra damping would prevent resonance. Furthermore, skiers and snowshoers are likely to exert much gentler and lower-frequency forces on the bridge than pedestrians.

Under the controlled circumstances in the summer when a maintenance vehicle crosses the bridge, its extra mass would cause the natural frequency to drop within the potential pedestrian excitation frequency range, but because there would be no pedestrians on the bridge, no low frequency periodic forcing function is foreseen.

6.9 Ramps & Connection to Bridge

This component is sometimes called a “transition.” With concrete embankments, the transition can be shorter, but without a retaining wall, the transition length is increased.

A 24” long ramp has been designed that uses the same rectangular tubing as the stringers in the bridge, oriented in the same direction (parallel to the long dimension of the bridge). Because these tubes are designed to handle a 48” span between bridge cross beams, they were not re-analyzed for the ramp. No I-beams are used for the ramp.

Ramp tubes are hinged to the mating tube on the bridge using a single bolt, with enlarged holes to allow relative motion of the bridge components. The ramp angle should be set approximately level to minimize horizontal vehicle loading on the bridge, but can be adjusted slightly.

At the exposed end of the ramp, the tubes are joined together by a heavy angle that keeps the deck aligned and spreads the load to the ground. To minimize settling, a load spreading material should be installed under the end of the ramp; compacted gravel is recommended.

6.10 Fasteners

ASTM F593C hex bolts are used. Sizes are as follows:

Stringers to cross beams: ½”

Cross beams to I-beams: mostly ½”, some 5/8”

Diagonal brace attachment brackets: 5/8”

Diagonal brace crossover: ½”

Stringer splice: ½”

Deck to substructure: 5/8”

Ramp to main bridge: 5/8”

Where possible, bolts with unthreaded shanks are used to maximize shear strength.

To avoid crushing the tubes used for cross beams and stringers, bolts have been positioned near the edge of a face, so they do not apply an appreciable moment to the tube walls.

Precise fastener torque is not critical. One method is to tighten the nut finger-tight and then one additional turn. Another is to tighten to ~520 in-lb (1/2”) or 1100 in-lb (5/8”).

Because the chosen stainless steel bolts cannot be tightened enough to ensure that they do not loosen over time, they will be locked using nylon insert nuts. In locations with slotted holes that are designed to allow some slip, Loctite and lower nut torque will also be used.

Wooden deck components are joined using GRK RSS Climatek large-head wood screws.

6.11 Footings & Connection to Bridge

The embankment at each end of all bridges will be stabilized for erosion using rocks of size range approximately 6 to 10”. Slope will not exceed 45 degrees.

Soil at the M1 location, shown in Figure 10, contains many large rocks that would interfere with helical pile installation, so reinforced concrete pads with embedded studs to attach the bridge will be used. Reinforcement will have a factor of safety of at least 3 to cope with stress within the footing due to bridge loads. Footing area will be roughly 30 ft^2 to ensure soil contact pressure does not exceed 1,000 psf. (This includes the factor of 2 reduction of soil bearing capacity required by the building code when close to the water table.) Compacted gravel will be placed under the concrete pad for stability and to enhance drainage to reduce frost heave. Studs will allow level adjustment. There is flexibility in footing placement, so span will be minimized to improve bridge stiffness; target span is 30 ft.

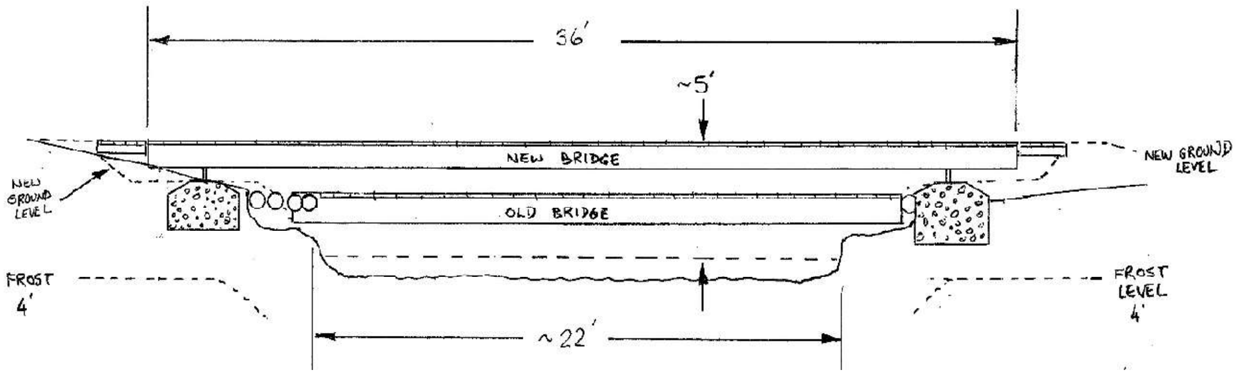


Figure 10. M1 location: existing bridge and replacement (looking downstream)

The M2 and X1 locations can be mounted directly to bedrock. Concrete will be anchored to the rock and will provide the required elevation for the bridge. Embedded studs will attach to the bridge.

For the X2 location, helical piles are planned. Valley Window & Door supplies and installs Techno Metal Post brand of helical piles in a suitable size.

6.12 Railings vs. Curbs

As explained below, railings are judged to be detrimental to the function of the bridge and have not been included in the design.

Ontario MNR's "Snowmobile Bridge Guidelines" only suggests railings if the height of the bridge deck exceeds 5 feet above the creek. If included, railings are to be about 42" high and capable of withstanding simultaneous 50 lb per lineal foot horizontal and vertical loads. The Silver Spoon bridges will be roughly 5 feet above Kennedy Creek. Heights will vary slightly between locations because of different high-water levels.

The US Forest Service's "Trail Bridge Rail Systems" has a more applicable guideline that considers non-motorized trails more typical of the Silver Spoon trials. It says that for bridge heights greater than 8 feet, railings are required, but between 4 and 8 feet, the need for a railing should be assessed based on expected usage and location. It requires curbs if railings are not used.

Reasons why railings have not been included in the design for the Silver Spoon bridges are listed below.

- The trails are backcountry trails with relatively low usage and no expectation for urban-type guardrails.
- The 8-foot width of these bridges allows skiers and pedestrians to stay well away from the edges by passing down the centre of the bridge. The groomer sets the ski track

down the middle and, in summer, the middle section of the bridge without runners forms a natural pathway.

- As nearby terrain is flat, skier and cyclist speeds at bridge locations are not high.
- Cyclists always ride on the centre portion of the bridge because they prefer planks running across the bridge rather than the longitudinal runners.
- Railings introduce their own risks, such as collision with the railing at the entry to the bridge and entanglement of grooming equipment (which is up to 7.5 ft wide).

A curb will be included to address the US Forest Service guidelines.

7. ASSEMBLY AND MAINTENANCE

As positioning of holes in bridge components in the field would be problematic, the metal structure should be pre-assembled in controlled working conditions, then transported to the installation site for final installation and assembly. The current plan is to transport the fully-assembled metal structure to site on a farm wagon. The wooden deck may be assembled either in the shop or at site.

Periodic checks on the integrity of the wooden deck, movement of the footings, bolt tightness, erosion, and proper support for the ramps should be undertaken. A schedule should be established and revised as necessary if the period between inspections is judged too long or short.