RIVERFRONT PARK BRIDGES

INSPECTION AND ANALYSIS

EAST WOODEN BRIDGE

NOVEMBER 14, 2014 | Final Report

With Revisions – 12/22/2014

kpff Consulting Engineers

EAST WOODEN BRIDGE

November 14, 2014 - With Revisions – 12/22/2014

Prepared for

City of Spokane
Steve Hansen, Senior Engineer

Prepared by

Kpff Consulting Engineers
Marijean Frymoyer, PE
Project Engineer / Inspection Team Leader

Sub-consultants

SWCA Environmental Consultants
Eileen Heideman
Architectural Historian

Echelon Engineering, Inc.
Shelly Sommerfield, PE
President

Table of Contents

1. BRIDGE DESCRIPTION

The two wooden deck bridges carry pedestrians and bikes across the south channel of the Spokane River in Riverfront Park. They were built for Expo in 1973. Both bridges consist of a wooden deck supported by a welded, weathering steel floor system supported on steel-encased, reinforced concrete piles. The east bridge, discussed in this report, is 178 feet long and 36 feet wide.

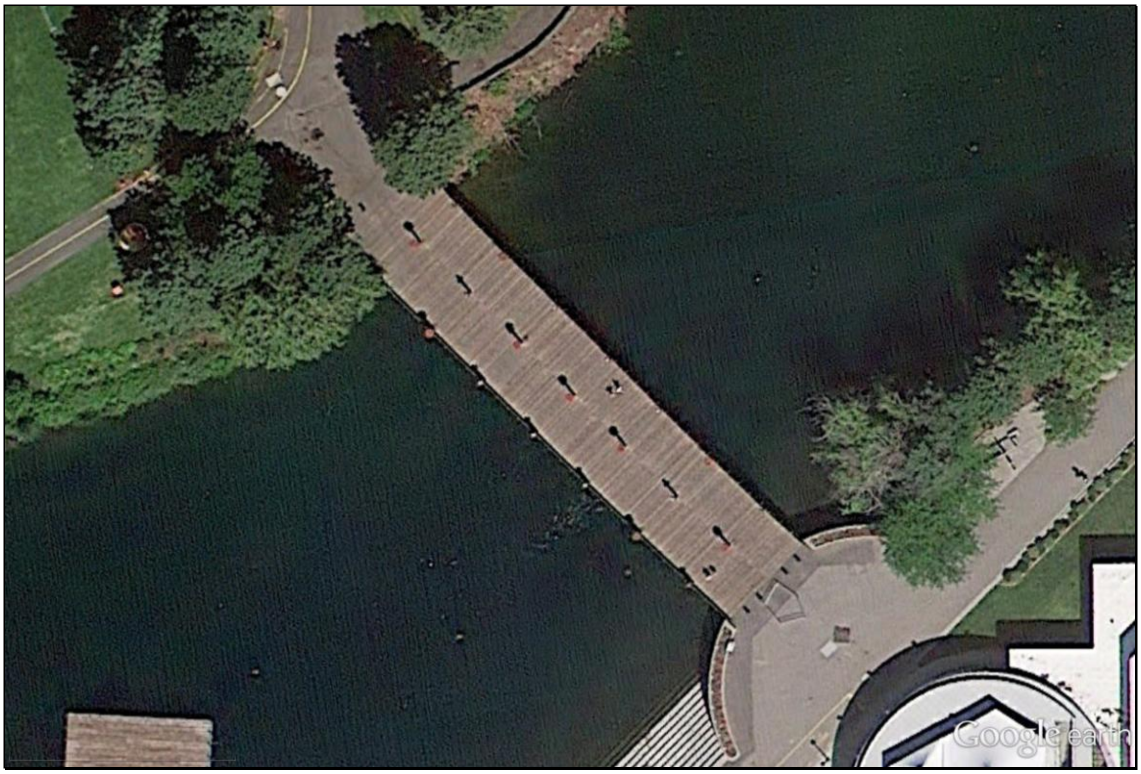


Figure 1: Aerial view of the East Wooden Bridge

2. DOCUMENT REVIEW

In preparation for this evaluation, Kpff reviewed the following documents related to the East Wooden Bridge:

- Erection Plan E2, Steel Details 6,8, and 9
- Previous routine inspection reports

3. EVALUATION PROCEDURES

ROUTINE BRIDGE INSPECTION

A visual inspection of the top of the deck and railings was performed. These components were accessed by foot. A visual inspection of the steel framing system, concrete filled steel piles, and abutments was also performed. These components were accessed by raft.

UNDERWATER INSPECTION

An underwater inspection was performed by Echelon Engineering. The purpose of the diving inspection was to assess the condition of the in-water substructure components and determine if there were any scour problems.

STRUCTURAL ANALYSIS

The timber deck, steel stringers, and floorbeams were load rated using the LRFR method. The analysis was performed by hand and using Excel. A uniform pedestrian live load of 90 psf and the H10 design vehicle were used in the analysis. The analysis assumed that there was only one vehicle on the bridge at a time and the vehicle load did not act concurrently with the uniform pedestrian live load. Impact was not included in the analysis.

4. EVALUATION FINDINGS

ROUTINE BRIDGE INSPECTION

The steel components are in good condition throughout, with only minor surface rust (no measurable surface loss). All of the steel connections are intact. The timber deck has evidence of normal wear and tear, with missing and loose bolts. There are many missing nuts on the railing-to-post connection. Many sections of the railing are loose. The grout pad below Stringer 8A is missing.

The bridge inspection report, bridge component labeling system, and photographs are included in Appendix A.

UNDERWATER INSPECTION

All substructure components appear sound. The top of the footing at Pier 9 is exposed.

The complete underwater inspection report is included in Appendix D.

STRUCTURAL ANALYSIS

The load rating analysis is reported as a Rating Factor (RF). The RF is the ratio of available capacity in each primary superstructure component over the specified live load combination under investigation. Based on AASHTO specifications, a RF less than 1.0 is interpreted to mean that one or more of the superstructure components do not meet current minimal capacity code standards and consideration should be given to either strengthening the subject component(s), or posting a sign identifying a maximum allowable load for the structure linked to the actual RF of the structure. Rating factors greater than 1.0 are interpreted to mean that all of the superstructure components have sufficient capacity to safely support the load under investigation, per the AASHTO specifications.

The controlling rating factor is dependent on the timber deck fully bracing the compression (top) flange of the steel stringers. The design drawings show a positive connection between the timber deck, the timber longitudinal nailers, and the steel stringers. This connection could not be inspected, but assuming it is still intact, the timber deck provides enough rigidity to brace the top flange of the stringers.

For the pedestrian inventory load case, the controlling RF = 0.82. For the pedestrian operating load case, the controlling RF = 1.07. The controlling component is the floorbeam in flexure. Although the pedestrian inventory rating factor is less than 1.0, an immediate retrofit of the floorbeams is not necessary. The inventory load case applies a 90 psf uniform pedestrian load multiplied by a 1.75 load factor. This is a very conservative load combination, which the bridge is not likely to see in its lifetime. When the deck is replaced, a positive connection between the deck and floorbeam should be added to fully brace the top flange. With the compression flange fully braced, the pedestrian inventory RF would equal 1.02.

For the vehicle inventory load case, the controlling RF = 0.16 for timber deck members in poor condition. The controlling component is the deck in flexure. The timber deck is not designed to carry vehicle loads, which reflects the low rating factor for the AASHTO H10 design vehicle. The City Parks Department should ensure that the bollards at the abutments remain in place to prevent vehicles from driving across the bridge.

The load rating calculations are included in Appendix C.

5. CONCLUSIONS AND RECOMMENDATIONS

If the current condition is maintained, this bridge will serve the community indefinitely.

In general, structural steel components that support bridges are susceptible to corrosion from environmental conditions such as water, salts, air pollution, dirt and plants, bird droppings, and bird nests. The more these items are kept a bay the longer the bridge will last. Maintenance is critical, especially in the form of cleaning and removing debris, bird nests, and droppings from anyplace on the structure they collect. The East Wooden Bridge structural components, despite experiencing minor levels of corrosion over the past 30 years, have performed quite well. Currently there is not sufficient reason to suspect that this bridge will not be in service for at least another 50 years if routinely inspected and properly maintained.

The steel used for this bridge is weathering steel. Its protective coat is a result of a thin film of rust. It is an excellent system for this environment. However, if over time this protection system appears to degrade, painting the bridge becomes an option which can easily achieve another 20 to 30 years of service life.

Maintenance of a few items, discussed below, will also help preserve the bridge and improve safety for the public

bearings

The grout pad below Stringer 8A should be replaced or the bearing plate should be shimmed to provide positive contact for the steel flooring system.

TIMBER DECK

The twisted and deteriorated boards should be replaced. Alternatively, the City might be well served by replacing the timber deck with a different material with a longer lifespan. By using a colored concrete mix with a special stamp or form liner, the concrete deck options could resemble a timber plank deck. Appendix B includes details and a cost comparison of different deck options. The total estimated cost of the deck replacement, dependant on the material selected, is between $410,000 and $530,000. The existing timber deck life span is near completion. A timber deck replacement in kind has a life span of approximately 10 years. The concrete deck, glulam deck panels, and Ironwood deck have a life span of approximately 50 to 75 years.

TIMBER RAILING

The missing nuts on the railing-to-post connection should be replaced and the bolts should be tightened on the loose sections of railing.

CONDUITS

The disconnected conduits below the deck should be repaired.

FUTURE INSPECTIONS AND ANALYSIS

A routine walk-through inspection should be performed every two years. Kpff has provided inspection forms which, if utilized on a continual basis, will provide an invaluable record of the bridge condition and areas of continual problems over time. This record will help inform the best way to care for the bridge over the next 75 years and thereby preserve the City's investment in its infrastructure. The bridge will not need to be reanalyzed unless the bridge will be used in a manner different than considered during the original design, or there is significant deterioration to the primary structural elements.

6. PERMITS AND CULTURAL RESOURCE REQUIREMENTS

An environmental permit matrix and cultural resource study was prepared by SWCA Environmental Consultants for the Riverfront Park Bridges. The proposed bridge improvement work may require a Hydraulic Project Approval permit from the Washington Department of Fish and Wildlife. More information can be found in SWCA’s report.

APPENDIX A

LIST OF PHOTOGRAPHS

CITY OF SPOKANE

PEDESTRIAN BRIDGE INSPECTION FORM

Bridge No.	874				
Bridge Name	King Cole Bridge / East Wooden Bridge	**Bridge Location**	South channel of Spokane River, near Opera House		
Inspection Date	8/12/14	**Inspector(s)**	T. Whiteman	**Agency**	KPFF
Access Method	Raft for in water access (contact Avista before performing in-water work)			**Weather**	Sunny/Hot
Load Rating Date	November 14, 2014	**Live Load**	Pedestrian: 90 psf ped.	Vehicle: H10 AASHTO design vehicle	
Load Rating Factor(s)	Ped.: 0.82	Veh.: 0.16	**Controlling Component**	Pedestrian: floorbeam - flexure	Vehicle: timber deck - flexure

Description of Bridge

This Page Intentionally Left Blank

Summary of Condition and Critical Findings

All of the structural components are in good condition - exhibiting minor surface rust and no measurable section loss. All connections are intact. The timber deck exhibits normal wear and deterioration. Several of the deck boards are loose and warped. There are a few connection nuts missing in the railing system.

Summary of Recommendations

1) Replace twisted and deteriorated timber deck planks. 2) Replace missing bolts and tighten loose bolts in deck.
3) Replace missing nuts in railing connection. 4) Repair disconnected conduits below bridge deck.

Summary of Bridge Condition

	Bridge Component	No. of Compon.	% of **	Condition Rating* 8 – 7 Good	6 – 5 Fair	4 – 3 Poor	Comments
1	Timber Deck	1	area	40%	20%	40%	See notes next page
2	Timber Railing & Posts	2	length	80%	20%	0%	East railing has nuts missing from connections
3	Steel Floorbeams	7	length	100%	0%	0%	Minor surface rust
4	Steel Stringers	56	length	100%	0%	0%	Minor surface rust
5	Concrete Filled Steel Piles	21	length	100%	0%	0%	Minor surface rust
6	Concrete Abutments	2	area	95%	5%	0%	SE abutment has noticeable crack
7	Bearings	14	each	100%	0%	0%	Stringer 8A bearing is missing grout
8	Expansion Joints	2	length	100%	0%	0%	
9	Scour	23	each	100%	0%	0%	
10	Erosion at Abutments	2	length	100%	0%	0%	
11							
12							
13							

*See Page 2 for detailed descriptions **Condition rating percentages are based on the % of area, length, or each of the bridge components inspected.

GENERAL NOTES

Scour components are 1 for each in-water pile and 1 for each in-water abutment. Erosion components are 1 for each bank. Railing and Posts are one for each side of the bridge.
Underwater inspection was performed on 7/29/2014 by Echelon Engineering.

DESCRIPTION OF CONDITION OF BRIDGE COMPONENT

Condition Value	Material	Description
8 – 7 Very good → Good 2 yr. insp. Cycle No repairs.	Steel	Like new, surface rust, minor pitting, no material loss. Connections are good. No damage.
	Concrete	No to minor/ insignificant defects includes: cracks, spalls, chips, consolidation, efflorescence.
	Timber	Beams: Minor splits, checks, or defects (one side), no decay or insects – sounds solid. Posts: Splits or cracks less than ⅜” (one side), no decay or insects – sounds solid.
	Paint	No defects, no sign of rust including no freckled rust, no peeling, no exposed steel.
	Scour / Erosion	None or minor.
6 – 5 Satisfactory → Fair 1 – 2 yr insp. cycle Monitor for repairs Paint: Max 10 year life estimate	Steel	Moderate corrosion, pitting, flaking, pack rust. Material loss is evident but barely measurable. Connections have up to moderate corrosion but remain fully functional. No cracks.
	Concrete	Some spalling but exposed rebar (if any) is insignificant or exhibits some surface rust; delamination is evident with or without evidence of rebar corrosion. Shear zone cracks are tight, barely measureable, and low density. Flexure zone cracks are measurable but less than .035 inch and low density. Concrete may exhibit: efflorescence (moderate to heavy), surface rust, heavy map cracking, very poor consolidation. Settlement cracks in foundations and wall are stable and less than ¼” wide.
	Timber	Beams: Less than ⅜” splits – two sides or greater than ⅜” on one side. Some decay (max 10% by volume), some softness but sounds solid – no insects. Posts: More than ½ “splits – two sides or greater than ¾” on one side. Decay is evident (greater than 20% by volume), timber may have extensive wetness and softness.
	Paint	Freckled rust, small areas of exposed steel, some peeling, oxidized.
	Scour / Erosion	Evidence of scour, exposed footing, no undermining. Banks are sloughing, protection, if any, needs repair.
4 – 3 Poor → Critical 3 mo – 1 yr. insp. cycle (as needed) Repairs needed. (ASAP or one year) Re - paint	Steel	Heavy to severe: corrosion, pitting, pack rust. Measurable material loss. Connections are heavily corroded, missing, and questionable functionality. Fatigue cracks.
	Concrete	Large spalls, deep w/ exposed and corroded rebar w/ material loss evident. Cracks are wider, closely spaced, clearly structural in nature both in shear and flexure zone. Concrete quality appears poor w/ heavy scaling, stagilites, efflorescence, map cracking, extensive surface rust and delamination, and very poor consolidation of concrete. Settlement cracks are significant.
	Timber	Beams: Greater than ⅜” on two sides. Moderate decay up to 20%, surface softness, do not sound solid – may have insects. Posts: Less than ½ “splits – two sides or greater than ½” on one side. Decay is evident (20%), wetness and soft.
	Paint	Extensive freckled rust, larger areas of exposed steel, heavily oxidized, extensive peeling.
	Scour / Erosion	Undermining or threatens undermining in a manner that could impact structure stability. Banks are heavily eroded, protection if any is non-functional.

Additional Comments by Component Number

Bridge Comp. No.	Comments
1	A total of 3 bolts were noted to be missing on the deck planks. There are 6 locations where planks have rotated upward about 3/4" creating a slight tripping hazard.

Photo 1 –East Wooden Bridge Deck (Looking North)

Photo 2 – East Wooden Bridge Elevation (Looking East)

Photo 3 – Missing Bolt in Timber Plank Deck

Photo 4 – Rotated Timber Plank, ~3/4-inch Grade Difference

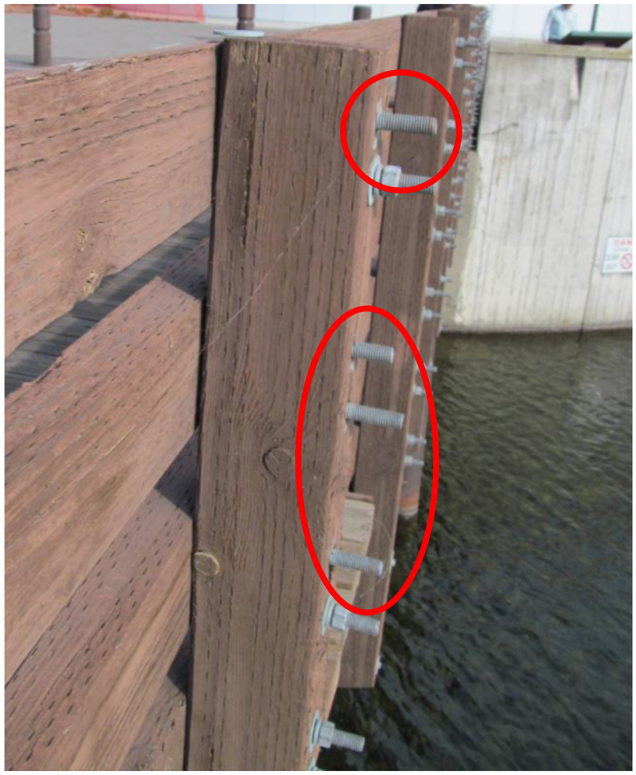
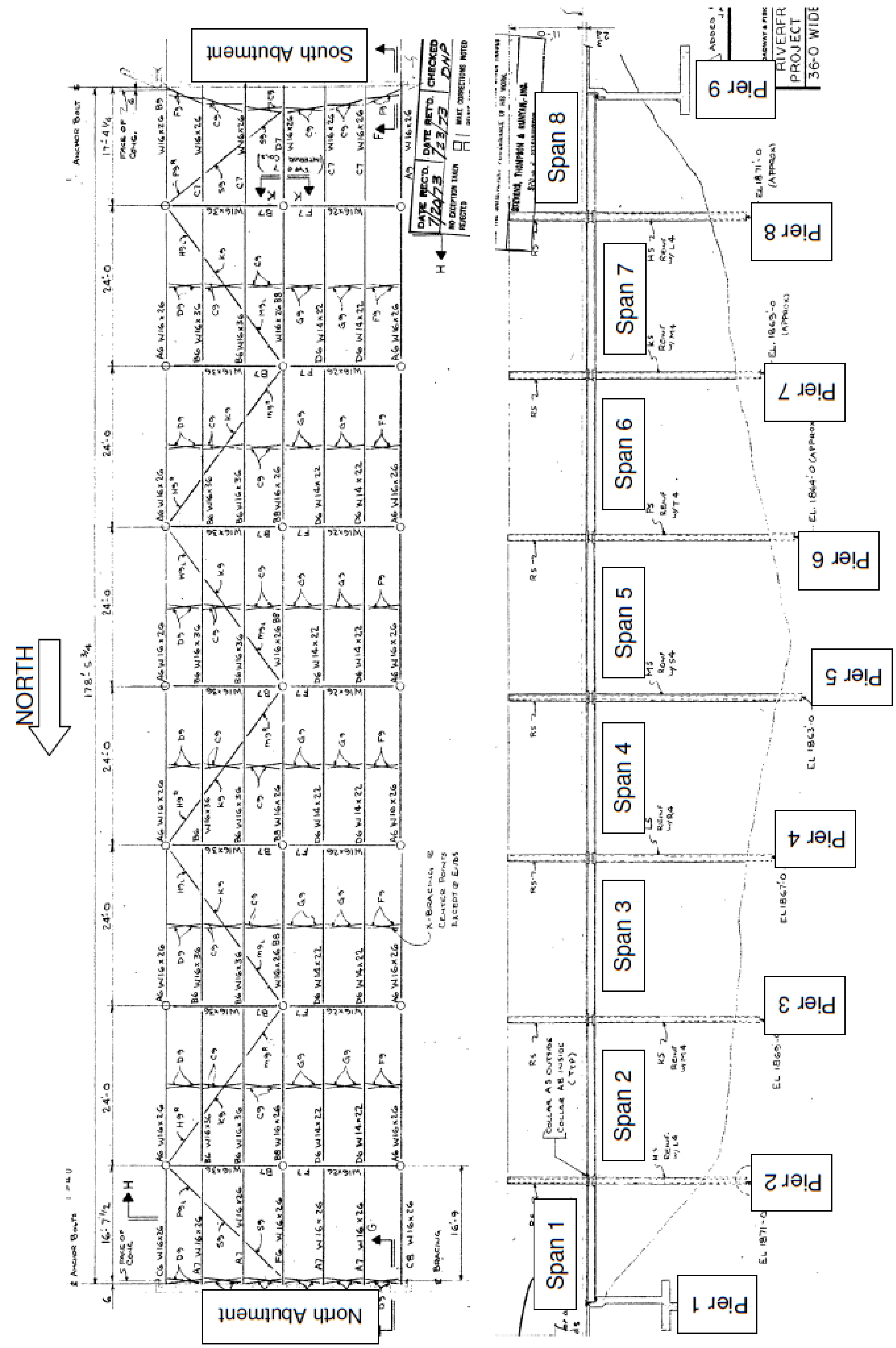


Photo 5 –Missing Nuts in Railing Connection

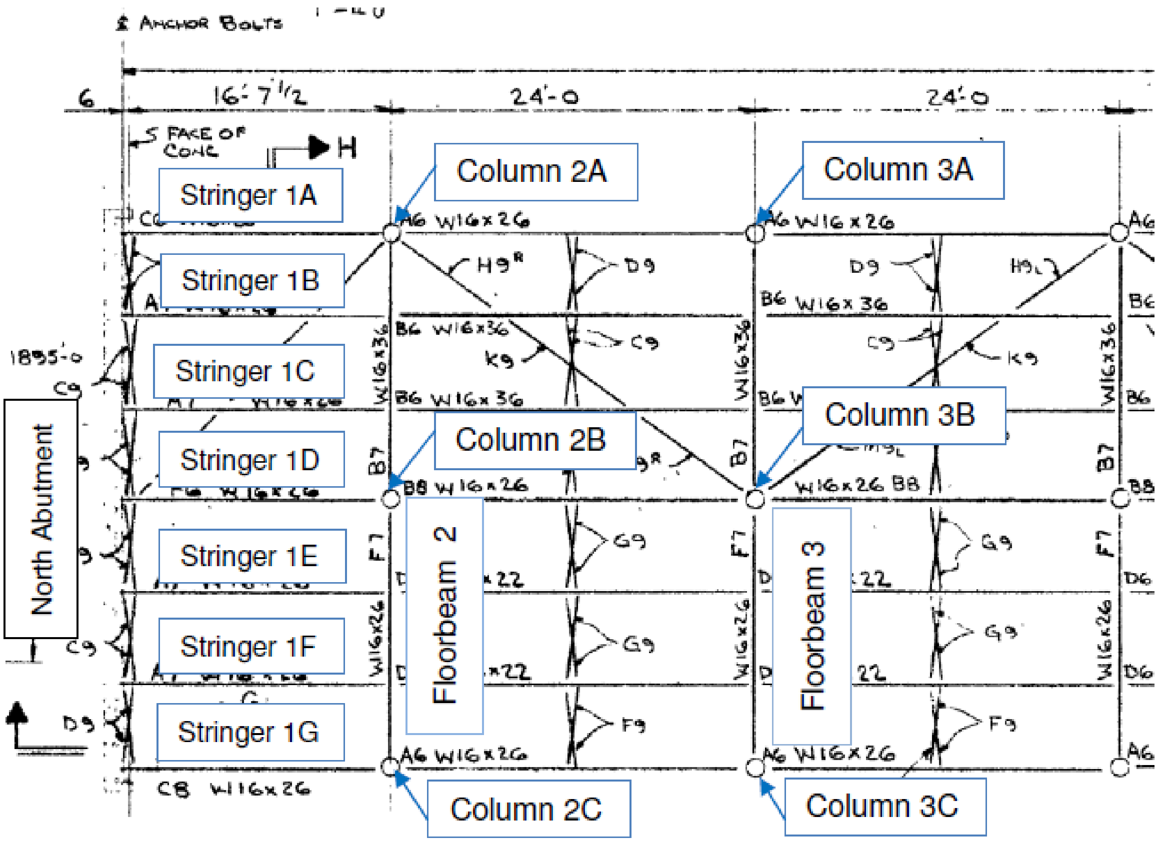
Photo 6 – Stringer 8A is not Bearing on Grout Pad

Photo 7 –Exposed Footing at Pier 9

Photo 8 –Disconnected Conduits Running below the Deck



Bridge Component Labeling System

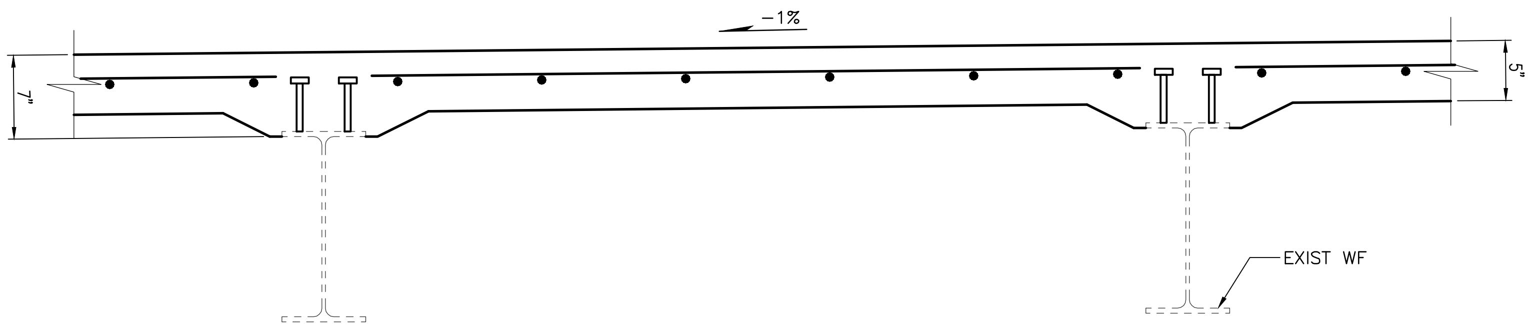


Bridge Component Labeling System: Columns, Floorbeams, and Stringers

APPENDIX B

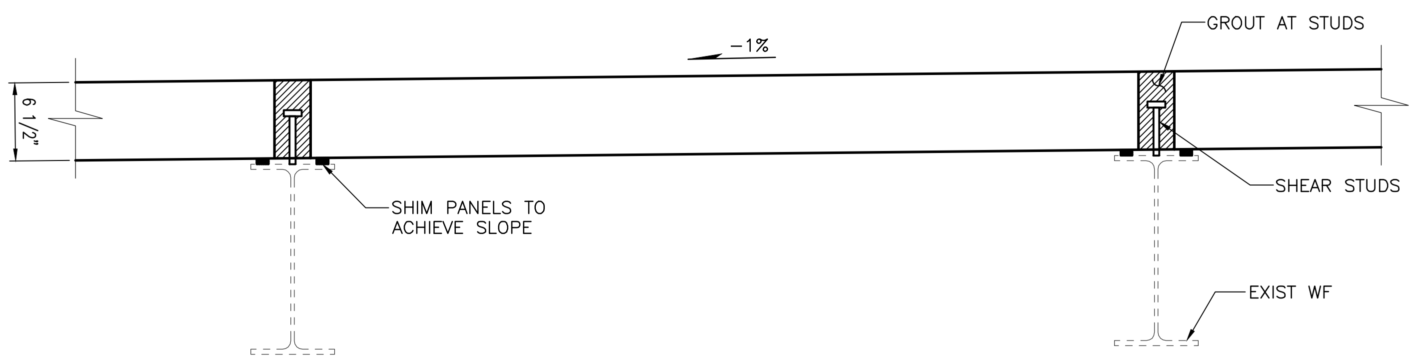
BRIDGE IMPROVEMENT DETAILS
COST ESTIMATES

REGION NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.
EASTERN	WASH.	---	



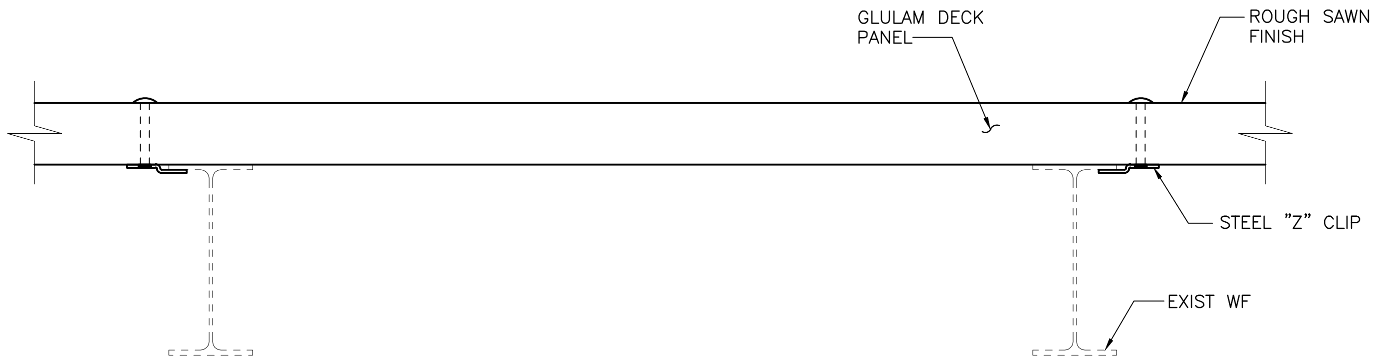
CAST-IN-PLACE CONCRETE SLAB

SCALE: 1 1/2" = 1'-0"



PRECAST CONCRETE PANELS

SCALE: 1 1/2" = 1'-0"



GLULAM DECK PANEL

SCALE: 1 1/2" = 1'-0"

Name: TaunieA Date: Nov 11, 2014-11:45:27am File: V:\114176 (Spokane River Front Bridges)\Deck Replecment\TWB-S01.dwg

REVISIONS: DATE | BY | PROJ. | DESCRIPTION

AS BUILT: DATE | BY | PROJ. | E.P.N. | U.S.N. | FROM | TO | COUNCIL ACCEPT DATE

GRADE ORDINANCE LIST: FROM | TO | ORD. NO. | DATE | FILE NO.

NAVD88 = (OLD CBM ELEV.) - (13.13)

AS OF JANUARY, 2000 USE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)

BENCH MARK LOCATION -

NAVD88 ELEV. -

CBM NO. -

NAVD88 DATUM

BAR IS ONE INCH ON ORIGINAL DRAWING.

IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

HORIZONTAL PLAN&PROFILE: AS SHOWN

VERTICAL PROFILE ONLY

SCALE

CURRENT C.O.S. DRAFTING STANDARDS ADOPTED FEB,2007

	BY	DATES
DRAWN:	HT	11/14
REVISED:	MLF	11/14
CHECKED:	TW	11/14
APPROVED:	MLF	11/14

CITY OF SPOKANE, WASHINGTON

DEPARTMENT OF ENGINEERING SERVICES

808 WEST SPOKANE FALLS BLVD.
SPOKANE, WASHINGTON 99201-3343
(509) 625-6700

PROJECT NAME: RIVERFRONT PARK BRIDGES

BRIDGE NAME: TRIANGLE AND WOODEN BRIDGES

DECK REPLACEMENT

TYPE OF IMPROVEMENT: BRIDGE

CITY PROJECT NUMBER: 2013186

EFN: --

PLAN NUMBER: 1 of 1

CALL BEFORE YOU DIG 1-800-424-5555

WESTERN WOOD
STRUCTURES, INC.

20675 S.W. 105th Ave.
Post Office Box 130
Tualatin, OR 97062-0130

Telephone: (503) 692-6900
Fax: (503) 692-6434

wwsi@westernwoodstructures.com
www.westernwoodstructures.com

Timber Bridge Maintenance Procedures

Western Wood Structures offers forty years of experience and expertise in the design and fabrication of your modern timber bridge. You can be assured that our state-of-the-art techniques result in a bridge that will deliver an effective service life of 75+ years, with only a few simple maintenance procedures to follow.

A pressure-treated timber bridge typically requires minimal maintenance in order to achieve its projected life expectancy. Our accurate fabrication details allow the bridge members to be fabricated before pressure treatment, thus the initial pressure-treating process provides a comprehensive, protective envelope for the wood.

The following guidelines can be used to further enhance the protections already implemented in a Western Wood Structures timber bridge.

1. A timber bridge is designed to provide air movement around the timber members, which works naturally to reduce moisture. Moisture control is essentially a common sense method of identifying and taking corrective action against sources of moisture, This includes routing the drainage patterns of the approach roadways to channel water away from the bridge. Dirt and debris can trap and retain moisture, and should be removed periodically.

2. All nuts and bolts should be checked and tightened after the first year of service, as necessary. Thereafter, the bridge should be visually inspected on an annual basis.

3. Virtually all bridges designed by Western Wood Structures are pressure-treated, providing a long and useful service life. during the course of several years, as the color of the bridge fades to a driftwood gray, be assured that the effectiveness of the treatment continues.

Following these simple recommendations will provide a long service life for your Western Wood Structures timber bridge. If you need further information, please contact me at (800) 547-5411, or e-mail me at: jagidius@westernwoodstructures.com.

Glulam Deck Rough Sawn Finish (Western Wood Structures, Inc.)

Glulam Deck Rough Sawn Finish Detail (Western Wood Structures, Inc.)

Bridge Decking and Rail Components

TIMBER HOLDINGS USA

Phone: 414-445-8989
www.ironwoods.com

Bridge Decking and Rail Components

Designers, manufacturers and their customers have long recognized the aesthetic, life cycle performance and environmental benefits associated with naturally durable hardwoods like Iron Woods® Ipe in bridge construction.

A stream anchor from the Margarita was found with a well-preserved wooden stock. An analysis by Forest Products Laboratories of the U.S. Department of Agriculture showed that it was made of a wood known as ipe or lapacho. On its crown are several well-preserved inscriptions: the date, 1618, and a foundry mark.

140 years – That's Durability

Phone: 414-445-8989
www.ironwoods.com

Bridge Decking and Rail Components

An environmentally superior alternative to Treated Wood, PVC or Composites… products carrying the ‘Green By Nature™ ‘Build with Conscience’ Certificate of Compliance meet a specific set of Controlled Wood, Chain of Custody, Life Cycle Analysis and Due Diligence criteria that support environmental sustainability initiatives as follows….

All of the material carrying the Green By Nature Certificate of Compliance have been verified as being, legally harvested, transported, exported, imported and documented in compliance with all country of origin, international and domestic laws, rules, regulations and treaties pertaining to the fair and legal trade of forest products including but not limited to the U.S. Department of Agriculture Lacey Act, ITTA (International Tropical Timber Trade Agreement), CITES (Convention On The International Trade of Endangered Species), and U.S. Buy American Act as per Green By Nature Controlled Wood Chain Of Custody Policies and Procedures.

Additionally, material carrying the Green By Nature Certificate of Compliance, are derived from a naturally occurring, renewable and sustainable resource base and are harvested from forests that have not been converted to plantations or where civil rights are violated. These materials are 100% organic and grown without the use of genetic modification or chemical fertilization and are regenerated naturally or by seeding and replanting. The natural service life of these materials exceeds their natural growth cycle. These materials trap and store carbon and they are able to be reclaimed, reused or recycled. These materials do not require for service any petroleum based or inorganic chemical treatments adhesives or coatings. These materials do not require for service any specialized handling storage or disposal procedures and generate zero post-industrial or post-consumer non-biodegradable waste. These materials are also safe for human and animal contact and meet Low VOC emission standards and meet International Building Code and International Residential Code requirements for naturally durable wood.

Phone: 414-445-8989
www.ironwoods.com

Bridge Decking and Rail Components

The following is a summary of technical information designed to assist in the material selection and specification process.

Technical Data - Iron Woods® Ipe

Features	Iron Woods® Ipe	
Composition	Naturally Durable Hardwood Untreated	
Species	Tabebuia spp. (Lapacho Group)	
Surface	Dressed / Profiled / Roughsawn	
Color	Natural	
Installation	Stainless Steel Fasteners	
Max overhand beyond joist	6"	
Weight per net bf AD 18%+ (avg)	5.5 - 6 lbs	
Weight per net bf KD 18% - (avg)	5 - 5.5 lbs	
Lengths	To 20'	
Property Description	**ASTM Standard**	**Iron Woods® Ipe**
Modules of Elasticity	ASTM D-143	3145000 psi
Bending Strength	ASTM D-143	22.475 psi
Compression Parallel to Grain	ASTM D-143	13,140 psi
Compression Perpendicular to Grain	ASTM D-143	3,595 psi
Shear Parallel to Grain	ASTM D-143	2,290 psi
Screw Pull Out		Avg. 1102 lbs Max Load
Coefficient of Friction - Leather	ASTM C1028-89	Dry - .55 FP / Wet .79 FP (ADA Compliant)
Coefficient of Friction - Neolite	ASTM C1028-89	Dry - .73 FP / Wet .69 FP (ADA Compliant)
Surface Burning	ASTM E-84 (1989)	NFPA Class A, UBC Class 1
Flame Spread (20 minutes)	ASTM E-84 (1989)	0

Flame Spread (10 minutes)	ASTM E-84 (1989)	5
Smoke Developed (10 minutes)	ASTM E-84 (1989)	3
Fuel Contribution (10 minutes)	ASTM E-84 (1989)	0
Acute Inhalation	NYS Modified Pittsburg Protocol	LC 50 0f 63.60g.
Combustion Toxicity Test	NYSUFPBC, Art 15, Part 1120,9 NYCRR 1120	Pass (19.7g or greater)
Surface Burning	ASTM E84 (2007)	NFPA Class B
Calculated Flame Spread (10 minutes)	ASTM E84 (2007)	33.37
Flame Spread Index	ASTM E84 (2007)	35
Calculated Smoke Developed	ASTM E84 (2007)	273.3
Smoke Developed Index	ASTM E84 (2007)	250

Additional Compliance Fire

City Of NY Dept. of Buildings	Fire Retardant Wood Code Sections 27-328	MEA # 220-01-M (Approved)
San Francisco Building Code	Code Section 1511.5 (rooftop decks)	(Approved)
CalFire Wildlife Urban Interface Areas	Code Section Chapter 7A (CSFM 12-7A-4)	(Approved)
Materials and Construction Methods	Exterior Wildlife Exposure: Decking	
International Building Code	Fire Resistant Wood	(Compliant)
International Residential Code	Fire Resistant Wood	(Compliant)

Additional Compliance Technical

International Building Code	Naturally Durable Wood	(Class 1 / Compliant)
International Residential Code	Naturally Durable Wood	(Class 1 / Compliant)



TIMBER HOLDINGS USA

Phone: 414-445-8989
www.ironwoods.com

IRON WOODS

CUMARU

IRON WOODS® Cumaru

"Iron Woods... Its Only Natural"

Species: Dipteryx Odorta
Common names: Cumaru, Brazilian Teak, Tonka
General Characteristics: Heartwood is reddish-brown to light yellowish-brown. Sapwood is distinct and narrow. It has a low to medium luster with a fine texture and an interlocking grain. Cumaru has a waxy or oily feel; and though it has no distinctive taste, it may have a vanilla-like odor. It is rated as easy to air season with a slight tendency to check and with moderate warping.
Durability: The timbers have a reputation for being very durable.
Working Properties: Slightly abrasive, responds well to planing and other machining operations.
Good nailing, screwing and gluing properties.
Uses: Common applications include heavy construction, decking, dock fenders, flooring, railroad crossties and tool handles.

Cumaru (Diperyx odorata)

Similar in appearance to Ipe, it can at times be difficult to differentiate to the less trained eye. Cumaru does however have a more coarse and interlocking grain which results in a slightly lower dimensional stability requiring Kiln-drying in dimensions in under 2" nominal in both storage and application. Cumaru is currently being used heavily in the commercial boardwalk industry in 2x4 and 2x6 decking as a lower cost alternative to IPE and where marine borers is not an issue.

Strength & Durability

Cumaru is a golden to reddish brown species of tropical hardwood with similar technical properties to Ipe with exception of its resistance to marine borers.

How does Iron Woods® Cumaru compare to other lumber and decking products?

	Cumaru	CCA-Treated Pine	Composite/PVC Decking
Type	Hardwood	Softwood	Plastic Wood
Maintenance	Low	High	Low
Decay Resistance	High	Varies	Varies
Termite Resistance	High	Varies	Varies
Strength	High	Medium	Low
Movement in Service	Medium-Low	High	High
Fire Rating Class	High	Varies	Low
Weight per cu. ft.	67lbs.	35lbs.	60 to 64lbs.
Bending Strength	22,400	9,900 - 14,500	1,423 - 4,500
E-modulus	3,010,000	1,170,000 - 1,510,000	175,000 to 480,000
Shear Strength	2,395	1,370	561 - 1,010
Hardness	3,340	690	940 - 1,390

IRON WOODS

TIMBER HOLDINGS USA

Cumaru

Availability

Cumaru is sold in two varieties: yellow and red and is typically sold mixed. Cumaru is best used in applications such as commercial decking, boardwalks, bridges, benches and exterior construction.

Decking – 1x4, 1x6, 5/4x6, 2x4, 2x6

Timbers – up to 12x12 by special order only.

All other dimensions up to 12x12 clear of heart center are special order only.

Finishing

We recommend coating Cumaru to assist the acclimation process and reduce checking. For best performance, coat all four sides and the ends of each board before installation. Use high-quality penetrating oil or water-based exterior sealers that contain mildewcides, fungicides, and UV inhibitors. Ask your local dealer about factory finishing.

"See Installation Guide for Pre Installation Handling and Storage Requirements"

Green by Nature

Green by Nature products meet a specific set of Life Cycle environmental criteria defined as:

* Product derived froma naturally occuring, renewable and sustainable resources.
* Not endangered or at risk as per CITES (Convention On the International Trade of Endangered Species)
* Not harvested from forest areas where traditional or civil rights are violated, converted for plantations or non-forest use.
* Harvested legally and sourced in compliance with all international laws and regulations pertaining to the trade of plant products and more specifically in U.S. Department of Agriculture "Lacey Act Compliant".
* 100% organic, grown without the use of genetic modification or chemical fertilization.
* Service life exceeds natural growth cycle, sequesters and stores carbon throughout its life cycle.
* Generates zero post industrial and post consumer non-biodegradable waste.
* Does not require for service, any specialized handling, storage or disposal procedures. Generates zero post industrial and post consumer non-biodegradable waste.
* Does not require petroleum based or inorganic chemicals treatments, safe for human and animal contact and meets low VOC emmission standards.

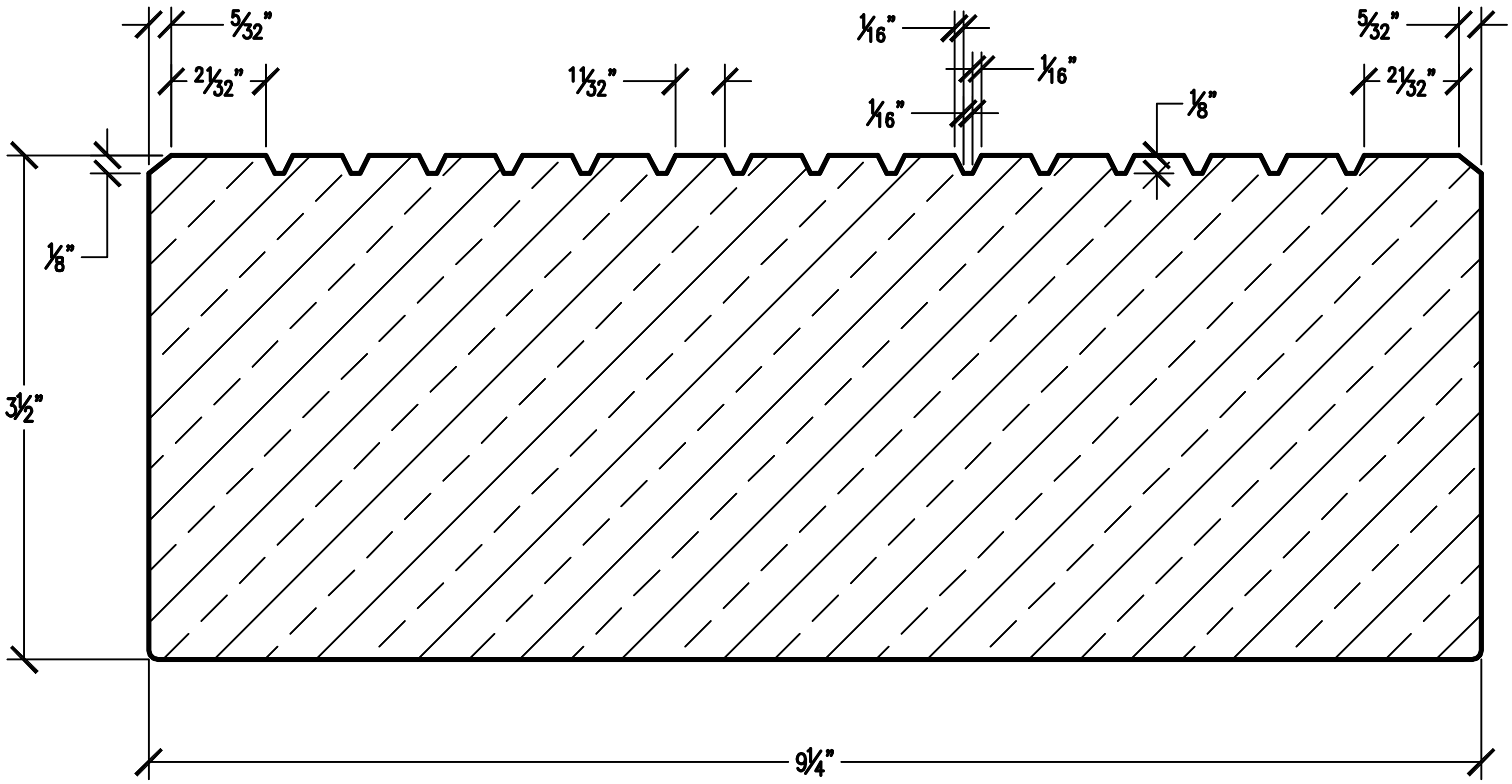
Green by nature BUILD WITH CONSCIENCE

To learn more about Green By Nature Certification go to www.greenbynature.com

TIMBER HOLDINGS USA

www.ironwoods.com

414-445-8989



GROOVE DETAIL

SCALE: 3/4"=1"

Rev. #	Appr. By	Date	Description	Rev. By

DATE: 01/08/13
SCALE: AS SHOWN
DRAWN BY: EGH
CHECKED BY: GN
APPROVED: JD
JOB No.
ACCT. No.

CEC
GENERAL CONTRACTOR, INC.
P.O. Box 22621
Lake Buena Vista, FL 32830
Phone 407 938-0177
FAX 407 938-0195

Structural Engineer of Record
Janine Pardee 407-595-3617
2513 Catalina Dr, Orlando, FL 32805
Professional Engineer PE 29522

GROOVE DETAIL

GROOVED 4x10 IPE DECK BOARD

SHEET
1 OF 1

City of Spokane Pedestrian Bridges Nov-14

Cost Estimates for Bridge Improvements Based on the 2014 KPFF Inspection and Analysis Recommendations

Bridge Name: East Wooden Bridge

Bridge Length and Width (feet) 179 37

Recommendations for Improvements - Include: Deck Replacement

Option 1 - Cast in Place Concrete Deck

Item no	Item Description	Cost Unit	Quantity	Unit Cost	Item Cost
1	Existing Rail Remove and Re-install	LF	358	35	$ 12,530
2	Remove Existing Deck	SF	6623	4	$ 26,492
3	New Deck	SF	6623	25	$ 165,575
4	Misc	LS	1	25000	$ 25,000
	Total				$ 229,597

Option 2 - Precast Concrete Panels

Item no	Item Description	Cost Unit	Quantity	Unit Cost	Item Cost
1	Existing Rail Remove and Re-install	LF	358	35	$ 12,530
2	Remove Existing Deck	SF	6623	4	$ 26,492
3	New Deck	SF	6623	30	$ 198,690
4	Misc	LS	1	25000	$ 25,000
	Total				$ 262,712

Option 3 - Glulam Deck Panels/Ironwood Deck*

Item no	Item Description	Cost Unit	Quantity	Unit Cost	Item Cost
1	Existing Rail Remove and Re-install	LF	358	35	$ 12,530
2	Remove Existing Deck	SF	6623	4	$ 26,492
3	New Deck	SF	6623	35	$ 231,805
4	Misc	LS	1	25000	$ 25,000
	Total				$ 295,827

Option 4 - Timber Deck Planks

Item no	Item Description	Cost Unit	Quantity	Unit Cost	Item Cost
1	Existing Rail Remove and Re-install	LF	358	35	$ 12,530
2	Remove Existing Deck	SF	6623	4	$ 26,492
3	New Deck	SF	6623	17	$ 112,591
4	Misc	LS	1	25000	$ 25,000
	Total				$ 176,613

Item no	Item Description	Cost Unit	Quantity	Unit Cost	Item Cost
5	Mobilization	10%	(of option 2)		$ 26,271
6	Design, Permits, Survey	20%	(of option 2)		$ 52,542
7	Construction Management	13%	(of option 2)		$ 32,839
8	Taxes	8%	(of option 2)		$ 21,017
9	Contingency	30%	(of option 2)		$ 78,814
10	Excalation (1 year)	3%	(of option 2)		$ 7,881
11	Agency Project Development & Mngmt.	5%	(of option 2)		$ 13,136
	Total				$ 232,500

Option 1 Total Project Cost (2015)	**$ 462,097**
Option 1 Square Foot Cost - ($/SF)	$ 70
Option 2 Total Project Cost (2015)	**$ 495,212**
Option 2 Square Foot Cost - ($/SF)	$ 75
Option 3 Total Project Cost (2015)	**$ 528,327**
Option 3 Square Foot Cost - ($/SF)	$ 80
Option 4 Total Project Cost (2015)	**$ 409,113**
Option 4 Square Foot Cost - ($/SF)	$ 62

*Ironwood Deck cost is comparable to glulam deck panels

APPENDIX C

LOAD RATING RESULTS AND CALCULATIONS

Structural Analysis –
Load Rating Summary

LRFR Bridge Rating Summary

Strength I – Rating Factors (RF):

	Pedestrian		Vehicle	
	Inventory	**Operating**	**Inventory**	**Operating**
Deck RF	5.19	6.72	**0.16**	**0.21**
Controlling Point	Deck - flexure		Deck - flexure	
Steel RF (discretely braced)	**0.82**	1.07	1.07	1.38
Controlling Point	Floorbeam - flexure		Floorbeam - flexure	
Steel RF (continuously braced)	1.02	1.32	1.32	1.71
Controlling Point	Floorbeam - flexure		Floorbeam - flexure	

<u>Maximum Wheel Live Load:</u>

Inventory = 0.16*8,000 lb = 1280 lb
Operating = 0.21*8,000 lb = 1680 lb

<u>Maximum Pedestrian Live Load for floorbeams braced at stringers:</u>

Inventory = 0.82*90 psf = 74 psf

Pedestrian = 90 psf uniform distributed load

Vehicle = H-10 Truck
(16,000 lb. front axle, 4,000 lb. rear axle, 14' axle spacing)

Figures C3.1-1 and C3.1-2 from the *LRFD Guide Specifications for the Design of Pedestrian Bridges* (December 2009) give a visual representation of the uniform pedestrian live load.

Figure C3.1-1—Live Load of 50 psf

Figure C3.1-2—Live Load of 100 psf

Structural Analysis - Load Rating

Design Parameters:

Steel

Yield Stress, fy = 50 ksi
Modulus of Elasticity, E = 29,000 ksi

Timber Deck

Pine
G = 0.55

Dead Loads

Superstructure self weight

Live Loads

Pedestrian Uniform Load = 90 psf
Vehicle Load = 20,000 lb H-10 Truck
Impact is not included
Pedestrian and Vehicle Loads do not act concurrently

Analysis Methods:

The bridge geometry and section properties were based on the steel erection drawings.

The moment, shear, and axial capacities and demands were calculated in Excel. The Strength I rating factors were calculated in Excel using the peak demands for each element type.

The visual bridge inspection completed on August 14, 2014 found the deck to be in poor condition. All other superstructure components were shown to be in good condition. The condition rating factor, φ_c, is equal to 1.0 for good members and 0.85 for the poor deck members. The system rating factor, φ_s is equal to 1.0 for the deck due to its redundant nature, but 0.85 for all other members.

The controlling rating factor depends on if the deck fully braces the compression (top) flange of the steel stringers. The connection between the timber deck longitudinal nailer and the stringers is not visible, but is assumed to still be intact. The timber deck planks have sufficient rigidity to brace the compression flange of the stringers.

In the current condition, the floorbeams are only braced at the stringer locations. For the future condition, when the deck is replaced, the deck can be connected to the floorbeams to continuously brace the compression flange. Load rating results have also been provided for the condition with the floorbeam is fully braced.

The Strength I Load Rating checks flexure and shear.

kpff Consulting Engineers
1601 Fifth Avenue, Suite 1600 Seattle, WA 98101
(206) 622-5822 fax (206) 622-8130

REVISION 1

Wooden Bridge East Load Rating Summary

Inventory Rating γ_{LL} = 1.75

Load Rating for Pedestrian Live Load (90 psf)

Interior Stringer - cont. braced compression flange

	Rating Factor
Flexure W16x36	3.21
Shear W16x36	11.22
Flexure W16x26	2.19
Shear W16x26	9.49
Flexure W14x22	1.62
Shear W14x22	7.56

Floor Beam - braced at stringers

Flexure W16x26	0.82
Shear W16x26	4.66

Floor Beam - Continously braced compression flange

Flexure W16x26	1.02

Timber Deck

Flexure	
Good Condition	6.11
Fair Condition	5.81
Poor Condtion	5.19
Shear	
Good Condition	16.75
Fair Condition	15.91
Poor Condtion	14.23

Minimum Rating Factor 0.82
Controlling Component Floorbeam Flexure

Load Rating for Vehicle Live Load (H10)

Interior Stringer - cont. braced compression flange

	Rating Factor
Flexure W16x36	2.60
Shear W16x36	8.23
Flexure W16x26	1.77
Shear W16x26	6.96
Flexure W14x22	1.31
Shear W14x22	5.54

Floor Beam - braced at stringers

Flexure W16x26	1.07
Shear W16x26	6.04

Floor Beam - Continously braced compression flange

Flexure W16x26	1.32

Timber Deck

Flexure	
Good Condition	0.19
Fair Condition	0.18
Poor Condtion	0.16
Shear	
Good Condition	0.57
Fair Condition	0.54
Poor Condtion	0.48

Minimum Rating Factor 0.16
Controlling Component Deck, Flexure

Operating Rating γ_{LL} = 1.35

Floor Beam - braced at stringers

Flexure W16x26	1.07

REVISION 1

Timber Plank Decking
3 ⅝"x7⅝" net dimensions
Southern Pine

G = 0.55 Specific Gravity (NDS Table 11.3.2A)
γ_{water} 62.4 lb/ft^3
γ_{timber} 34.32 lb/ft^3
σ_{timber} 9.65 psf

Interior Stringer

L = 24 ft
Spacing = 6 ft
Size W16x36
W16x26

Dead Loads

deck = 6 ft * 9.6525 psf = w = 57.92 lb/ft
stringers
W16x36 w = 36 lb/ft
W16x26 w = 26 lb/ft
W14x22 w = 22 lb/ft

Moment M = w l^2 / 8
W16x36 =(36+57.915)*24^2/8 = 6761.88 ft-lb 6761.88/1000 = 6.76 ft-kip
W16x26 =(26+57.915)*24^2/8 = 6041.88 ft-lb 6041.88/1000 = 6.04 ft-kip
W14x22 =(22+57.915)*24^2/8 = 5753.88 ft-lb 5753.88/1000 = 5.75 ft-kip

Shear V = w l / 2
W16x36 = (57.915+36)*24/2 = 1126.98 lb 1126.98/1000 = 1.13 kip
W16x26 = (57.915+26)*24/2 = 1006.98 lb 1006.98/1000 = 1.01 kip
W14x22 = (57.915+22)*24/2 = 958.98 lb 958.98/1000 = 0.96 kip

Live Loads Note: pedestrian load and vehicle load do not act concurently

<u>Pedestrian</u> 90 psf (LFRD Ped Bridge 3.1)

w = =90*6= 540 lb/ft

Moment M = w l^2 / 8
=540*24^2/8 = 38880 ft-lb 38880/1000 = 38.88 ft-kip

Shear V = w l / 2 = 540*24/2 = 6480 lb 6480/1000 = 6.48 kip

<u>Vehicle</u> H10 (LFRD Ped Bridge 3.2)
Clear deck > 10 ft
wheel 1 8 kip
wheel 2 2 kip
spacing 14 ft
For interior stringers: distribute one line of wheel loads to each stringer (AASHTO 4.6.2.2.2a)

Moment

for this stringer length, maximum moment occurs when 8 kip wheel is at midspan
M = P l / 4 =8*24/4 = 48 kip-ft

Shear maximum shear occrus when 8 kip wheel is over support
sum moments at opposite support
$\sum M_{R1}$ = 2 kip(10 ft) + 8 kip (24 ft) - R2 (24 ft)
=(2*10+8*24)/24 8.83 kip

Interior Stringers Cont. Braced

REVISION 1

E 29000 ksi
Fyc / Fyt 50 ksi

Capacity of Stringer

Local Buckling Resistance — Rb = 1.0

		W16x36	W16x26	W14x22		
λ_f = bfc / (2 tfc)		W16x36	W16x26	W14x22		(AASHTO 6.10.8.2.2-3)
b_{fc} = compression flange width		6.99	5.5	5	in	
t_{fc} = compression flange thickness		0.43	0.345	0.335	in	
D_c = depth of web in compression		7.52	7.505	6.515	in	
t_w = web thickness		0.295	0.25	0.23	in	
2*Dc/tw		51.0	60.0	56.7		
$\lambda_{rw} = 5.7 \sqrt{(E/Fyc)}$	137.274178					
if 2*Dc/tw ≤ λrw		YES	YES	YES		
λ_f =	=6.99/(2*0.43) =	8.13	7.97	7.46		(AASHTO 6.10.8.2.2-4)
$\lambda_{pf} = 0.38 \sqrt{(E/Fyc)}$ =	=0.38*SQRT(29000/50) =	9.15				
$\lambda_f \le \lambda_{pf}$		YES	YES	YES		
if $\lambda_f \le \lambda_{pf}$ then Fnc = Rb Rh Fyc						(AASHTO 6.10.8.2.2-1)

R_h = 1.0 for rolled shapes (AASHTO 6.10.1.10.1)
R_b = 1.0 if 2*Dc/tw ≤ λrw (AASHTO 6.10.1.10.2)

F_{nc} = =1*1*50= 50 ksi

	W16x36	W16x26	W14x22		
Sx	56.5	38.4	29	in^3	
Zx	64.0	44.2	33.2	in^3	
$R_{pc} = M_p/M_{yc} = Z_x/S_x$ =	1.13	1.15	1.14		(AASHTO A6.2.1-4)
$M_{yc} = F_{yc} S_x$ =	2825	1920	1450	k-in	(AASHTO D6.2)
$M_{nc} = R_{pc} M_{yc}$ =	3200	2210	1660	k-in	(AASHTO A6.3.2-1)

Lateral Torsional Buckling Resistance

		W16x36	W16x26	W14x22		(AASHTO 6.10.8.2.3)
$L_p = 1.0\, r_t \sqrt{(E/F_{yc})}$		43.53	33.16	30.51	in	
		3.63	2.76	2.54	ft	
$r_t = b_{fc} / (\sqrt{(12*(1+1/3*D_c*t_w / b_{fc} / t_{fc})}$		1.808	1.377	1.27	in	
L_b = unbraced length =	0 ft	0 in				Timber deck braces compression flange
$L_b \le L_p$		YES	YES	YES		
if $L_b < L_p$ $F_{nc} = R_b R_h F_{yc}$		50	50	50	ksi	
if $L_b < L_p$ $M_{nc} = R_{pc} M_{yc}$		3200	2210	1660	k-in	
$L_r = \pi r_t \sqrt{(E/F_{yr})}$		163.47	124.52	114.56	in	
		13.62	10.38	9.55	ft	
$F_{yr} = 0.7 F_{yc}$	35 ksi					
$L_b \le L_r$		YES	YES	YES		

if $L_b < L_r$ $F_{nc} = C_b (1-(1-F_{yr}/R_h/F_{yc})*(L_b-L_p)/(L_r-L_p) R_b R_h F_{yc} \le R_b R_h F_{yc}$

F_{nc} = W16x36 pedestrian 50.00 ksi
vehicle 50.00 ksi

$C_b = 1.75-1.05(M1/M2)+0.3(M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)

Bracing Check bracing strength of timber deck planks AISC 6.3.1a

$P_{br} = 0.008*M_r*C_d/h_o$ (Required brace strength) AISC A-6-5

h_o	13.37	*in*	(Distance between flange centroids)
C_d	1		(For bending single curvature)
M_r	1094	*k-in*	(Factored moment with LRFD factors)
d	3.375	in	(depth of timber planks)

$P_{br} =$ 0.655 *k*

$\Phi P_n = \Phi*F_c*A_g*C_p$ AASHTO 8.8.2-1

$\Phi_{comp.}$	0.9		AASHTO 8.5.2
$A_{g\ actual}$	24.89	in^2	
Deck condition factor	0.5		(to reduce strength)
$A_{g\ reduced}$	12.45	in^2	

$F_c = F_{co}*C_{KF}*C_M*C_F*C_i*C_\lambda$ AASHTO 8.4.4

F_{co}	1.45	*ksi*	AASHTO Table 8.4.1.1.4-1 - No. 1
C_{KF}	2.78		AASHTO 8.4.4.2 - format conversion factor = 2.5/Φ
C_M	0.8		AASHTO 8.4.4.3 - wet service factor, ≤ 4" thick
C_F	1.05		AASHTO 8.4.4.4-1, Size Effect Factor, 8" width & F_{co}
C_i	0.8		AASHTO 8.4.4.7 - incising factor
$C_{\lambda\ (Str\text{-}I)}$	0.8		AASHTO 8.4.4.9 - time effect factor
F_c	2.17	*ksi*	

$C_p = (1+B)/2c - \sqrt{((1+B)/2c)^2 - B/c} \leq 1$ AASHTO 8.8.2-2

K_{cE}	0.52		AASHTO 8.8.2
K	1		AASHTO 4.6.2.5 (assume pinned-pinned)
L	72	*in*	(Stringer spacing)
$L_e = KL =$	72	*in*	AASHTO 8.8.2
E_o	1500	*ksi*	AASHTO Table 8.4.1.1.4-1 - No. 1
$E = E_o*C_M*C_i$			AASHTO 8.4.4.1-6
=	960	*ksi*	
$F_{cE} = K_{cE}*E*d^2/L_e^2$			AASHTO 8.8.2-4
=	1.10	*ksi*	
$B = F_{cE}/F_c \leq 1$			AASHTO 8.8.2-3
=	0.51		
c	0.8		AASHTO 8.8.2, for sawn lumber
C_p	0.44		

ΦP_n 10.6 *k*

$\beta_{br} = 1/\Phi*(4*M_r*C_d)/(L_b*h_o)$ (Required brace stiffness) AISC A-6-6

Φ	0.75		
L_b	144	*in*	(length between steel braces)

β_{br} 3.03 *k/in*

$k_{timber\ planks} = A_{red.}*E/L$ (timber stiffness)

= 165.9 *k/in*

$\Phi P_n \geq P_{br}$ & $k_{planks} \geq \beta_{br}$ → Deck/nailer effectively braces stringers, LTB not considered

REVISION 1

E 29000 ksi
Fyw 50 ksi

Capacity of Stringer (continued)

~~for **vehicle** load, M1 = 0, therefor Cb = 1.75~~ ~~1.75~~

~~for **pedestrian** load~~
~~M2 = maximum bending moment at either end of unbraced length =~~ ~~38.88 ft-kip~~
~~Mmid = bending moment in middle of unbraced length (i.e. 6 ft. from end)~~
~~Mx = wx/2 (l-x)~~ ~~=540*6/2*(24-6)/1000 =~~ ~~29.16 ft-kip~~
~~M1 = 2Mmid - M2 ≥ M0~~ ~~=2*29.16-38.88 =~~ ~~19.44 ft-kip~~

~~Cb =1.75-1.05*(19.44/38.88)+0.3*(19.44/38.88)^2 =~~ ~~1.3~~

			W16x26	W14x22	
~~if $L_b > L_r$, $F_{nc} = F_{cr} \le R_b R_h F_{yc}$~~	~~W16x26~~	~~pedestrian~~	~~#DIV/0!~~	~~#DIV/0!~~	~~ksi~~
~~$F_{cr} = C_b R_b \pi^2 E/(L_b/r_t)^2$~~		~~vehicle~~	~~#DIV/0!~~	~~#DIV/0!~~	~~ksi~~

Tension Flange Flexural Resistance

$F_{nt} = R_h F_{yt}$ 50 ksi

Nominal Shear Resistance of Unstiffened Web

$\phi_v V_n = \phi_v C V_p$ (AASHTO 6.10.9.2-1)

	W16x36	W16x26	W14x22		
$V_p = 0.58\ F_{yw}\ D\ t_w$	128.67	108.82	86.9101	kip	(AASHTO 6.10.9.2-2)
D = depth of web	15.04	15.01	13.03	in	
D/t_w =	50.98	60.04	56.65		
k = 5.0 (given in 6.10.9.2)					
$1.12\sqrt{(E\,k/F_{yw})}$	60.31				
if $D/t_w \le 1.12\sqrt{(E\,k/F_{yw})}$	YES	YES	YES		(AASHTO 6.10.9.3.2-4)
C = 1.0					
$C\,V_p$	128.67	108.82	86.91	kip	

REVISION 1

Strength I

γ_{DC}	1.25	
γ_{LL}	1.75 Inventory	
γ_{LL}	1.35 Operating	
ϕ_f =	1.0	(AASHTO 6.5.4.2)
ϕ_v =	1.0	

ϕ_c = 1.00 Good or Satisfactory, BMS Condition 1 or 2

ϕ_s = 0.85

$\phi_c \phi_s$ = 0.85

Stringer - Flexure

Capacity, C, based on controlling case of local buckling and lateral torsional buckling, tension

$$RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_p P)}{\gamma_{LL} LL(1 + IM)}$$

	W16x36	W16x26	W14x22	
C = ϕf M_{nc}				
pedestrian	2720	1879	1411	k-in
vehicle	2720	1879	1411	k-in
dead				
M_{DC}	81.14	72.50	69.05	k-in
γ_{DC} DC	101.4	90.63	86.31	k-in
ped			Inventory	
M_{LL}	466.6	466.6	466.6	k-in
γ_{LL} LL	816.5	816.5	816.5	k-in
vehicle			Inventory	
M_{LL}	576.0	576.0	576.0	k-in
γ_{LL} LL	1008	1008	1008	k-in

RF ped = (2720-101.43)/816.48

RF vehicle = (2720-101.43)/1008

	W16x36	W16x26	W14x22
	Inventory	Inventory	Inventory
RF ped =	3.21	2.19	1.62
RF vehicle =	2.60	1.77	1.31

Stringer- Shear

	W16x36	W16x26	W14x22	
C = $\phi_v V_n$	128.67	108.82	86.91	kip
V_{DC}	1.13	1.01	0.96	kip
γ_{DC} DC	1.41	1.26	1.20	kip
Pedestrian				
V_{LL}	6.48	6.48	6.48	kip
γ_{LL} LL	11.34	11.34	11.34	
Vehicle				
V_{LL}	8.83	8.83	8.83	kip
γ_{LL} LL	15.46	15.46	15.46	

RF ped = (128.67-1.41)/11.34

RF vehicle = (128.67-1.41)/15.46

	W16x36	W16x26	W14x22
	Inventory	Inventory	Inventory
RF ped =	11.22	9.49	7.56
RF vehicle =	8.23	6.96	5.54

Floorbeam

L = 17.47 ft
Size W16x26

Dead Loads

Stringers + Deck — P_{DC} = 1.13 kip — Shear reaction from end of stringer
2 stringers/connection — P_{DC} = 2.25 kip — W16x36, V_{DC} = 1.13 kip

Floorbeam
W16x26 — w = 26 lb/ft

Moment — M = w l^2 / 8 floorbeam
W16x26 =(26)*17.47^2/8 = 991.760986 ft-lb — 991.8/1000 = 0.99 ft-kip

M = P a — =2.25*6 = 13.52 ft-kip
a = 6 ft

M_{DC} = — =0.99+13.52+2*0= — **14.52 ft-kip**

Shear — V = (2 P +w l)/ 2
W16x26 = 26*17.46875/2 = 227.09 lb — =227.09/1000 = 0.23 kip

Shear from stringers = 2P / 2 = — = 2*2.25/2 = 2.25 kip

V_{DC} = — **2.48 kip**

Live Loads — Note: pedestrian load and vehicle load do not act concurently

Pedestrian

Maximum moment and shear occurs when all stringers are loaded — Shear reaction from end of stringer
Stringer Live Load — P_{LL} = 6.48 kip — W16x36, V_{LL} = 6.48 kip
2 stringers/connection — P_{LL} = 12.96 kip

Moment — M = P a — =12.96*6 = 77.76 ft-kip
a = 6 ft

$M_{LL\ PED}$ = — **77.76 ft-kip**

Shear — V = 2 P / 2 — = 2*12.96/2 = — **12.96 kip**

for calculating Cb: M2 = 77.76 ft-kip — max moment at end of braced length

M1 = 77.76 ft-kip — max moment at opposite end of braced length (i.e. at stringer connection)

Vehicle

Assume full wheel line load at both stringers
P_{LL} = 10 kip

Moment — M = P a — =10*6 = 60.00 ft-kip

$M_{LL\ VEHICLE}$ = — =60+0= — **60.00 ft-kip**

Shear
V = 2 P / 2 — = 2*10/2 = — **10.00 kip**

for calculating Cb: M2 = 60.00 ft-kip — max moment at end of braced length

M1 = 60.00 ft-kip — max moment at opposite end of braced length (i.e. at stringer connection)

E	29000	ksi
Fyc / Fyt	50	ksi
Sx	38.4	in^3
Zx	44.2	in^3

Capacity of Stringer

Local Buckling Resistance

λ_f = bfc / (2 tfc)	W16x26		(AASHTO 6.10.8.2.2-3)
b_{fc} = compression flange width	5.5	in	
t_{fc} = compression flange thickness	0.345	in	
d = depth of member	15.7	in	
D_c = depth of web in compression	7.6775	in	
t_w = web thickness	0.25	in	

λ_f =	=5.5/(2*0.345) =	7.97	
$\lambda_{pf} = 0.38*\sqrt{(E/Fyc)}$ =	=0.38*SQRT(29000/50) =	9.15	(AASHTO 6.10.8.2.2-4)

$\lambda_f \le \lambda_{pf}$ YES

if $\lambda_f \le \lambda_{pf}$ then Fnc = Rb Rh Fyc (AASHTO 6.10.8.2.2-1)

R_h = 1.0 for rolled shapes (AASHTO 6.10.1.10.1)

R_b = 1.0 constructability does not need to be checked (AASHTO 6.10.1.10.2)

F_{nc} = =1*1*50= 50 ksi

2*Dc/tw 61.4

$\lambda_{rw} = 5.7\sqrt{(E/Fyc)}$ 137.3

if 2*Dc/tw ≤ λrw YES

$R_{pc} = M_p/M_{yc} = Z_x/S_x$ = 1.15 (AASHTO A6.2.1-4)

$M_{yc} = F_{yc}\, S_x$ = 1920 k-in (AASHTO D6.2)

$M_{nc} = R_{pc}\, M_{yc}$ = 2210 k-in (AASHTO A6.3.2-1)

Lateral Torsional Buckling Resistance W16x26 (AASHTO 6.10.8.2.3)

$L_p = 1.0\ r_t \sqrt{(E/F_{yc})}$ 33.07 in
2.76 ft

$r_t = b_{fc} / (\sqrt{(12*(1+1/3*D_c*t_w / b_{fc} / t_{fc})}$ 1.373 in

L_b = unbraced length = 6 ft 72 in

$L_b \le L_p$ NO

$L_r = \pi\ r_t \sqrt{(E/F_{yr})}$ 124.16 in
10.35 ft

$F_{yr} = 0.7\ F_{yc}$ 35 ksi

$L_b \le L_r$ YES

if $L_b < L_r$ $F_{nc} = C_b\ (1-(1-F_{yr}/R_h/F_{yc})*(L_b-L_p)/(L_r-L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$

F_{nc} =	pedestrian	43.59	ksi
	vehicle	43.59	ksi

if $L_b < L_r$ $M_{nc} = C_b\ (1-(1-F_{yr}S_{xc}/R_{pc}/M_{yc})*(L_b-L_p)/(L_r-L_p)R_{pc}M_{yc} \le R_{pc}M_{yc}$

M_{nc} =	pedestrian	1839.9	k-in
	vehicle	1839.9	k-in

E 29000 ksi
Fyw 50 ksi

Capacity of Floorbeam (continued)

$C_b = 1.75\text{-}1.05(M1/M2)+0.3(M1/M2)^2 \leq 2.3$ (AASHTO A6.3.3-6)

for **pedestrian** load =1.75-1.05*(77.76/77.76)+0.3*(77.76/77.76)^2 = 1.00

for **vehicle** load =1.75-1.05*(60/60)+0.3*(60/60)^2 = 1.00

if $L_b > L_r$, $F_{nc} = F_{cr} \leq R_b R_h F_{yc}$
$F_{cr} = C_b R_b \pi 2E/(L_b/r_t)^2$

Tension Flange Flexural Resistance

$F_{nt} = R_h\ F_{yt}$ 50 ksi

Nominal Shear Resistance of Unstiffened Web

$\phi_v V_n = \phi_v C V_p$ (AASHTO 6.10.9.2-1)

W16x26
$V_p = 0.58\ F_{yw}\ D\ t_w$ 108.82 kip (AASHTO 6.10.9.2-2)

D = depth of web 15.01 in

$D/t_w =$ 60.04

k = 5.0 (given in AASHTO 6.10.9.2)

$1.12\ \sqrt{(E\ k/F_{yw})}$ 60.31

if $D/t_w \leq 1.12\ \sqrt{(E\ k/F_{yw})}$ YES (AASHTO 6.10.9.3.2-4)

C = 1.0

$C\ V_p$ 108.82 kip

Rating Equation (LRFR Method)

$$RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_p P)}{\gamma_{LL} LL(1 + IM)}$$

where:

RF = Rating Factor
C = $\phi_c \phi_s \phi_n R_n$ for strength limit state $\phi_c \phi_s \geq 0.85$
C = f_R for service limit state
ϕ_c = Condition factor
ϕ_s = System factor
ϕ_n = Resistance factor based on construction material
R_n = Nominal Capacity of member
f_R = Allowable Stress per LRFD Specs.

γ_{DC} = Dead load factor for structural components and attachments
DC = Dead load due to structural components and attachments

γ_{DW} = Dead load factor for wearing surface and utilities
DW = Dead load due to wearing surface and utilities

γ_p = Load factor for permanent load
P = Permanent load other than dead loads

γ_{LL} = Live load factor
LL = Live load effect
IM = Dynamic load allowance (Impact)

ϕ_c = 1.00 Good or Satisfactory, BMS Condition 1 or 2
ϕ_s = 1.00
$\phi_c \phi_s$ = 0.85 MBE Table 6A.4.2.4-1

Live Loads:

Pedestrian Load = 90 psf
Vehicle Load = H5 or H10

Summary Dead and Live Load Factors for Prestressed Concrete Bridge

Load Combination Limit State	γ_{DC}	γ_{DW}	γ_P	γ_{LL}
Strength I	1.25	1.50	1.00	1.75
Strength II	1.25	1.50	1.00	-
Service I	1.00	1.00	1.00	-
Service III	1.00	1.00	1.00	1.00

Dynamic Load Allowance (Impact) per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges

IM 0%

REVISION 1

$$RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_p P)}{\gamma_{LL} LL(1 + IM)}$$

Strength I

γ_{DC}	1.25	
γ_{LL}	1.75	Inventory
γ_{LL}	1.35	Operating
ϕ_f =	1.0	(AASHTO 6.5.4.2)
ϕ_v =	1.0	

$\phi_c \phi_s$ = 0.85

Floorbeam - Flexure

Capacity, C, based on controlling case of local buckling and lateral torsional buckling

	W16x26	$\phi_c \phi_s$ = 1.00
C = $\phi f M_{nc}$		
pedestrian	1564 k-in	1839.9 k-in
vehicle	1564 k-in	1839.9 k-in
dead		
M_{DC}	174.2 k-in	
γ_{DC} DC	217.7 k-in	
ped		
M_{LL}	933.1 k-in	
γ_{LL} LL inventory	1633 k-in	
γ_{LL} LL operating	1260 k-in	
vehicle		
M_{LL}	720.0 k-in	
γ_{LL} LL inventory	1260 k-in	
γ_{LL} LL operating	972.0 k-in	

	Inventory
RF ped =	(1563.9-217.73)/1632.96
RF vehicle =	(1563.9-217.73)/1260

	$\phi_c \phi_s$ = 0.85		$\phi_c \phi_s$ = 1.00	
	W16x26		W16x26	
	Inventory	Operating	Inventory	Operating
RF ped =	0.82	1.07	0.99	1.29
RF vehicle =	1.07	1.38	1.29	1.67

Floorbeam- Shear

	W16x26
C = $\phi_v V_n$	108.82 kip
V_{DC}	2.48 kip
γ_{DC} DC	3.10 kip
Pedestrian	
V_{LL}	12.96 kip
γ_{LL} LL	22.68 kip
Vehicle	
V_{LL}	10.00 kip
γ_{LL} LL	17.50 kip

RF ped = (108.82-3.1)/22.68

RF vehicle = (108.82-3.1)/17.5

	W16x26
	Inventory
RF ped =	4.66
RF vehicle =	6.04

Floorbeam

L = 17.47 ft
Size W16x26

Dead Loads

Shear reaction from end of stringer
W16x36, V_{DC} = 1.13 kip

Stringers + Deck P_{DC} = 1.13 kip
2 stringers/connection P_{DC} = 2.25 kip

Floorbeam
W16x26 w = 26 lb/ft

Moment $M = w\,l^2 / 8$ floorbeam
W16x26 =(26)*17.47^2/8 = 991.760986 ft-lb 991.8/1000 = 0.99 ft-kip

M = P a =2.25*6 = 13.52 ft-kip
a = 6 ft

M_{DC} = =0.99+13.52+2*0= $\boxed{14.52 \text{ ft-kip}}$

Shear V = (2 P +w l)/ 2
W16x26 = 26*17.46875/2 = 227.09 lb =227.09/1000 = 0.23 kip

Shear from stringers = 2P / 2 = = 2*2.25/2 = 2.25 kip

V_{DC} = $\boxed{2.48 \text{ kip}}$

Live Loads Note: pedestrian load and vehicle load do not act concurently

<u>Pedestrian</u>

Maximum moment and shear occurs when all stringers are loaded
Shear reaction from end of stringer
W16x36, V_{LL} = 6.48 kip

Stringer Live Load P_{LL} = 6.48 kip
2 stringers/connection P_{LL} = 12.96 kip

Moment M = P a =12.96*6 = 77.76 ft-kip
a = 6 ft

$M_{LL\ PED}$ = $\boxed{77.76 \text{ ft-kip}}$

Shear V = 2 P / 2 = 2*12.96/2 = $\boxed{12.96 \text{ kip}}$

for calculating Cb: M2 = 77.76 ft-kip max moment at end of braced length

M1 = 77.76 ft-kip max moment at opposite end of braced length (i.e. at stringer connection)

<u>Vehicle</u>

Assume full wheel line load at both stringers
P_{LL} = 10 kip

Moment M = P a =10*6 = 60.00 ft-kip

$M_{LL\ VEHICLE}$ = =60+0= $\boxed{60.00 \text{ ft-kip}}$

Shear
V = 2 P / 2 = 2*10/2 = $\boxed{10.00 \text{ kip}}$

for calculating Cb: M2 = 60.00 ft-kip max moment at end of braced length

M1 = 60.00 ft-kip max moment at opposite end of braced length (i.e. at stringer connection)

E	29000	ksi
Fyc / Fyt	50	ksi
Sx	38.4	in^3
Zx	44.2	in^3

Capacity of Stringer

Local Buckling Resistance

λ_f = bfc / (2 tfc) W16x26 (AASHTO 6.10.8.2.2-3)

b_{fc} = compression flange width 5.5 in

t_{fc} = compression flange thickness 0.345 in

d = depth of member 15.7 in

D_c = depth of web in compression 7.6775 in

t_w = web thickness 0.25 in

λ_f = =5.5/(2*0.345) = 7.97

$\lambda_{pf} = 0.38\sqrt{(E/Fyc)}$ = =0.38*SQRT(29000/50) = 9.15 (AASHTO 6.10.8.2.2-4)

$\lambda_f \le \lambda_{pf}$ YES

if $\lambda_f \le \lambda_{pf}$ then Fnc = Rb Rh Fyc (AASHTO 6.10.8.2.2-1)

R_h = 1.0 for rolled shapes (AASHTO 6.10.1.10.1)

R_b = 1.0 (AASHTO 6.10.1.10.2)

F_{nc} = =1*1*50= 50 ksi

2*Dc/tw 61.4

$\lambda_{rw} = 5.7\sqrt{(E/Fyc)}$ 137.3

if 2*Dc/tw ≤ λrw YES

$R_{pc} = M_p/M_{yc} = Z_x/S_x$ = 1.15 (AASHTO A6.2.1-4)

$M_{yc} = F_{yc}\ S_x$ = 1920 k-in (AASHTO D6.2)

$M_{nc} = R_{pc}\ M_{yc}$ = 2210 k-in (AASHTO A6.3.2-1)

Lateral Torsional Buckling Resistance W16x26 (AASHTO 6.10.8.2.3)

$L_p = 1.0\ r_t \sqrt{(E/F_{yc})}$ 33.07 in
2.76 ft

$r_t = b_{fc} / (\sqrt{(12*(1+1/3*D_c*t_w / b_{fc} / t_{fc})}$ 1.373 in

L_b = unbraced length = 0 ft 0 in New deck connections ensure that compression flange is fully braced

$L_b \le L_p$ YES

if $L_b < L_p$ $F_{nc} = R_bR_hF_{yc}$ 50 ksi

if $L_b < L_p$ $M_{nc} = R_{pc}M_{yc}$ 2210 k-in

$L_r = \pi\ r_t \sqrt{(E/F_{yr})}$ 124.16 in
10.35 ft

$F_{yr} = 0.7\ F_{yc}$ 35 ksi

$L_b \le L_r$ YES

if $L_b < L_r$ $F_{nc} = C_b\ (1-(1-F_{yr}/R_h/F_{yc})*(L_b-L_p)/(L_r-L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$

F_{nc} = pedestrian 50.00 ksi
vehicle 50.00 ksi

E 29000 ksi
Fyw 50 ksi

Capacity of Stringer (continued)

$C_b = 1.75\text{-}1.05(M1/M2)+0.3(M1/M2)^2 \leq 2.3$ (AASHTO A6.3.3-6)

~~for **pedestrian** load~~ ~~=1.75-1.05*(77.76/77.76)+0.3*(77.76/77.76)^2 =~~ ~~1.00~~

~~for **vehicle** load~~ ~~=1.75-1.05*(60/60)+0.3*(60/60)^2 =~~ ~~1.00~~

if $L_b>L_r$, $F_{nc} = F_{cr} \leq R_bR_hF_{yc}$
$F_{cr} = C_bR_b\pi 2E/(L_b/r_t)^2$

<u>Tension Flange Flexural Resistance</u>

$F_{nt} = R_h\ F_{yt}$ 50 ksi

<u>Nominal Shear Resistance of Unstiffened Web</u>

$\phi_vV_n = \phi_vCV_p$ (AASHTO 6.10.9.2-1)

W16x26

$V_p = 0.58\ F_{yw}\ D\ t_w$ 108.82 kip (AASHTO 6.10.9.2-2)

D = depth of web 15.01 in

$D/t_w =$ 60.04

k = 5.0 (given in AASHTO 6.10.9.2)

$1.12\ \sqrt{(E\ k/F_{yw})}$ 60.31

if $D/t_w \leq 1.12\ \sqrt{(E\ k/F_{yw})}$ YES (AASHTO 6.10.9.3.2-4)

C = 1.0

$C\ V_p$ 108.82 kip

Rating Equation (LRFR Method)

$$RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_p P)}{\gamma_{LL} LL(1 + IM)}$$

where:

RF = Rating Factor
C = $\phi_c \phi_s \phi_n R_n$ for strength limit state $\phi_c \phi_s \geq 0.85$
C = f_R for service limit state
ϕ_c = Condition factor
ϕ_s = System factor
ϕ_n = Resistance factor based on construction material
R_n = Nominal Capacity of member
f_R = Allowable Stress per LRFD Specs.

γ_{DC} = Dead load factor for structural components and attachments
DC = Dead load due to structural components and attachments

γ_{DW} = Dead load factor for wearing surface and utilities
DW = Dead load due to wearing surface and utilities

γ_p = Load factor for permanent load
P = Permanent load other than dead loads

γ_{LL} = Live load factor
LL = Live load effect
IM = Dynamic load allowance (Impact)

ϕ_c = 1.00 Good or Satisfactory, BMS Condition 1 or 2
ϕ_s = 1.00
$\phi_c \phi_s$ = 0.85 MBE Table 6A.4.2.4-1

Live Loads:

Pedestrian Load = 90 psf

Vehicle Load = H5 or H10

Summary Dead and Live Load Factors for Prestressed Concrete Bridge

Load Combination Limit State	γ_{DC}	γ_{DW}	γ_P	γ_{LL}
Strength I	1.25	1.50	1.00	1.75
Strength II	1.25	1.50	1.00	-
Service I	1.00	1.00	1.00	-
Service III	1.00	1.00	1.00	1.00

Dynamic Load Allowance (Impact) per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges

IM 0%

$$RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_p P)}{\gamma_{LL}LL(1+IM)}$$

Strength I

γ_{DC}	1.25	
γ_{LL}	1.75	Inventory
γ_{LL}	1.35	Operating
ϕ_f =	1.0	(AASHTO 6.5.4.2)
ϕ_v =	1.0	

$\phi_c \phi_s$ = 0.85

Floorbeam - Flexure

Capacity, C, based on controlling case of local buckling and lateral torsional buckling

	W16x26
C = $\phi f \, M_{nc}$	
pedestrian	1879 k-in
vehicle	1879 k-in
dead	
M_{DC}	174.2 k-in
γ_{DC} DC	217.7 k-in
ped	
M_{LL}	933.1 k-in
γ_{LL} LL inventory	1633 k-in
γ_{LL} LL operating	1260 k-in
vehicle	
M_{LL}	720.0 k-in
γ_{LL} LL inventory	1260 k-in
γ_{LL} LL operating	972.0 k-in

Inventory

RF ped = (1878.5-217.73)/1632.96

RF vehicle = (1878.5-217.73)/1260

W16x26	Inventory	Operating
RF ped =	1.02	1.32
RF vehicle =	1.32	1.71

Floorbeam- Shear

	W16x26
C = $\phi_v V_n$	108.82 kip
V_{DC}	2.48 kip
γ_{DC} DC	3.10 kip
Pedestrian	
V_{LL}	12.96 kip
γ_{LL} LL	22.68 kip
Vehicle	
V_{LL}	10.00 kip
γ_{LL} LL	17.50 kip

RF ped = (108.82-3.1)/22.68

RF vehicle = (108.82-3.1)/17.5

W16x26	Inventory
RF ped =	4.66
RF vehicle =	6.04

Floorbeam Cont. Braced

Timber Plank Decking	G =	0.55 Specific Gravity	(NDS Table 11.3.2A)
3 ⅜"x7⅜" net dimensions	γ_{water}	62.4 lb/ft^3	
Southern Pine	γ_{timber}	34.32 lb/ft^3	
Assume Southern Pine No. 2	σ_{timber}	9.65 psf	

Check timber planks as simple spans between steel stringers — Timber Plank Dimensions

L =	6 ft	Stringer spacing	d = depth	3.375 in
L clr =	6-5.5/12		w = width	7.375 in
L clr =	5.54 ft			
Width of nailer	5.5 in			

Dead Loads deck = 7.375/12 ft * 9.6525 psf w = 5.930 lb/ft

Moment $M = w\,l^2 / 8$ =5.93*5.54^2/8 = 22.75 ft-lb

Shear $V = w\,l / 2$ = 5.93*5.54/2=" 16.43 lb

Live Loads Note: pedestrian load and vehicle load do not act concurently

Pedestrian 90 psf (LFRD Ped Bridge 3.1)

w = =90*7.375/12 = 55.31 lb/ft

Moment $M = w\,l^2 / 8$ =55.3125*5.54^2/8 = 212.20 ft-lb

Shear $V = w\,l / 2$ = 55.3125*5.54/2 = 153.2 lb

Vehicle H10 (LFRD Ped Bridge 3.2)

wheel 1	8 kip	8000 lb	Use maximum of wheel loads
wheel 2	2 kip	2000 lb	

for planks < 10" wide, reduce wheel load by ratio of (w_p/10") (AASHTO 4.6.2.1.3)

w_p = width of plank 7.375 in

wheel load (wp/10") 5900 lb

Moment maximum moment occurs when wheel is at midspan

distribute wheel load over 20" = 1.67 ft

w = 3532.93413 lb/ft

R = 2950 reaction at stringer

∑M midspan =2950*5.54/2-3532.93*1.67/2*1.67/4 = 6940 ft-lb

Shear for calculating max. design shear, live load shall be place at dist. From support = min(3d, 1/4 L)

3d = 0.84 ft (AASHTO 4.6.2.2.2a)

1/4L = 1.39 ft

min = 0.84 ft

sum moments at opposite support

"b" = l_{clr} - min(3d, 1/4L) 4.70 ft

$V_{LD} = Pb / l$ 5001.42 lb

V_{LU} = 6781.58845 =5001*8000/5900

V_{LL} = $0.5[.6V_{LU}+V_{LD})$ (AASHTO 4.6.2.2.2a-1)

V_{LL} = 4535 lb

Capacity of Timber Plank (Deck)

$Mr = \phi\ Mn$ (AASHTO 8.6.1-1)

$Mn = Fb\ S\ C_L$ (AASHTO 8.6.2-1)

$\phi =$ 0.85 flexure

C_L = 1.0 when depth < width 1.0 (AASHTO 8.6.2)

S = section modulus

$S = bd^2 / 6 =$ =7.375*3.375^2/6

14.00 in^3

$F_b = F_{bo}\ C_{KF}\ C_M\ C_F\ C_{fu}\ C_i\ C_d\ C_\lambda$

F_{bo} =	1.2 ksi		(AASHTO Table 8.4.1.1.4-1)	
C_{KF} =	2.5/ϕ =	2.94		format conversion factor
C_M =	0.85			wet service factor
C_F =	1	already adjusted for southern pine		size factor for visually graded lumber
C_{fu} =	1.05			flat use factor
C_i =	0.8			incising factor
C_d =	1.15			deck factor
C_λ =	0.8	for Strength I		time effect factor

F_{bo} = 2.3184 ksi

Mn = 32.46 kip-in

2705 ft-lb

ϕ = 0.75 shear (AASHTO 8.5.2.2)

Shear parallel to grain

$F_v = F_{vo}\, C_{KF}\, C_M\, C_i\, C_\lambda$ (AASHTO 8.4.4.1)

F_{vo} =	0.175 ksi		AASHTO Table 8.4.1.1.4-1
C_{KF} =	2.5/ϕ =	3.33	(AASHTO 8.4.4.2)
C_M =	0.97		(AASHTO 8.4.4.3)
C_i =	0.8		(AASHTO 8.4.4.7)
C_λ =	0.8		(AASHTO 8.4.4.9)

F_v = 0.36 ksi
362.1 psi

b = 7.375
d = 3.375

V_n = F_v b d /1.5 (AASHTO 8.7-2)
6009 lb

$C = \phi_c \phi_s \phi_n R_n$ for strength limit state $\phi_c \phi_s \geq 0.85$

$$RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_p P)}{\gamma_{LL} LL(1 + IM)}$$

ϕ_s = 1.0 Deck is redundant system

Strength I

γ_{DC}	1.25	
γ_{LL}	1.75	Inventory
γ_{LL}	1.35	Operating
ϕ_f =	0.85	
ϕ_v =	0.75	

Deck - Flexure

ϕ_c = condition factor		
Good	Fair	Poor
1	0.95	0.85
C = $\varphi_c\varphi_s\phi f\ M_n$ (ft-lb)		
2299.24	2184.28	1954.35

γ_{DC} DC 28.44 ft-lb

Pedestrian

			RF		
Inventory γ_{LL} LL	371.36	ft-lb	6.11	5.81	5.19
Operating γ_{LL} LL	286.47	ft-lb	7.93	7.53	6.72

Vehicle

			RF		
Inventory γ_{LL} LL	12144.8	ft-lb	0.19	0.18	0.16
Operating γ_{LL} LL	9368.8	ft-lb	0.24	0.23	0.21

for vehicles
RF x wheel load = maximum wheel for RF = 1.0

	Max Wheel Load (lb)		
Inventory	1496	1420	1269
Operating	1939	1841	1645

Floorbeam- Shear

ϕ_c = condition factor		
Good	Fair	Poor
1	0.95	0.85
C = $\varphi_c\varphi_s\phi v\ V_n$ (lb)		
4506.86	4281.52	3830.83

γ_{DC} DC 16.43 lb

Pedestrian

			RF		
Inventory γ_{LL} LL	268.13	lb	16.75	15.91	14.23
Operating γ_{LL} LL	206.84	lb	21.71	20.62	18.44

Vehicle

			RF		
Inventory γ_{LL} LL	7936.6	lb	0.57	0.54	0.48
Operating γ_{LL} LL	6122.5	lb	0.73	0.70	0.62

for vehicles
RF x wheel load = maximum wheel for RF = 1.0

	Max Wheel Load (lb)		
Inventory	4526	4299	3845
Operating	5867	5573	4984

APPENDIX D

UNDERWATER INSPECTION REPORT - SEE SEPARATE FILE

APPENDIX E

PHOTOGRAPH LOG

PHOTOGRAPH CONTACT SHEET

kpff Consulting Engineers	Project	Riverfront Park Bridges Inspection	By	MLF	Sheet No.
1601 Fifth Avenue, Suite 1600 Seattle, WA 98101	Location	Spokane	Date	8/14/2014	1 OF 1
(206) 622-5822 fax (206) 622-8130	Client	City of Spokane			Job No.
	Inspection Photo Log				114176.12

Bridge Name: Wooden Bridge East

Date of Inspection: 8/13/2014

Photo No.	Location	Notes	By
1786	Pier 9	Abutment wall	TW
1787	Pier 9	Abutment wall	TW
1788	Pier 9	Exposed footing	TW
1789	Pier 9	Exposed footing	TW
1790	Pier 9, Stringer 1A	Cracked grout pad	TW
1791	General	Split in longitudinal deck member	TW
1792	General	Underside of deck	TW
1793	Pier 9	Disconnected conduits	TW
1794	Pier 9	Vertical cracks in abutment wall	TW
1795	General	Split in longitudinal deck member caused by inproper bolt installation	TW
1796	General	Piers/columns, stringers, floorbeams	TW
1797	General	Conduits	TW
1798			TW
1799	Deck	Twisted deck boards	TW
1800	Deck		TW
1801	Deck		TW
1802	Deck	Splits in deck boards	TW
1803	Deck	Twisted/uplift deck boards	TW
1804	Deck	Twisted/uplift deck boards	TW
1805	Deck	Twisted/uplift deck boards	TW
1806	Deck	Missing bolt	TW
1807	General	Railing	TW
1808	General	Loose bolts in railing	TW
1809	General	Missing nuts in railing	TW
1810	Elevation	Elevation, looking east	TW
1811	General	Bridge plaque	TW
1812	Deck	Deck, looking north	TW
1986	Elevation	Elevation, looking east	MF
1987	Elevation	Elevation, looking east	MF
1988	Elevation	Elevation, looking east	MF

East Wooden Bridge Photographs


IMG_1988.JPG


IMG_1786.JPG


IMG_1787.JPG


IMG_1788.JPG


IMG_1789.JPG


IMG_1790.JPG


IMG_1791.JPG


IMG_1792.JPG


IMG_1793.JPG


IMG_1794.JPG


IMG_1795.JPG


IMG_1796.JPG


IMG_1797.JPG


IMG_1798.JPG


IMG_1799.JPG


IMG_1800.JPG


IMG_1801.JPG


IMG_1802.JPG

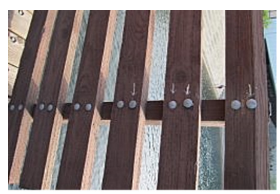

IMG_1803.JPG

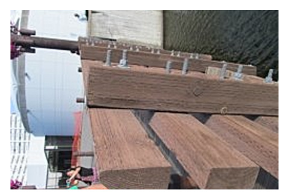

IMG_1804.JPG


IMG_1805.JPG


IMG_1806.JPG


IMG_1807.JPG

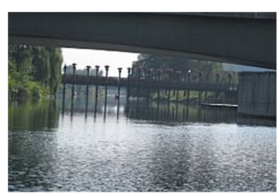

IMG_1808.JPG


IMG_1809.JPG


IMG_1810.JPG

IMG_1811.JPG


IMG_1812.JPG


IMG_1986.JPG


IMG_1987.JPG