

# **RIVERFRONT PARK BRIDGES**

## INSPECTION AND ANALYSIS

# EAST WOODEN BRIDGE

NOVEMBER 14, 2014 | Final Report With Revisions – 12/22/2014



### EAST WOODEN BRIDGE

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

#### **Prepared for**

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#### Prepared by

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#### **Table of Contents**

#### Bridge Inspection and Analysis Report

1.	Bridge Description	1
2.	Document Review	.1
3.	Evaluation Procedures	.2
4.	Evaluation Findings	2
5.	Conclusions and Recommendations	3
6.	Permits and Cultural Resource Requirements	4

#### Appendix A

Bridge Inspection Form Key Photographs Bridge Component Labeling System

#### Appendix B

Bridge Improvement Details Cost Estimate

#### Appendix C

Load Rating Results & Calculations

### Appendix D

Underwater Inspection Report

#### Appendix E

Photograph Log Photograph Contact Sheet

# 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

	PAGE
BRIDGE INSPECTION FORM	A-1

### LIST OF PHOTOGRAPHS

<u>PHOTO</u>	DESCRIPTION	PAGE
1	East Wooden Bridge Deck (Looking North)	A-3
2	East Wooden Bridge Elevation (Looking East)	A-3
3	Missing Bolt in Timber Plank Deck	A-4
4	Rotated Timber Plank, ~3/4-inch Grade Difference	A-4
5	Missing Nuts in Railing Connection	A-5
6	Stringer 8A is not Bearing on Grout Pad	A-5
7	Exposed Footing at Pier 9	A-6
8	Disconnected Conduits Running below the Deck	A-6

### PAGE

BRIDGE COMPONENT LABELING SYSTEM	A-7
BRIDGE COMPONENT LABELING SYSTEM:	
COLUMNS, FLOORBEAMS, AND STRINGERS	A-8



# CITY OF SPOKANE

						Bridge No.	
Bridge Name				Bridge Location			
Inspection Date	Inspector(s)				Agency		
Access Method						Weather	
Load Rating Date			Live Load	Pedestri	an	V	ehicle
Load Rating Factor(s)	Ped.	Veh.	Controlling Component	Pedestrian		Vehicle	
			PN				

### **Description of Bridge**

Summary of Condition and Critical Findings

#### **Summary of Recommendations**

#### **Summary of Bridge Condition**

Bridge Component		No. of	%	Condition Rating*			
		Compon.	of **	8 – 7 Good	6 – 5 Fair	4 – 3 Poor	Comments
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
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**

DESCRIPTION OF CONDITION OF BRIDGE COMPONENT				
Condition Value	Material	Description		
8 – 7	Steel	Like new, surface rust, minor pitting, no material loss. Connections are good. No damage.		
0-1	Concrete	No to minor/ insignificant defects includes: cracks, spalls, chips, consolidation, efflorescence.		
Very good $\rightarrow$ Good	Timber	Beams: Minor splits, checks, or defects (one side), no decay or insects – sounds solid. Posts: Splits or cracks less than $\frac{3}{6}$ " (one side), no decay or insects – sounds solid.		
2 yr. insp. Cycle	Paint	No defects, no sign of rust including no freckled rust, no peeling, no exposed steel.		
No repairs.	Scour / Erosion	None or minor.		
6 – 5	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.		
Satisfactory $\rightarrow$ Fair	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		
1 – 2 yr insp. cycle		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 $\frac{1}{4}$ " wide.		
Monitor for repairs	Timber	Beams: Less than 3/3" splits – two sides or greater than 3/6" on one side. Some decay (max 10% by volume), some softness but sounds solid – no insects. Posts: More than 1/2 "splits – two sides or greater than 3/4" on one side. Decay is evident (greater than 20% by volume), timber may have extensive wetness and softness.		
Paint: Max 10 year life	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	Steel	Heavy to severe: corrosion, pitting, pack rust. Measurable material loss. Connections are heavily corroded, missing, and questionable functionality. Fatigue cracks.		
Poor $\rightarrow$ Critical 3 mo – 1 yr. insp. cycle	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.		
(as needed) Repairs needed. (ASAP or one year)	Timber	Beams: Greater than <sup>3</sup> / <sub>8</sub> " on two sides. Moderate decay up to 20%, surface softness, do not sound solid – may have insects. Posts: Less than <sup>1</sup> / <sub>2</sub> "splits – two sides or greater than <sup>1</sup> / <sub>2</sub> " on one side. Decay is evident (20%), wetness and soft.		
Re - naint	Paint	Extensive freckled rust, larger areas of exposed steel, heavily oxidized, extensive peeling.		
Ne - paint	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



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 PRO	JECT NO.	SHEET NO.
		EASTERN	WASH.			
	PANEL			FINISH	H SAWN	N
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			~ <i>I</i>			
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		1	```	— SIEEL "Z	CLIP	
		" " /	—EX	IST WF		
	E = 2					
GLULAM	DECK PANEL					
SCALE	1 1/2" = 1'-0"					
	RIVERFRONT	PARK	BR	IDGES		
INGTON		0 EG	TYPE OF	IMPROVEMENT:	BRID	GE
RVICES	IRIANGLL AND WOUDEN BRID(	JLJ	CIT	Y PROJECT NUMBER	PLAN N	UMBER
3	DECK DEDI ACENT		2	013186		of I
	DUCK KEFLACEMENI	ORF YOU	EFN:	-800-424-55	55	
	CALL DLI	JIL 100	2.0 1			





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.)







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



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







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.



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	Dry55 FP / Wet .79 FP (ADA Compliant)
Coefficient of Friction - Neolite	ASTM C1028-89	Dry73 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

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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 NYSUFPBC, Art 15, Part 1120,9 NYCRR	LC 50 0f 63.60g.
Combustion Toxicity Test	1120	Pass (19.7g or greater)
Surface Burning Calculated Flame Spread (10	ASTM E84 (2007)	NFPA Class B
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 CalFire Wildlife Urban Interface	Code Section 1511.5 (rooftop decks)	(Approved)
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)



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# $\overline{\underline{IRON WOODS}}$

# CUMARU

### IRON WOODS<sup>®</sup> 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 lpe with exception of its resistance to marine borers.

How does Iron Woods<sup>®</sup> Cumaru compare to other lumber and decking products?

	<u>Cumaru</u>	CCA-Treated Pine	Composite/PVC Decking
Туре	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



# 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,

To learn safe for human and animal contact and meets low VOC emmission standards.

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





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City of Sp	okane Pedestrian Bridges					
Cost Estin	nates for Bridge Improvements Based on the 2014 KF	PFF Inspection and Analys	sis Recommendatio	ns		
<b></b>						
Bridge Na	me:	East Wooden I	Bridge			
Bridge Lei	ngth and Width (feet)	179	37			
Recomme	endations for Improvements - Include:		Deck Replacemen	t		
Option 1	Cast in Place Concrete Deck	Const Unit	Quantita			ltare Cast
Item no	Item Description	Cost Unit	Quantity		ć	Item Cost
1	Existing Rall Remove and Re-Install	LF	358	35	Ş	12,530
2	New Deek	SF	0023	4	Ş	20,492
3	New Deck	SF	0023	25	Ş	105,575
4	IVIISC Total	LS	1	25000	\$	25,000
Option 2	Precast Concrete Panels				Ş	229,397
Item no	Item Description	Cost Unit	Quantity	Unit Cost		Item Cost
1	Existing Rail Remove and Re-install	LF	358	35	Ś	12.530
2	Remove Existing Deck	SF	6623	4	Ś	26.492
3	New Deck	SF	6623	30	Ś	198.690
4	Misc	LS	1	25000	Ś	25.000
-	Total				\$	262,712
Option 3	Glulam Deck Panels/Ironwood Deck*					
ltem 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
5	Mobilization	10%	(of option 2)		ć	26 271
5	Design Permits Survey	20%	(of option 2)		ڊ خ	52 542
7	Construction Management	13%	(of option 2)		ڊ خ	32,342
, 8		8%	(of option 2)		ć	21 017
Q	Contingency	30%	(of option 2)		ç	78 81/
10	Excelation (1 year)	3%	(of option 2)		ç	7 8 8 1
11	Agency Project Development & Mngmt	5%	(of option 2)		ć	13 136
	Total	570	(01 001011 2)		\$	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

Nov-14

\*Ironwood Deck cost is comparable to glulam deck panels

# APPENDIX C

LOAD RATING RESULTS AND CALCULATIONS



Riverfront Park Bridges Inspections & Analysis East Wooden Bridge

### Structural Analysis – Load Rating Summary

### LRFR Bridge Rating Summary

### Strength I – Rating Factors (RF):

	Peo	destrian	Vehicle		
	Inventory	Operating	Inventory	Operating	
Deck RF	5.19	6.72	0.16	0.21	
Controlling Point	Deck	<ul> <li>flexure</li> </ul>	Deck -	flexure	
Steel RF	0 82	1.07	1.07	1 20	
(discretely braced)	0.02	1.07	1.07	1.30	
Controlling Point	Floorbe	am - flexure	Floorbean	n - flexure	
Steel RF	1.02	1 22	1 22	1 71	
(continuously braced)	1.02	1.32	1.32	1.71	
Controlling Point	Floorbe	am - flexure	Floorbean	n - flexure	

Maximum Wheel Live Load:

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

Maximum Pedestrian Live Load for floorbeams braced at stringers: 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



Riverfront Park Bridges Inspections & Analysis East Wooden Bridge

### **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,  $\phi_c$ , is equal to 1.0 for good members and 0.85 for the poor deck members. The system rating factor,  $\phi_s$  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.

10066	Project	Riverfront Park Bridges Inspection & Analysis	Ву	M. Frymoyer	Sheet No.
<b>KPII</b> Consulting Engineers	Location	Spokane	Date	9/9/2014	1 of 20
1601 Eifth Avenue Suite 1600 Seattle WA 08101	Client	City of Spokane			Job No.
(206) 622-5822 fax (206) 622-8130	Wooden Brid	ge East Load Rating			114176

#### Wooden Bridge East Load Rating Summary

Inventory Rating  $\gamma_{LL} = 1.75$ 

### **REVISION 1**

#### Load Rating for Pedestrian Live Load (90 psf)

Interior Stringer - cont. braced compression flange

	Rating Factor
Flexure W16x36	3.21
Shear W16x36	11.22
Elevure W16x26	2 10
TIEXULE WTONED	2.13
Shear W16x26	9.49
Elovuro M/14/22	1.60
Flexure W14x22	1.02
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

	Flexure W16x26	1.32
Timber	Deck	
1111001	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

Floor Beam - Continously braced compression flange

Load Rating for Vehicle Live Load (H10)

Flexure W16x36

Flexure W16x26

Shear W16x26

Flexure W14x22

Shear W14x22

Shear W16x26

Floor Beam - braced at stringers Flexure W16x26

Shear W16x36

Interior Stringer - cont. braced compression flange

Rating Factor

2.60

8.23

1.77

6.96

1.31

5.54

1.07

6.04

Minimum Rating Factor	0.16
Controlling Component	Deck, Flexure

#### Operating Rating $\gamma_{LL} = 1.35$

0.82

1.07

Floorbeam Flexure

Floor Beam - braced at stringers Flexure W16x26

Minimum Rating Factor

Controlling Component

Market Reserver         Market Reserver         Market Reserver         Market Reserver         Market Reserver           Worden Bidge East Loads         0.05 Specific Gravity Vaca         (ADS Table 11:3.24)         REVISION 1           Str.XXF and dimensions Str.XXF and dimensions         0.05 Specific Gravity Vaca         (ADS Table 11:3.24)         REVISION 1           Interior Stringer         L - 24 H Specing - 6 H Size         2.5 L H <sup>2</sup> Viscol         (ADS Table 11:3.24)         REVISION 1           Dead Loads         dock - 6 H 1 9.625 pdf - w - 57.42 bit singen         9.6 J Size         0.1 Size         Without Print           Without Print         L - 24 H Specing - 6 H         Specing - 6 H         57.42 bit singen         11.1 Size         Without Print           Without Print         L - 7.4 H Specing - 6 H         Specing - 6 H         57.42 bit singen         11.1 Size         11.1 Size           Without Print         M - 7.4 H Without Print         Without Print         N - 52 bit Without Print         57.51 Mip           Without Print         M - 7.4 H Without Print         Without Print         57.53 Mip         57.53 Mip           Without Print         M - 57.54 Size Size         57.53 Mip         57.53 Mip         57.53 Mip           Without Print         M - 57.64 Mip         11.1 Size         11.1 Size         11.1 Size </th <th>k p f f</th> <th>a Engineera</th> <th>Project</th> <th>Riverfront Park Bridges I</th> <th>nspection &amp; Analysis</th> <th>By</th> <th>M. Frymoyer</th> <th>Sheet No.</th>	k p f f	a Engineera	Project	Riverfront Park Bridges I	nspection & Analysis	By	M. Frymoyer	Sheet No.
Decision was was and the state of	<b>N P I I</b> Consultin	iy Liigineeis	Client	Spokane City of Spokane		Dale	9/9/2014	2 01 20
Those Plank Decking 3 x7.X7, Find dimensions 3 x7.X7, Find dimensions 3 x7.X7, Find dimensions Southern Pline         G =	1601 Fifth Avenue, Suite 1600 Se (206) 622-5822 fax (206) 622-813	attle, WA 98101 10	Wooden Brid	ge East Load Rating				114176
Tarbox Doxing Southorn Pine         0			•	0 0				
	Timber Diank Deaking		0	0.55.0				<b>REVISION 1</b>
Southon Pine         Name         State list?           Interior Stringer         L=         2.4 ft           Security =         0 ft           Size         W1623           Dead Loads $deck$ -6 ft * 6.852 psi - w - 57.92 [MT]           singles         W1626         w - 28 bit           W1626         w - 28 bit         W1626           W1626         -60.57 915/24/28 - 675.68 ht bit         673.68 ht bit           W1626         -60.57 915/24/28 - 675.68 ht bit         673.68 ht bit           Stear         W1626         -60.57 915/24/28 - 675.68 ht bit           W1626         -60.57 915/24/28 - 675.68 ht bit         675.68 ht bit           W1626         -60.57 915/24/28 - 675.68 ht bit         675.68 ht bit           W1626         -675.915/24/28 - 675.68 ht bit         685.68 ht bit           W1626         -60.57 915/24/28 - 60.58 ht bit         685.68 ht bit           Live Loads         Nete: potestrian load and whick load do not accommentit           Live Loads         Nete: potestrian load and whick load do not accommentit           Morrey	3 3/2 1/2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	he	G =	0.55 Specific Gr	avity (NDS	5 Table TT.3.2A)		
underlief if the boots         true of a description of the boots           interior Stringer         L = 24 ft = 50000 = 6 ft = 0.0000000000000000000000000000000000	S /s X/ /s Thet unitension	15	Ywater	02.4 10/11				
$ \frac{1}{1000} = \frac{1}{1000} = \frac{1}{1000} $ $ \frac{1}{1000} = \frac{1}{10000} = \frac{1}{100000} = \frac{1}{1000000} = \frac{1}{10000000} = \frac{1}{100000000} = \frac{1}{10000000000000000000000000000000000$	Southern Fille		Ytimber	34.32 ID/It				
Inviro Stringer         L= Size         24 tr Wisses           Ded Loads			$\sigma_{timber}$	9.65 pst				
Interior Stringer								
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Interior Stringer							
$ \frac{1}{2} 1$	interior othinger		1 -	24 ft				
$ \frac{1}{12} $			E =	24 ft				
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Dead Loads $ \frac{deck}{deck} = 6   1^{\circ} 9.6525 \text{ pr} = w = 57.92 \text{ bT} $ $ \frac{deck}{deck} = 6   1^{\circ} 9.6525 \text{ pr} = w = 36 \text{ bT} $ $ \frac{deck}{deck} = 6   1^{\circ} 9.6525 \text{ pr} = w = 36 \text{ bT} $ $ \frac{deck}{deck} = 20 \text{ cT} $ $ \frac{deck}{deck} $ $ \frac{deck}{deck} $ $ \frac{deck}{deck} $ $ \frac{deck}{deck} $ $ deck$			3120	W16x26				
Pard Loads         desk         - 617 9.8525 pd - w - 57.5 b h           Singer $0.000 + $				W 10X20				
Dead Loads $ \begin{array}{ccccccccccccccccccccccccccccccccccc$								
$\frac{dek}{der} = 61^{+9}.6328 \text{ ps}^{-1} \text{ w}^{-1} 57.32 \text{ b}^{+1}$ $\frac{dek}{der} = 36 \text{ b}^{+1}, 36.328 \text{ ps}^{-1} \text{ w}^{-1} 36 \text{ b}^{+1}, 36.328 \text{ ps}^{-1} 166 \text{ b}^{-1} 36.1000 \text{ c}^{-1} 16.189 \text{ ps}^{-1} 166.388 \text{ ps}^{-1} 160.65.891.000 \text{ c}^{-1} 30.188 \text{ ps}^{-1} 166.388 \text{ ps}^{-1} 166.3888 \text{ ps}^{-1} 166.38888 \text{ ps}^{-1} 166.38888 \text{ ps}^{-1} 166.388888 \text{ ps}$	Dead Loads							
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			deck	= 6 ft * 9 6525 psf =	w =	57 92 lb/ft		
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			stringers			07102 10/10		
$ \frac{1}{10^{10} \text{ M}^{2}} = \frac{1}{2} \frac$			W16v36		\M/	36 lb/ft		
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			W16x26		W =	26 lb/ft		
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			W14x22		w –	20 lb/ft		
$ \begin{array}{c} \mbore{l} Moment & M = wF / 8 \\ & W18x36 = (36+57.915)^{24/28} = & 676.188 11-b & 6761.88/1000 = & 6.76 15.40p \\ & W18x26 = (26+57.915)^{24/28} = & 753.88 11-b & 6751.88/1000 = & 5.75 15.40p \\ & W18x26 = (27.915)^{24/28} = & 753.88 11-b & 5753.88/1000 = & 5.75 15.40p \\ & W18x26 = (57.915)^{24/28} = & 753.88 1b & 1126.98/1000 = & 1.13 18p \\ & W18x26 = (57.915)^{24/28} = & 958.98 b & 958.98/1000 = & 0.96 hp \\ & W18x26 = (57.915)^{24/28} = & 958.98 b & 958.98/1000 = & 0.96 hp \\ & W18x26 = (57.915)^{24/28} = & 958.98 b & 958.98/1000 = & 0.96 hp \\ & W18x26 = (57.915)^{24/28} = & 958.98 b & 958.98/1000 = & 0.96 hp \\ & W18x26 = (57.915)^{24/28} = & 540 br1 \\ & W18x26 = (57.915)^{24/28} = & 38880 ft-b & 38830/1000 = & 38.88 ft-4p \\ & & & 90^{\circ}6 & 540 br1 \\ & & & & & & & & & & & & \\ & & & & & $			VV 14722		w –			
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		Moment	M = w l <sup>-</sup> / 8					
$ \frac{1}{124} = \frac{1}{126} + 1$			W16x36	=(36+57.915)*24^2/8 =	6761.88 ft-lb	6761.88/10	00 = 6.7	6 ft-kip
$ \frac{1}{2} 1$				· · · ·				
$ W14x22 = (22+57.915)'24'2.8 = 5753.88 11:b 5753.88'1100 = 5.75 11:k1p \\ W16x26 = (57.915+26)'24/2 = 1126.98 ib 1126.98'1000 = 1.13' k1p \\ W16x26 = (57.915+26)'24/2 = 958.98 ib 958.98'1000 = 0.96' k1p \\ W14x22 = (57.915+22)'24/2 = 958.98 ib 958.98'1000 = 0.96' k1p \\ W14x22 = (57.915+22)'24/2 = 958.98' ib 958.98'1000 = 0.96' k1p \\ W14x22 = (57.915+22)'24/2 = 958.98' ib 958.98'1000 = 0.96' k1p \\ W14x22 = (57.915+22)'24/2 = 958.98' ib 958.98'1000 = 0.96' k1p \\ W14x22 = (57.915+22)'24/2 = 954.98' ib 958.98'1000 = 0.96' k1p \\ W14x22 = (57.915+22)'24/2 = 90' paf (LFRD Ped Bridge 3.1) \\ W = -90'6 = 540' ib T \\ Moment M = W'/B \\ = -540'24'2/8 = 38880' ft:b 38880'1000 = 38.88' ft:k1p \\ 5hear V = w1/2 = 540'24/2 = 6480 ib 6480' 1000 = 64.48' k1p \\ Shear V = w1/2 = 540'24/2 = 6480 ib 6480' 1000 = 64.48' k1p \\ Wheel 1 & 8 k1p \\ wheel 2 & 2 k1p \\ spacing 1 & 1 t \\ Wheel 1 & 8 kip \\ wheel 2 & 2 kip \\ spacing 1 & 1 t \\ For interior stringers: distributed level koads to each stringer (AASHTO 4.6.2.22a) \\ Moment \\ Merment \\ M = P1/4 \\ For interior stringers: distributed linger stringers (AASHTO 4.6.2.22a) \\ Marment \\ M = P1/4 \\ Shear maximum moment occurs when 8 kip wheel is at midgpan \\ M = P1/4 \\ Shear maximum shear occrus when 8 kip wheel is over support sum moments at opposite support sub moments at opposi$			W16x26	=(26+57.915)*24^2/8 =	6041.88 ft-lb	6041.88/10	00 = 6.0	4 ft-kip
$\begin{array}{cccccccccccccccccccccccccccccccccccc$								
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			W14x22	=(22+57.915)*24^2/8 =	5753.88 ft-lb	5753.88/10	00 = 5.7	5 ft-kip
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		Shear	V = w I / 2					
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			W16x36	= (57.915+36)*24/2 =	1126.98 lb	1126.98/10	00 = 1.1	3 kip
W 1622 = $(57.915+26)'242 = 106.98 \text{ lb} 1066.98 \text{ lb} 006.98 \text{ lb} 006.98 \text{ lb} 000 = 1.01 \text{ kp}$ W 14x22 = $(57.915+22)'242 = 958.98 \text{ lb} 958.98/1000 = 0.96 \text{ kp}$ Live Loads Note: pedestrian load and vehicle load do not act concurrently Pedestrian $90 \text{ psf}$ (LFRD Ped Bridge 3.1) $w = =90^{\circ}6 = 540 \text{ lb/t}$ $w = =90^{\circ}6 = 540 \text{ lb/t}$ $w = = -540^{\circ}24'2/8 = 38880 \text{ ft-lb} 38880/1000 = 38.88 \text{ ft-kip}$ Shear $V = w 1/2 = 540^{\circ}24'2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 2 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24'2 = 8 \text{ lp} \text{ lb} 0 \text{ lp} 0 $								
W14x2 = (57,915+22)*24/2 =958.98 lb958.98 l0958.98/1000 =0.96 klpLive LoadsNote: pedestrian90 pdf(LFRD Ped Bridge 3.1)Memerit $w = = 90^{\circ}6 = 540$ lb/lt $w = = 90^{\circ}6 = 540$ lb/ltMomerit $M = w l^2/3$ 38880 ft-lb38880/1000 =38.88 ft-klpShear $V = w 1/2$ $e 540^{\circ}24^{\prime}228 = 38800$ lb $6480/1000 = 6480/1000 = 6488 klp6480/1000 = 6488 klpVehicleH10(LFRD Ped Bridge 3.2)Clear deck > 10 ftwheel 18 klpwheel 22 klpspacing4 t ttFor interior stringers: distribute one line of wheel loads to each stringer(AASHTO 4.6.2.2.2a)ASHTO 4.6.2.2.2a)MomentM = P1/4B klp wheel is ver supportMar, 1 = B^{\circ}24/4 = 48 klp-ftB klp wheel is ver supportSM_{rit} = 2 klp (24 ft)B klg wheel is ver supportSM_{rit} = 2 klp (24 ft)B klg wheel is ver supportSM_{rit} = 2 klp (24 ft)B klg wheel is ver supportSM_{rit} = 2 klp (24 ft)B klg wheel is ver supportSM_{rit} = 2 klp (24 ft)B klg wheel is ver supportSM_{rit} = 2 klp (24 ft)B klg klp - ft$			W16x26	= (57.915+26)*24/2 =	1006.98 lb	1006.98/10	00 = 1.0	1 kip
Live Loads Note: pedestrian load and vehicle load do not act concurently Pedestrian $90 \text{ psf}  (LFRD \text{ Ped Bridge 3.1})$ $w = =90^{\circ}6= 540 \text{ lb/ft}$ $Moment  M = w F'/8 = 540^{\circ}24'2/8 = 38880 \text{ ft-lb} 38880'1000 = 38.88 \text{ ft-kip}$ $Shear  V = w1/2 = 540^{\circ}24'2/8 = 6480 \text{ lb} 6480'1000 = 6448 \text{ kip}$ $V = w1/2 = 540^{\circ}24'2/8 = 6480 \text{ lb} 6480'1000 = 6448 \text{ kip}$ $V = w1/2 = 540^{\circ}24'2/8 = 28 \text{ kip} \text{ the load Bridge 3.2}$ $Clear deck > 10 \text{ ft} \text{ wheel 1 } 8 \text{ kip} \text{ wheel 2 } 2 \text{ kip} \text{ spacing 14 ft}$ For interior stringers: distribute one line of wheel loads to each stringer $M = P1/4 \qquad = 8'24/4 = 48 \text{ kip-ft}$ $Shear \qquad maximum shear occrus when 8 \text{ kip wheel is over support} \text{ sum moments at opposite support} 2M_{\text{H}1} = 2 \text{ kip}(10 \text{ ft}) + 8 \text{ kip}(24 \text{ ft}) + R (24 \text{ ft}) = (2^{\circ}10 \text{ s}^{\circ}24)/24 \qquad 8.83 \text{ kip}$			W1400	(EZ 01E, 00)*04/0		050 00/100		C kin
Live Loads       Note: pedestrian load and table load of out and concurrently         Pedestrian $90 \text{ psf}$ (LFRD Ped Bridge 3.1)         We = $90^{\circ}6^{\circ}$ $540 \text{ lo/ft}$ Moment       W = $\sqrt{-540^{\circ}24^{\circ}2/8}$ $38880 \text{ ft-lib}$ $38880/1000 =$ $38.88 \text{ ft-kip}$ Shear       V = w 1/2 $540^{\circ}24/2/8 =$ $6480 \text{ lo}$ $6480/1000 =$ $6.48 \text{ kip}$ Vehicle       H10       (LFRD Ped Bridge 3.2)       Clear dock > 10 ft wheel 1 $8 \text{ kip}$ $8 \text{ kip}$ Vehicle       H10       (LFRD Ped Bridge 3.2)       Clear dock > 10 ft wheel 2 $2 \text{ kip}$ $2 \text{ kip}$ For interior stringers: distribute one line of wheel loads to each stringer       (AASHTO 4.6.2.2.2a)         Moment $8 \text{ kip}$ $8 \text{ kip}$ $8^{\circ}2^{\circ}4^{\circ}4 =$ $48 \text{ kip-ft}$ Shear       maximum moment occurs when 8 kip wheel is at midspan $M = P1/4$ $8^{\circ}2^{\circ}4^{\circ}4 =$ $48 \text{ kip-ft}$ Shear       maximum strat opposite support sum moments at opposite support 			VV 14XZZ	= (57.915+22) 24/2 =	900.90 ID	956.96/100	0.9	5 кір
Note: production and very finite test contraction with the contr	Live Loads	Note: nedes	trian load and v	vehicle load do not act co	ncurently			
Pedestrian90 psf(LFRD Ped Bridge 3.1) $w = = 90^{\circ}6 = 540 \text{ loft}$ Moment $M = w^{1/2}$ $h = w^{1/2}$ $= 540^{\circ}24^{\circ}2/8 = 38880 \text{ ft-lb} 38880/100 = 38.88 \text{ ft-kip}$ Shear $V = w^{1/2} = 540^{\circ}24/2 = 6480 \text{ lb} 6480 \text{ lo0} = 648 \text{ kip}$ Shear $V = w^{1/2} = 540^{\circ}24/2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ VehicleH10(LFRD Ped Bridge 3.2)Clear deck > 10 ft wheel 1 & 8 kip spacing in 14 ftFor interior stringers: distribute one line of wheel loads to each stringer(AASHTO 4.6.2.2.2a)Moment $m_{1} = P_{1/4}$ $m_{1} = P_{1/4}$ $m_{2} = 10^{\circ} m_{1} + 10^{\circ} m_{2} + 10^{\circ} m_{1} + 10^{\circ} $		Note: peace			neurentry			
$w = = 90^{\circ}6 = 540 \text{ lb/f}$ $Moment$ $M = w l^{2}/3$ $= 540^{\circ}24^{\circ}2/8 = 38680 \text{ ft-lb} 36860/1000 = 38.88 \text{ ft-kip}$ $= 540^{\circ}24^{\circ}2/8 = 36680 \text{ ft-lb} 36860/1000 = 6.48 \text{ kip}$ $Shear$ $V = w l/2 = 540^{\circ}24/2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $Shear$ $V = w l/2 = 540^{\circ}24/2 = 6480 \text{ lb} 6480/1000 = 6.48 \text{ kip}$ $Clear deck > 10 \text{ ft}$ $wheel 1 \qquad 8 \text{ kip}$ $wheel 2 \qquad 2 \text{ kip}$ $spacing 14 \text{ ft}$ For interior stringers: distribute one line of wheel loads to each stringer $Moment$ $Moment$ $M = P l/4 \qquad = 8^{\circ}24/4 = 48 \text{ kip-ft}$ $Shear$ $maximum shear occrus when 8 \text{ kip wheel is over support}$ $Sum moments at opposite support Sum = 2 \text{ kip}(10 \text{ ft}) + 8 \text{ kip}(24 \text{ ft}) \cdot R2 (24 \text{ ft})$ $= (2^{\circ}10+8^{\circ}24)/24 \qquad 8.83 \text{ kip}$	Pedestrian			90 psf	(LFR	D Ped Bridge 3.1)		
$w = -90^{\circ}6 = 540 \text{ b/ft}$ $Moment  M = w \ F / 8$ $= 540^{\circ}24^{\circ}2/8 = 3880 \text{ ft-lb}  3880/1000 = 38.88 \text{ ft-kip}$ $Shear  V = w 1/2 = 540^{\circ}24/2 = 6480 \text{ lb}  6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24/2 = 6480 \text{ lb}  6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24/2 = 6480 \text{ lb}  6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24/2 = 2640 \text{ lb}  6480/1000 = 6.48 \text{ kip}$ $V = w 1/2 = 540^{\circ}24/2 = 2640 \text{ lb}  6480/1000 = 6.48 \text{ kip}$ $Wheel 2 = 2 \text{ kip}$ $Shear  V = w 1/2 = 540^{\circ}24/2 = 2640 \text{ lb}  6480/1000 = 6.48 \text{ kip}$ $Wheel 2 = 2 \text{ kip}$ $Shear  V = w 1/2 = 540^{\circ}24/2 = 2640 \text{ lb}  860 \text{ lb} = 6400 \text{ lb} = 64000 \text{ lb} = 640000 \text{ lb} = 6400000 \text{ lb} = 64000000 \text{ lb} = 6400000000000000000000000000000000000$								
$Moment  M = w f' / 8$ $= 540^{\circ}24^{\circ}2/8 = 38880 \text{ ft-lb} 38880 / 1000 = 38.88 \text{ ft-kip}$ $Shear  V = w 1/2 = 540^{\circ}24/2 = 6480 \text{ lb} 6480 / 1000 = 6.48 \text{ kip}$ $Moment  V = w 1/2 = 540^{\circ}24/2 = 6480 \text{ lb} 6480 / 1000 = 6.48 \text{ kip}$ $Vehicle \qquad H10 \qquad (LFRD Ped Bridge 3.2)$ $Clear deck > 10 \text{ ft} wheel 1 \qquad 8 \text{ kip} wheel 2 \qquad 2 \text{ kip} spacing \qquad 14 \text{ ft}$ For interior stringers: distribute one line of wheel bads to each stringer (AASHTO 4.6.2.2.2a) $Moment \qquad for this stringer length, maximum moment occurs when 8 kip wheel is at midspan M = P1/4 = 8'24/4 = 48 \text{ kip-ft}$ $Shear \qquad maximum shear occurs when 8 kip wheel is over support SMHT = 2 kip(10 \text{ ft}) + 8 kip(24 \text{ ft}) - R2 (24 \text{ ft}) = (2'10+8'24)/24 (8.83 \text{ kip})$			w =	=90*6=	540 lb/ft			
MomentM = w l²/ 8 $=540^{\circ}24^{\circ}2/8 =$ $38880$ ft-lb $38880/1000 =$ $38.88$ ft-kipShearV = w l / 2 $=540^{\circ}24/2 =$ $6480$ lb $6480/1000 =$ $6.48$ kipWehicleH10(LFRD Ped Bridge 3.2) $Clear deck > 10$ ftwheel 1 $8$ kipwheel 22 kip $gaacing$ 14 ftFor interior stringers: distribute one line of wheel loads to each stringer(AASHTO 4.6.2.2.2a)Momente8'24/4 = $48$ kip-ftShearmaximum shear occrus when 8 kip wheel is over support $SM_{R1} = 2$ kip(10 ft) + 8 kip (24 ft) - R2 (24 ft) $=(2*10+8*24)/24$ $8.83$ kip								
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$\sum_{i=1}^{N}  i_{R1} ^{2} = 2 \exp(10 it) + 0 \exp(24 it) - \frac{1}{2} (24 it) = (2^{*}10 + 8^{*}24)/24 $ $= (2^{*}10 + 8^{*}24)/24 $ $8.83 \text{ kip}$			5M 0.14-7	sum moments at opposit	e support			
=(2 <sup>-1</sup> 0+8 <sup>-</sup> 24)/24 8.83 Kip			$\sum_{k=1}^{\infty} r_{k1} = 2 r_{k1} \rho(k)$	10 IL) + 0 KIP (24 IL) - M2 (	( <b>-</b> 7 11)	(0*10.0*0	4)/04	9 kin
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$ \begin{array}{c} [ \mbox{Trial} Tr$	Avenue, Suite 1600 Seattle, WA 98101	Client	City of Spok	ane A Dating					Job No.
$ \begin{array}{c c c c c c } & 2000 \ \mbox{ bis} \\ F_{V} \in F_{V} & 50 \ \mbox{ bis} \\ F_{V} \in F_{V} & 50 \ \mbox{ bis} \\ F_{V} \in F_{V} & 50 \ \mbox{ bis} \\ F_{V} = Call Locking Resistance & F_{V} = 1.0 \\ F_{V} = Call Locking Resistance & VI & 5.0 \\ F_{V} = Call Locking Resistance & 7.52 & 7.565 & 6.515 \ \mbox{ bis} \\ F_{V} = campession Intege With & 6.99 & 6.5 & 5.15 \ \mbox{ bis} \\ F_{V} = campession Intege With (F_{V} = 0.028 \ \mbox{ bis} & 0.335 \ \mbox{ bis} & $	5822 fax (206) 622-8130	wooden Brid	bge East Load	Rating					114176
Fyc         Fyc         S0 kal         PPC / Fyr         S0 kal           Ivy OF Stringer         Ive OF Stringer         Rb = 1.0         (ASHTO 6.10.8.2.2.3)           b_n = compression flange with         6.99         5.5         5.1           b_n = compression flange with         6.99         0.235         0.235           b_n = compression flange with         6.99         0.255         0.235           b_n = compression flange with         6.99         0.255         0.235           b_n = compression flange with         0.265         0.255         0.235           2Dob W         107.274478         100.0         5.0         7.97           A_n = 0.3374(E/Ivc) =         10.35750RT(20005) =         8.13         7.97         7.46           A_n = 0.3374(E/Ivc) =         0.38750RT(20005) =         8.13         7.97         7.46         (ASHTO 6.10.8.2.2.4)           A_n = 0.3374(E/Ivc) =         0.38750RT(20005) =         8.13         7.97         7.46         (ASHTO 6.10.8.2.4)           A_n = 0.3374(E/Ivc) =         0.38750RT(20005) =         9.13         1.13         1.14         (ASHTO 6.10.8.2.4)           A_n = 0.111500         1.820         1.81         2.14         (ASHTO 6.10.8.2.1)         1.10.1           A_n	E 29000	ksi							
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y u Stried <b>Local Low for (Dirtic) Building Paintaines</b> <b>Local Low (Dirtic) Building Paintaines</b> <b>Low Set (Dirtic) Building Paintaines</b> <b>Low Set (Dirtic) Building Paintaines</b> <b>Low Set (Dirtic) Building Paintaines</b> <b>Low Set (Dirtic) Building Paintaines</b> <b>Dirtic Dirtic Building Paintaines</b> <b>Dirtic Dirtic Building Paintaines</b> <b>Dirtic Dirtic Building Paintaines</b> <b>Dirtic Dirtic Building Paintaines</b> <b>Dirt Dirtic Building Paintaines</b> <b>Dirt Dirt Dirt Dirt Dirt Dirt Dirt Dirt </b>									
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$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Local Buckling Resistant	се				Bb = 1.0			
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$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	b <sub>c</sub> = compression flange w	vidth		6.99	55	5 ii	n	(	
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$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$\lambda_{pf} = 0.38*V(E/Fyc) =$	=0.38^SQRT	(29000/50) =	9.15					
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$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$\lambda_{\rm f} \leq \lambda_{\rm pf}$			YES	YES	YES			
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$ \begin{split} F_{\mu_{e}} = 1.0 \ if \ 2^{2} \ Der \ W = 30 \ S \ S \ S \ S \ S \ S \ S \ S \ S \ $	R <sub>h</sub> = 1.0	for rolled sha	apes					(AASHTO 6.10.1.10.1)	
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	R <sub>b</sub> = 1.0	if 2*Dc/tw ≤	λrw					(AASHTO 6.10.1.10.2)	
$F_{wc} = -11^{+150} = 50 \text{ ksl}$ $Sx                                     $									
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$F_{nc} = = 1*1*50=$	50	) ksi						
Sk       56.5       38.4       29 <sup>m<sup>2</sup></sup> zx       64.0       44.2       33.2 <sup>in<sup>3</sup></sup> R <sub>m</sub> = M <sub>p</sub> /M <sub>m</sub> = Z/S <sub>x</sub> =       1.13       1.15       1.14       (AASHTO A6.2.1.4)         M <sub>m</sub> = F <sub>m</sub> S <sub>x</sub> =       2825       1920       1450 k·in       (AASHTO A6.2.1.4)         M <sub>m</sub> = F <sub>m</sub> S <sub>x</sub> =       2825       1920       1450 k·in       (AASHTO A6.2.1.4)         M <sub>m</sub> = F <sub>m</sub> M <sub>m</sub> =       2820       2210       1660 k·in       (AASHTO A6.3.2.1)         Lateral Torsional Buckling Resistance       W16x36       W16x26       W14x22       (AASHTO 6.10.8.2.3)         L <sub>h</sub> = 1.0 r, v(EF <sub>p</sub> )       1.808       1.377       1.27 in       Importance dength = 0 ft       0 ft       0 in       YES       YES         I L <sub>b</sub> < L <sub>p</sub> F <sub>m</sub> = R <sub>b</sub> R <sub>b</sub> F <sub>p</sub> c       50       50       50 ksi       Importance dength = 0 ft       163.47       124.52       114.56 in       13.55 ft       13.55 ft       9.55 ft       9.55 ft       13.55 ft       9.55 ft       13.55 ft       14.56 in       9.55									
$Zx$ 64.0       44.2 $33.2 \text{ in}^3$ $F_{\mu c} = M_{\mu}M_{\mu c} = Z_{\nu}S_{\nu} =$ 1.13       1.15       1.14       (AASHTO A6.2.1-4) $M_{\mu c} = F_{\mu c}S_{\nu} =$ 2225       1920       1450 k·in       (AASHTO A6.2.1-4) $M_{\mu c} = F_{\mu c}S_{\mu} =$ 2225       1920       1450 k·in       (AASHTO A6.3.2-1) $M_{\mu c} = F_{\mu c}M_{\mu c} =$ 3200       2210       1660 k·in       (AASHTO A6.3.2-1) $Lateral Torsional Buckling Resistance       W16x36       W16x26       W14x22       (AASHTO A6.3.2-1)         L_{\mu c} = unbraced length =       0 ft       0 in       2.76       2.54 ft       Timber deck braces compression flange         L_{\mu c} = unbraced length =       0 ft       0 in       YES       YES       YES       Timber deck braces compression flange         I^{\mu}_{L_{0}} < L_{\mu} M_{\mu c} = R_{\mu} M_{\mu c}       3200       2210       1660 k·in       1.55 ft         I^{\mu}_{L_{0}} < L_{\mu} M_{\mu c} = R_{\mu} M_{\mu c}       3200       2210       1660 k·in       1.55 ft         I_{\mu} = r_{\mu} r_{\mu} V(E/F_{\mu})       163.47       124.52       114.56 in       9.55 ft       1.55 ft         I_{\mu} = L_{\mu} - V_{\mu} F_{\mu} V_{\mu} V$	Sx			56.5	38.4	29 <sup>ii</sup>	nĭ		
$ \begin{array}{c c c c c c c } \hline Zx & 64.0 & 44.2 & 33.2 \ \mbox{in}^2 \\ \hline P_{nc} = M_{\mu}/M_{\mu}c = Z/S_{\pi} = & 1.13 & 1.15 & 1.14 & (AASHTO A6.2.1-4) \\ \hline M_{nc} = F_{\muc} S_{\pi} = & 2825 & 1920 & 1450 \ \mbox{k-in} & (AASHTO A6.2.1-4) \\ \hline M_{nc} = F_{\muc} M_{\mu c} = & 3200 & 2210 & 1660 \ \mbox{k-in} & (AASHTO A6.3.2-1) \\ \hline M_{nc} = R_{\muc} M_{\mu c} = & 3200 & 32.10 & 30.51 \ \mbox{in} & 30.51 \ \mb$									
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Zx			64.0	44.2	33.2 ii	n³		
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$									
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$R_{pc} = M_p/M_{yc} = Z_x/S_x =$			1.13	1.15	1.14		(AASHTO A6.2.1-4)	
$\begin{array}{c c c c c c c c c c c c c c c c c c c $									
$ \begin{split} & M_{nc} = R_{pc} M_{pc} = & 320 & 210 & 160 \text{ k-in} & (ASHTO A6.3.2-1) \\ & \frac{Lateral Torsional Buckling Resistance}{L_{p} = 1.0 \text{ r, } v(E/F_{pc})} & W16x36 & W16x26 & W14x22 & (ASHTO 6.10.8.2.3) \\ & 3.53 & 3.51 & 3.53 & 3.51 & 3.51 & 1.27 \text{ in} \\ & 1.69 = unbraced length = 0 \text{ ft} & 0.8 \text{ ft} & 0.8 \text{ ft} & 1.377 & 1.27 \text{ in} \\ & L_{9} = unbraced length = 0 \text{ ft} & 0.8 \text{ ft} & 9.8 \text{ ft} & 5.0 \text{ ft} & 5.0 \text{ ft} \\ & L_{9} = L_{9} - R_{pc} R_{pc} + S_{pc} + S_{pc} & 5.0 & 5.0 & 5.0 \text{ ft} \\ & 16 + 5 + 5 + 5 & 12.5 & 114.56 \text{ in} \\ & 16 + 5 + 5 + 5 & 13.62 & 10.38 & 9.55 \text{ ft} \\ & 16 + 5 + 5 + 5 & 13.62 & 10.38 & 9.55 \text{ ft} \\ & F_{p} = 0.7 \text{ F}_{pc} & 35 \text{ ksi} & 114.56 \text{ in} \\ & F_{p} = 0.7 \text{ F}_{pc} & 35 \text{ ksi} & 114.56 \text{ in} \\ & L_{9} + 1 + 5 + 5 + 5 \text{ ft} & 114.56 \text{ in} \\ & 16 + 5 + 5 + 5 + 5 \text{ ft} & 114.56 \text{ in} \\ & 16 + 5 + 5 + 5 + 5 \text{ ft} & 114.56 \text{ in} \\ & 9.55 \text{ ft} & 9.55 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 \text{ ft} & 114.56 \text{ in} \\ & 9.55 \text{ ft} & 9.55 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 \text{ ft} & 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 \text{ ft} & 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 + 5 + 5 + 5 \text{ ft} \\ & 16 + 5 + 5 + 5 + 5 + 5 + 5 + 5 + 5 + 5 + $	$M_{yc} = F_{yc} S_x =$			2825	1920	1450 k	k-in	(AASHTO D6.2)	
$\begin{array}{ c c c c c } & M_{nc} = R_{pc} M_{pc} = & & & & & & & & & & & & & & & & & & $									
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $	$M_{nc} = R_{pc} M_{yc} =$			3200	2210	1660 k	k-in	(AASHTO A6.3.2-1)	
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $									
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	Lateral Torsional Bucklir	ng Resistanc	e .	W16x36	W16x26	W14x22		(AASHTO 6.10.8.2.3)	
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	$L_p = 1.0 r_t v(E/F_{vc})$			43.53	33.16	30.51 ii	n		
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	P			3.63	2.76	2.54 f	t		
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $									
$L_b = unbraced length =$ 0 ft       0 in       Timber deck braces compression flange $L_b \le L_p$ $r_{ES}$ YES       YES         if $L_b < L_p$ $R_n = R_p B_n F_{yc}$ 50       50 ksi         if $L_b < L_p$ $M_n = R_{pc} M_{yc}$ 3200       2210       1660 k-in $L_r = \pi r_t v(E/F_v)$ 163.47       124.52       114.56 in $F_{yr} = 0.7$ $F_{yc}$ 35 ksi       9.55 ft $L_b \le L_r$ YES       YES       YES         if $L_b < L_r$ YES       YES       YES $L_b \le L_r$ YES       YES       YES         if $L_b < L_r$ YES       YES       YES $L_b \le L_r$ YES       YES       YES         if $L_b < L_r$ The C_b (1-(1-F_{yr}/R_h/F_{yc})'(L_b-L_p)/(L_r-L_p)R_bR_h F_{yc} \le R_bR_h F_{yc}       YES $F_{nc} =$ W16x36       pedestrian S0.00 ksi S0.00	$r_t = b_{fc} / (v(12^*(1+1/3^*D_c^*t_w)))$	/ b <sub>fc</sub> / t <sub>fc</sub> )		1.808	1.377	1.27 ii	n		
$ \begin{array}{c c c c c c } L_{b} = unbraced length = & 0 ft & 0 in \\ VES & VES & VES \\ \hline \\ If L_{b} < L_{p} R_{nc} = R_{p}R_{n}F_{yc} & 50 & 50 ksi \\ \hline \\ If L_{b} < L_{p} M_{nc} = R_{pc}M_{yc} & 3200 & 2210 & 1660 k-in \\ \hline \\ L_{r} = \pi r_{t} V(E/F_{yr}) & 163.47 & 124.52 & 114.56 in \\ 13.62 & 10.38 & 9.55 ft \\ \hline \\ F_{yr} = 0.7 F_{yc} & 35 ksi \\ \hline \\ L_{b} < L_{r} & VES & VES \\ \hline \\ 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_{n}F_{yc} \le R_{b}R_{n}F_{yc} \\ \hline \\ F_{nc} = & W16x6 & pedestrian \\ vehicle & 50.00 ksi \\ \hline \\ C_{b} = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^{c} \le 2.3 \\ \end{array} $									
$L_b \leq L_p$ YESYESYESif $L_b < L_p R_{nc} = R_b R_h F_{yc}$ 505050 ksiif $L_b < L_p M_{nc} = R_{pc} M_{yc}$ 320022101660 k-in $L_r = \pi r_t V(E/F_{yr})$ 163.47124.52114.56 in $L_r = \pi r_t V(E/F_{yr})$ 163.47124.52114.56 in $F_{yr} = 0.7 F_{yc}$ 35 ksi35 st9.55 ft $L_b \leq L_r$ YESYESYES $L_b \leq L_r$ YESYESYESif $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^* (L_b - L_p)/(L_r - L_p) R_b R_h F_{yc} \leq R_b R_h F_{yc}$ F $F_{nc} =$ W16x36pedestrian vehicle50.00 ksi 50.00 ksi $C_b = 1.75 - 1.05 (M1/M2) + 0.3 (M1/M2)^c \leq 2.3$ (AASHTO A6.3.3-6)	L <sub>b</sub> = unbraced length =	0	) ft	0 i	n			Timber deck braces com	pression flange
$ \begin{split} & \text{if } L_b < L_p \ F_{nc} = R_b R_h F_{yc} & 50 & 50 \text{ ksi} \\ & \text{if } L_b < L_p \ M_{nc} = R_{pc} M_{yc} & 3200 & 2210 & 1660 \text{ k-in} \\ & L_r = \pi \ r_r \ V(E/F_{yr}) & 163.47 & 124.52 & 114.56 \text{ in} \\ & 13.62 & 10.38 & 9.55 \text{ ft} \\ & F_{yr} = 0.7 \ F_{yc} & 35 \text{ ksi} & & & & & \\ & L_b \leq L_r & YES & YES & YES & YES \\ & L_b \leq L_r & YES & YES & YES & YES \\ & \text{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} \leq R_bR_hF_{yc} & & & & \\ & F_{nc} = W16x36 & pedestrian & 50.00 \text{ ksi} \\ & \text{vehicle} & 50.00 \text{ ksi} & & & \\ & C_b = 1.75 \cdot 1.05(M1/M2) + 0.3(M1/M2)^2 \leq 2.3 & & & & & & & & & & & & & & & & & & &$	$L_{b} \leq L_{p}$			YES	YES	YES			
$\begin{array}{cccccccccccccccccccccccccccccccccccc$									
if $L_b < L_p M_{nc} = R_{pc}M_{yc}$ 3200       2210       1660 k-in $L_r = \pi r_t v(E/F_{yr})$ 163.47       124.52       114.56 in $13.62$ 10.38       9.55 ft $F_{yr} = 0.7 F_{yc}$ 35 ksi       VES $L_b \leq L_r$ YES       YES         if $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^*(L_b - L_p)/(L_r - L_p)R_bR_hF_{yc} \leq R_bR_hF_{yc}$ YES $F_{nc} =$ W16x36       pedestrian       50.00 ksi $c_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \leq 2.3$ (AASHTO A6.3.3-6)	if $L_b < L_p F_{nc} = R_b R_b F_{vc}$			50	50	50 k	si		
if $L_b < L_p M_{nc} = R_{pc}M_{yc}$ 320022101660 k-in $L_r = \pi r_1 V(E/F_{yr})$ 163.47124.52114.56 in $L_r = \pi r_1 V(E/F_{yr})$ 13.6210.389.55 ft $F_{yr} = 0.7 F_{yc}$ 35 ksi1 $L_b \leq L_r$ YESYES $Ib \leq L_r$ YESYESif $L_b < L_r F_{nc} = C_b (1-(1-F_{yr}/R_h/F_{yc})*(L_b-L_p)/(L_r-L_p)R_bR_hF_{yc} \leq R_bR_hF_{yc}$ 4 $F_{nc} =$ W16x36pedestrian50.00 ksi $c_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \leq 2.3$ (AASHTO A6.3.3-6)	,-								
$L_r = \pi r_t V(E/F_{yr})$ 163.47124.52114.56 in $I_{yr} = 0.7 F_{yc}$ 35 ksi10.389.55 ft $L_b \le L_r$ YESYESYESif $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^* (L_b - L_p)/(L_r - L_p) R_b R_h F_{yc} \le R_b R_h F_{yc}$ F_nc =W16x36pedestrian $F_{nc} =$ W16x36pedestrian50.00 ksi50.00 ksi $C_b = 1.75 - 1.05 (M1/M2) + 0.3 (M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)	if $L_b < L_p M_{nc} = R_{pc}M_{vc}$			3200	2210	1660 k	k-in		
$L_r = \pi r_t V(E/F_{yr})$ 163.47 13.62124.52 10.38114.56 in 9.55 ft $F_{yr} = 0.7 F_{yc}$ 35 ksi9.55 ft $L_b \le L_r$ YESYESYESif $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^*(L_b - L_p)/(L_r - L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$ YESYES $F_{nc} =$ W16x36pedestrian vehicle50.00 ksi 50.00 ksi(AASHTO A6.3.3-6) $C_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)									
13.62       10.38       9.55 ft $F_{yr} = 0.7 F_{yc}$ 35 ksi $L_b \le L_r$ YES       YES         if $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^* (L_b - L_p)/(L_r - L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$ YES $F_{nc} =$ W16x36       pedestrian       50.00 ksi $vehicle$ 50.00 ksi       50.00 ksi $C_b = 1.75 - 1.05 (M1/M2) + 0.3 (M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)	$L_r = \pi r \cdot \sqrt{(E/F_w)}$			163.47	124.52	114.56 ii	n		
$F_{yr} = 0.7 F_{yc}$ 35 ksi $L_b \le L_r$ YES       YES         if $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^* (L_b - L_p)/(L_r - L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$ 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)				13.62	10.38	9.55 ft	t		
$L_{b} \leq L_{r} \qquad YES \qquad YES \qquad YES$ if $L_{b} < L_{r} F_{nc} = C_{b} (1 - (1 - F_{yr}/R_{h}/F_{yc})^{*}(L_{b} - L_{p})/(L_{r} - L_{p})R_{b}R_{h}F_{yc} \leq R_{b}R_{h}F_{yc}$ $F_{nc} = \qquad W16x36 \qquad pedestrian \qquad 50.00 \ ksi \qquad vehicle \qquad 50.00 \ ksi$ $C_{b} = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^{2} \leq 2.3 \qquad (AASHTO A6.3.3-6)$	$F_{vr} = 0.7 F_{vc}$	35	i ksi						
$L_b \le L_r$ YES       YES       YES         if $L_b < L_r F_{nc} = C_b (1 - (1 - F_{yr}/R_h/F_{yc})^* (L_b - L_p)/(L_r - L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$ F_{nc} =       W16x36       pedestrian       50.00 ksi $C_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)	j. jo								
$if L_b < L_r F_{nc} = C_b (1-(1-F_{yr}/R_h/F_{yc})^*(L_b-L_p)/(L_r-L_p)R_bR_hF_{yc} \le R_bR_hF_{yc}$ $F_{nc} = W16x36  pedestrian  50.00 \text{ ksi}$ $C_b = 1.75-1.05(M1/M2)+0.3(M1/M2)^2 \le 2.3  (AASHTO A6.3.3-6)$	$L_{h} \leq L_{r}$			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_bR_hF_{yc} \le R_bR_hF_{yc} $ $ F_{nc} = W16x36  pedestrian  50.00 \text{ ksi} $ $ vehicle  50.00 \text{ ksi} $ $ C_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \le 2.3 $ (AASHTO A6.3.3-6)	5								
$F_{nc} = W16x36 \text{ pedestrian } 50.00 \text{ ksi}$ $C_{b} = 1.75 \cdot 1.05(M1/M2) + 0.3(M1/M2)^{2} \le 2.3 $ (AASHTO A6.3.3-6)	if L, < L, E, = C, (1-(1-F, /	B⊾/E)*(I.⊾-I	)/(LL-)B+B+F	< B. B. F.					
$F_{nc}$ =       W16x36       pedestrian       50.00 ksi         vehicle       50.00 ksi $C_b$ = 1.75-1.05(M1/M2)+0.3(M1/M2) <sup>2</sup> < 2.3	$h L_b < L_r h_{nc} - O_b (h (h r yr))$	n'n' yc) (Lb Lp	//(⊑r ⊑p/itbithi	yc = rtbrthryc					
$C_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)	F -	W16x36	podestrian	50.00 1	(ci				
$C_b = 1.75 \cdot 1.05 (M1/M2) + 0.3 (M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)	nc —	10,50	vehicle	50.00 1	kei				
$C_b = 1.75 - 1.05(M1/M2) + 0.3(M1/M2)^2 \le 2.3$ (AASHTO A6.3.3-6)			venicie	50.001	1.51				
		$3(M1/M2)^2 < 3$	2.3			(		A6 3 3-6)	
	$C_{h} = 1.75 \cdot 1.05(M1/M2) + 0.5$	S() = -				(	ANOINC	778.8.8 87	
	$C_b = 1.75 \cdot 1.05(M1/M2) + 0.5$								

	onsulting Engineers	Location Spok	ane	Date	9/9/2014	4 of 20
Avenue Suite	1600 Seattle WA 98101	Client City	of Spokane	Duit	0/0/2011	Job No.
-5822 fax (200	6) 622-8130	Wooden Bridge Ea	ast Load Rating			114176
						<b>REVISION 1</b>
<u>Braci</u>	ng	Check bracing st	rength of timber deck planks	AISC 6.3	.1a	
P <sub>br</sub> =	0.008*M <sub>r</sub> *C <sub>d</sub> /h <sub>o</sub>	(Rec	uired brace strength)	AISC A-6	-5	
	h <sub>o</sub>	13.37 in	(Distance between	flange centro	ids)	
	C <sub>d</sub>	1	(For bending single	curvature)		
	M <sub>r</sub>	1094 <i>k-in</i>	(Factored moment	with LRFD fac	ctors)	
D -	0 651	3.375 III	(depth of timber plank	(5)		
P <sub>br</sub> =	0.05	J K				
ΦP	=		AASHTO 8.8.2-1			
<b>4</b> · n	Φ <sub>comp</sub>	0.9	AASHTO 8.5.2			
	A <sub>g actual</sub>	24.89 in <sup>2</sup>				
	Deck cond	ition factor	0.5 (to red	uce strength)		
	A <sub>g reduced</sub>	12.45 in <sup>2</sup>				
$F_c = F$	co*C <sub>KF</sub> *C <sub>M</sub> *C <sub>F</sub> *C <sub>i</sub> *	C <sub>λ</sub>	AASHTO 8.4.4			
	$F_{co}$	1.45 <i>ksi</i>	AASHTO Table 8.4.	1.1.4-1 - No. 1	L	
	C <sub>KF</sub>	2.78	AASHTO 8.4.4.2 - format conve	rsion factor =	2.5/Φ	
	C <sub>M</sub>	0.8	AASHTO 8.4.4.3 - wet service fa	actor, $\leq 4$ " thic	ck	
	C <sub>F</sub>	1.05	AASHTO 8.4.4.4-1, Size Effect F	actor, 8" widt	h & F <sub>co</sub>	
	C <sub>i</sub>	0.8	AASHTO 8.4.4.7 - incising factor	r		
	C <sub>λ (Str-I)</sub>	0.8	AASHTO 8.4.4.9 - time effect fa	ctor		
	F <sub>c</sub>	2.17 <i>ksi</i>				
c - 1	1, P)/2c y///1, P	$(2c)^2 P(c) < 1$	AASHTO 8.8.2-2			
$U_n = 1$	ттри 20 - VIII тр Кас	0.52	AASHTO 8.8.2			
	K	1	AASHTO 4.6.2.5 (as	sume pinned	-pinned)	
	L	72 in	(Stringer spacing)	·	. ,	
	$L_e = KL =$	72 in	AASHTO 8.8.2			
	Eo	1500 ksi	AASHTO Table 8.4.	1.1.4-1 - No. 1	L	
	$E = E_o * C_M *$	<sup>•</sup> C <sub>i</sub>	AASHTO 8.4.4.1-6			
	:	= 960 <i>ksi</i>				
	$F_{cF} = K_{cF} * E$	$*d^2/L^2$	AASHTO 8.8.2-4			
	:	= 1.10 <i>ksi</i>				
	$B = F_{cE}/F_{c} \le$	<u>1</u>	AASHTO 8.8.2-3			
	-	= 0.51		awn lumber		
	L	0.8	AASHTU 8.8.2, for 9	sawii iuliiber		
	C.	0.44				
<b>A</b> D	C <sub>0</sub>	- <i>i</i> .				
Ψ۲ <sub>n</sub>	10.6	эк				
β <sub>br</sub> =	1/Ф*(4*М <sub>r</sub> *С <sub>d</sub> )/(	L <sub>b</sub> *h <sub>o</sub> )	(Required brace stiffness)	AISC A-6	-6	
	Φ	0.75				
	L <sub>b</sub>	144 in	(length between steel braces)			
$\beta_{br}$	3.03	3 k/in				
ŀ	- A * F / J		(timber stiffners)			
к <sub>timbe</sub>	r planks = A <sub>red.</sub> TE/L = 165.9	) k/in	(umber sunness)			
	100.					

1 m f	6	Project	Riverfront Park	Bridges Inspection & An	alysis	Ву	M. Frymoyer	Sheet No.
крі	Consulting Engineers	Location	Spokane			Date	9/9/2014	5 of 20
1601 Fifth Aven	ue, Suite 1600 Seattle, WA 98101	Client	City of Spokane	) atina				Job No.
(206) 622-5822	fax (206) 622-8130	Wooden Brid	ige East Load R	ating				114176
								DEVISION 1
	E 29000	ksi						
	Fyw 50	ksi						
Capacity of S	Stringer (continued)							
	for <b>vehicle</b> load, M1 = 0, the	erefor Cb = 1.7	5		1.75			
	for <b>pedestrian</b> load							
	M2 = maximum bending me	<del>oment at eithe</del>	<del>r end of unbraced</del>	<del>l length =</del>	<del>38.88</del>	<del>ft-kip</del>		
	Mmid = bending moment in	middle of unl	<del>oraced length (i.e</del>	<del>. 6 ft. from end)</del>				
	Mx = wx/2 (l-	<del>x)</del>	=540*6/2*(24-6	5)/1000 =	<del>29.16</del>	<del>ft-kip</del>		
	$M1 = 2Mmid - M2 \ge M0$		= <u>2*29.16-38.88</u>	-	<del>19.44</del>	<del>ft-kip</del>		
	<del>Cb</del>	=1.75-1.05*(	19.44/38.88)+0.3	<del>*(19.44/38.88)^2 =</del>	1.3			
				<del>W16x26</del>	W14x22			
	$if L_b > L_r, F_{nc} = F_{cr} \le R_b R_h F_{yc}$	<del>W16x26</del>	pedestrian	#DIV/0!	#DIV/0!	<del>ksi</del>		
	$F_{er} = C_b R_b \pi^* E / (L_b / r_t)^*$		vehicle	#DIV/0!	#DIV/0!	<del>ksi</del>		
	Tension Flange Flexural	<u>Resistance</u>						
	$F_{nt} = R_h F_{yt}$			50 ksi				
	Nominal Shear Resistance	ce of Unstitle	ened web					
	+ y = + Q y							
	$\Phi_v V_n = \Phi_v C V_p$	1414.0.00	1440-00	W/4 4-00		(AASHIC	J 6.10.9.2-1)	
		W 16X36	W 16X26	W14X22				
	ν <sub>p</sub> = 0.58 F <sub>yw</sub> D ι <sub>w</sub>	128.67	108.82	86.9101 kip		(AASHIC	) 6.10.9.2-2)	
		15.04	15.04	10.00				
	D = depth of web	15.04	15.01	13.03 in				
	D#	50.00		50.05				
	$D/t_w =$	50.98	60.04	56.65				
	K = 5.0	(given in 6.1	0.9.2)					
		00.04						
	1.12 V(E K/F <sub>yw</sub> )	60.31						
		VES	VES V	EQ			7 6 10 9 3 2-4)	
	/ ••• = ···= · (= ··· yw/	TLO	123 1	L3		(AASHIC	5 0.10.9.3.2-4)	
	0 10							
	G = 1.0							
	C V <sub>p</sub>	128.67	108.82	86.91 KIP				

Indication       Spokane       Date       9/9/2014       6 of 20         Wooden Bridge East Load Rating	- 66	Project	Riverfront Park	Bridges Insp	ection & Analysis	By	M. Frymoyer		Sheet No.
M #101       Client       City of Spokane       Job No.         Wooden Bridge East Load Rating       114176         REVISION         Strength 1         No         1.25         1.00       Good or Satisfactory, BMS Condition 1 or 2       1.25 $\psi_c \phi_s = 0.85$ $\psi_c = 1.0$ (AASHTO 6.5.4.2) $\phi_c \phi_s = 0.85$ $\psi_c = 1.0$ (AASHTO 6.5.4.2)         apacity, C, based on controlling case of local buckling and lateral torsional buckling, tension       W16x36       W16x26 $\psi_c \phi_s = 0.85$ $\psi_c = 1.0$ (AASHTO 6.5.4.2) $\phi_c = 0.85$ $\psi_c = 1.0$ (AASHTO 6.5.4.2) $\psi_c \phi_s = 0.85$ W16x36       W16x26       W14x22         W16x36       W16x26       W14x22 $eff M_{n_c}$ ded Min_c $z^{720}$ 1879       1411 k-in         hitle $z^{720}$ 1879       1411 k-in         add       colspan="2">Colspan="2">No         add Inventory         colspan="2">Inventory         LL       N	<b>PII</b> Consulting Engin	eers Location	Spokane			Date	9/9/2014		6 of 20
Wooden Bridge East Load Rating       I14176         Strength I         1.00       0.85         1.00       0.85         0.05       Good or Satisfactory, BMS Condition 1 or 2       Yoc       1.25 $\psi_{c}$ $\phi_{s}$ 1.00       1.35       Operating $\phi_{c}$ $\phi_{s}$ $=$ 0.01       (AASHTO 6.5.4.2) $\psi_{c}$ $\phi_{s}$ $=$ 0.0       (AASHTO 6.5.4.2) $\psi_{c}$ $\phi_{s}$ $=$ $0.85$ $\psi_{s}$ $1.0$ $\psi_{c}$ $\phi_{s}$ $0.85$ $\psi_{s}$ $1.0$ (AASHTO 6.5.4.2) $\psi_{s}$ $0.85$ $\psi_{s}$ $1.0$ (AASHTO 6.5.4.2) $\psi_{s}$ $0.85$ $\psi_{s}$ $1.0$ (AASHTO 6.5.4.2) $\psi_{s}$ $0.85$ $0.81$ $W14x22$ $RF = \frac{(C - \gamma_{DW} DW \pm \gamma_{\mu} P)}{\gamma_{LL} LL(1 + IM)}$ widestrian       2720       1879       1411 k-in $RF$ $\frac{C}{C - \gamma_{DW} DW \pm \gamma_{\mu} P}$ widestrian $0.63$ 86.31 k-in $RF$ $\frac{C}{(2 + \gamma_{L} LL(1 + IM)}$ $\frac{W16}{16.5}$ $\frac{W16}{15.5}$ $\frac{W16}{15.5}$ $\frac{W16}{15.5}$ $W16$	1 Fifth Avenue, Suite 1600 Seattle, WA 9	Client	City of Spokane	•					Job No.
Strength I       REVISION         1.00 0.85       God or Satisfactory, BMS Condition 1 or 2 $\gamma_{DC}$ 1.25 VL       1.75 Inventory $\psi_{c}$ $\phi_{s}$ $=$ 0.85 $\psi_{c}$ $=$ 1.0       (AASHTO 6.5.4.2) $\psi_{c}$ $\phi_{s}$ $=$ $0.85$ $\psi_{c}$ $=$ $0.65$ $\psi_{c}$ $0.65$ (ASHTO 6.5.4.2) $\psi_{c}$ $\phi_{s}$ $=$ $0.65$ W14x22 $\psi_{c}$ $\psi_{c}$ $0.65$ $\psi_{c}$	i) 622-5822 fax (206) 622-8130	Wooden Br	dge East Load Ra	ating					114176
Strength I       REVISION         1.00 0.85       Good or Satisfactory, BMS Condition 1 or 2 $\sqrt{v_{00}}$ 1.25 $\gamma_{1L}$ 1.75 Inventory $\psi_c \phi_s = 0.85$ $\psi_c = 1.0$ (AASHTO 6.5.4.2) $\psi_r = 0.85$ $\psi_r = 1.0$ $\psi_r = 0.85$ $\psi_r = 1.0$ $\psi_r = 0.85$ <t< td=""><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></t<>									
1.00 0.85       Cood or Satisfactory, BMS Condition 1 or 2       YuL YuL 1.75 Inventory YuL 1.35 Operating $\phi_c \phi_s = 0.85$ $\phi_c \phi_s = 0.85$ $\phi_c = 1.0$ (AASHTO 6.5.4.2) $\phi_c = 1.0$ (AASHTO 6.5.4.2) $\psi_c = 1.0$ $\phi_c = 1.0$ apacity, C, based on controlling case of local buckling and lateral torsional buckling, tension $\psi_c = 1.0$ $W16x36$ W16x26       W14x22 $e^{f} M_{nc}$ 2720       1879         indestrian       2720       1879 $hicle$ 2720       1879 $c DC$ 101.4       90.63 $c DC$ 100.8       100.8 $c DC$ 101.4       100.8 $c DC$ 101.4       100.8 $c DC$ 101.4       100.8 $whick = 6$ </td <td></td> <td></td> <td></td> <td></td> <td></td> <td>Stre</td> <td>nath I</td> <td></td> <td>REVISION</td>						Stre	nath I		REVISION
1.00       0.05       Good or Satisfactory, BMS Condition 1 or 2       VLL       1.75 Inventory $y_{LL}$ 1.35 Operating $\psi_r =$ 1.0       (AASHTO 6.5.4.2) $\psi_c \phi_s =$ 0.85 $\psi_r =$ 1.0       (AASHTO 6.5.4.2)         urg       apacity, C, based on controlling case of local buckling and lateral torsional buckling, tension $W16x36$ $W14x22$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_p P)}{\gamma_{LL} LL(1 + IM)}$ add       2720       1879       1411 k-in         wad       00       81.14       72.50       69.05 k-in         oc       81.14       72.50       69.05 k-in       10.4         odestrian       2720       1879       1411 k-in       10.4         wad       00       0.05       86.31 k-in       10.4         oc       81.14       72.50       69.05 k-in       10.4         c DC       101.4       90.63       86.31 k-in       10.4         whicle       Inventory       1.4       10.8       10.8         LL       1008       1008       1008 k-in       10.4         LL       1008       1008       1008 k-in       10.4 $= ped =$ (2720-101.43)/816.48 <td< td=""><td></td><td></td><td></td><td></td><td></td><td>Voc</td><td>1.25</td><td></td><td></td></td<>						Voc	1.25		
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\phi_{-} = 1$	00 Good or Satis	factory BMS Conditio	in 1 or 2		Vu.	1 75	Inventory	
	d - 0	85	actory, Entereornatio			YLL VI.	1 35	Operating	
$\phi_{c} \phi_{s} = 0.85 \qquad \qquad$	$\varphi_s$ - 0	.00				YLL	1.00	Operating	
		4 4	0.05			φ <sub>f</sub> =	1.0		(AASHTO 0.5.4.2)
ure apacity, C, based on controlling case of local buckling and lateral torsional buckling, tension         W16x36       W16x26       W14x22 $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{p} P)}{\gamma_{LL} LL(1 + IM)}$ = of Mnc adestrian       2720       1879       1411 k-in         add       2720       1879       1411 k-in         add       2720       1879       1411 k-in         add       00       81.14       72.50       69.05 k-in         c DC       101.4       90.63       86.31 k-in         add       Inventory       466.6       466.6 k-in         L L       816.5       816.5 k-in         whicle       Inventory       1008         L L       1008       1008 k-in         = ped = (2720-101.43)/816.48       RF ped = 3.21       2.19         = vehicle = (2720-101.43)/1008       RF vehicle = 2.60       1.77       1.31	o. ·	$\varphi_c \varphi_s$	= 0.85			$\phi_v =$	1.0		
$RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{\gamma_{LL}LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{(I - 1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{(I - 1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{(I - 1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{(I - 1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{(I - 1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{P}P)}{(I - 1 + IM)}$ $RF = \frac{(C - \gamma_{DC}DC - \gamma_{D}DW \pm \gamma_{P}P)}{(I - 1 + I$	Stringer - Flexure								
W16x36       W16x26       W14x22 $RF = \frac{\left(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{p}P\right)}{\gamma_{LL}LL(1 + IM)}$ adestrian       2720       1879       1411 k-in         wide       2720       1879       1411 k-in         wide       2720       1879       1411 k-in         opc       81.14       72.50       69.05 k-in         c DC       101.4       90.63       86.31 k-in         wide       Inventory       1411         till       816.5       816.5         wide       Inventory         till       816.5         wide       Inventory         wide       1008         till       1008         wide       Inventory         wide       1008         wide       1008         till       1008         till       1008         wide       1008 k-in         till       1008         till	Capad	city, C, based on co	ntrolling case of lo	ocal buckling	and lateral torsional b	uckling, tension			
$RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $RF = \frac{(C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_{P} P)}{\gamma_{LL} LL(1 + IM)}$ $\frac{(M - 1)}{1 + 1 + 1}$ $(M - 1$						lc.		- אות	(م ب
$ \begin{array}{c} = \phi f M_{n_{c}} & \gamma_{LL} LL(1 + IM) \\ \text{destrian} & 2720 & 1879 & 1411 \text{ k-in} \\ \text{shicle} & 2720 & 1879 & 1411 \text{ k-in} \\ \text{shicle} & 2720 & 1879 & 1411 \text{ k-in} \\ \text{shicle} & 2720 & 1879 & 1411 \text{ k-in} \\ \text{shicle} & 2720 & 1879 & 1411 \text{ k-in} \\ \text{shicle} & 0.000 & 0.03 & 86.31 \text{ k-in} \\ \text{shicle} & 0.000 & 0.03 & 86.31 \text{ k-in} \\ \text{shicle} & 0.000 & 0.08 & 1008 & 1008 \text{ k-in} \\ \text{shicle} & 0.000 & 0.08 & 1008 & 1008 \text{ k-in} \\ \text{shicle} & 0.000 & 0.08 & 1008 \text{ k-in} \\ \text{shicle} & 0.000 & 0.008 & 1008 \text{ k-in} \\ \text{shicle} & 0.000 & 0.008 \text{ k-in} \\ \text{shicle} & 0.0000 & 0.0000 \text{ k-in} \\ \text{shicle} & 0.00000 \text{ k-in} \\ \text{shicle} & 0.00000 \text{ k-in} \\ \text{shicle} & 0.0000000000 \text{ k-in} \\ \text{shicle} & 0.0000000000000000000000000000000000$			W16x36 W	16x26	W14x22	$RF = \frac{(C - C)}{C}$	$\gamma_{DC}DC - \gamma$	$D_{DW}DW \pm j$	$(V_p P)$
bdestrian       2720       1879       1411 k-in       The second se	C = φ	f M <sub>nc</sub>				Iu	$\gamma_{IL}LL(1$	+IM)	
whicle       2720       1879       1411 k-in         pad       pc       81.14       72.50       69.05 k-in         pc       101.4       90.63       86.31 k-in         ad       Inventory       Inventory         tLL       466.6       466.6       466.6 k-in         LL       816.5       816.5 k-in         whicle       Inventory       Inventory         tLL       576.0       576.0 k-in         LL       1008       1008 k-in         = ped =       (2720-101.43)/816.48       Inventory         = vehicle = (2720-101.43)/1008       Invenice       2.60	pedes	trian	2720	1879	1411 k-in				
aad $p_{C}$ $81.14$ $72.50$ $69.05$ k-in $p_{C}$ $101.4$ $90.63$ $86.31$ k-in $p_{C}$ $101.4$ $90.63$ $86.31$ k-in $p_{L}$ $466.6$ $466.6$ $466.6$ k-in $LL$ $816.5$ $816.5$ $816.5$ k-in $p_{L}$ $576.0$ $576.0$ $576.0$ k-in $p_{L}$ $1008$ $1008$ k-in $q_{L}$ $1008$ $1008$ k-in $q_{L}$ $1008$ $1008$ k-in $q_{L}$ $1008$ $1008$ k-in $q_{L}$ $q_{L}$ $1000$ k-in $q_{L}$ $q_{$	vehicl	e	2720	1879	1411 k-in				
aad $DC$ 81.14       72.50       69.05 k-in $DC$ 101.4       90.63       86.31 k-in         ad       Inventory $LL$ 466.6       466.6 $LL$ 816.5       816.5 $LL$ 816.5       816.5 $LL$ 1008       1008 k-in $LL$ $RF ped = 3.21$ 2.19 $I.62$ RF vehicle = 2.60       1.77 $RF vehicle = 2.60$ 1.77       1.31									
DC $81.14$ $72.50$ $69.05$ k-in         pc $101.4$ $90.63$ $86.31$ k-in         ad       Inventory         LL $466.6$ $466.6$ LL $81.55$ $816.5$ hicle       Inventory         LL $576.0$ $576.0$ LL $1008$ $1008$ hicle       Inventory         LL $1008$ 1008 $1008$ k-in         Ped = $(2720-101.43)/816.48$ = vehicle = $(2720-101.43)/1008$ RF ped = $3.21$ $2.19$ RF vehicle = $2.60$ $1.77$ $1.31$	dead								
DC       101.4       90.63       86.31 k-in         ed       Inventory         LL       466.6       466.6       466.6 k-in         LL       816.5       816.5       816.5 k-in         whicle       Inventory       1008       1008 k-in         LL       1008       1008 k-in       1008 k-in         F ped =       (2720-101.43)/816.48       RF ped =       3.21       2.19       1.62         F vehicle =       (2720-101.43)/1008       RF vehicle =       2.60       1.77       1.31	M <sub>DC</sub>		81.14	72.50	69.05 k-in				
ed       Inventory         LL       466.6       466.6       466.6 k-in         LL       816.5       816.5       816.5 k-in         whicle       Inventory       Inventory         LL       576.0       576.0       576.0 k-in         LL       1008       1008       1008 k-in         Fped =       (2720-101.43)/816.48       RF ped =       3.21       2.19       1.62         F vehicle =       (2720-101.43)/1008       RF vehicle =       2.60       1.77       1.31	γ <sub>DC</sub> D	0	101.4	90.63	86.31 k-in				
ed       Inventory         LL       466.6       466.6       466.6       kin         LL       816.5       816.5       816.5       kin         whicle       Inventory       Inventory       Inventory         LL       576.0       576.0       576.0       kin         LL       1008       1008       1008 k-in       Inventory         F ped =       (2720-101.43)/816.48       Improve the state of the state o									
LL       466.6       466.6       466.6 k-in         LL       816.5       816.5       816.5 k-in         whicle       Inventory         LL       576.0       576.0 k-in         LL       1008       1008 k-in         F ped =       (2720-101.43)/816.48       Inventory         = vehicle = (2720-101.43)/1008       Inventory       Inventory         Inventore       Inventore       Inventory         Inventory       Inventory       Inventory	ped				Inventory				
LL 816.5 816.5 816.5 k-in shicle Inventory LL 576.0 576.0 576.0 k-in LL 1008 1008 1008 k-in F ped = $(2720-101.43)/816.48$ F vehicle = $(2720-101.43)/1008$ RF vehicle = $2.60$ 1.77 1.31	M <sub>LL</sub>		466.6	466.6	466.6 k-in				
Phicle     Inventory       LL     576.0     576.0     576.0     k-in       LL     1008     1008     1008 k-in       F ped =     (2720-101.43)/816.48     RF ped =     3.21     2.19     1.62       F vehicle =     (2720-101.43)/1008     RF vehicle =     2.60     1.77     1.31	γ <sub>LL</sub> LL		816.5	816.5	816.5 k-in				
Phicle     Inventory       LL     576.0     576.0     576.0     k-in       LL     1008     1008     1008 k-in     W16x36     W16x26     W14x22       Inventory     Inventory     Inventory     Inventory     Inventory       F ped =     (2720-101.43)/816.48     RF ped =     3.21     2.19     1.62       F vehicle =     (2720-101.43)/1008     RF vehicle =     2.60     1.77     1.31									
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LL       1008       1008       1008 k-in         M16x36       W16x26       W14x22         Inventory       Inventory       Inventory         F ped =       (2720-101.43)/816.48       RF ped =       3.21       2.19       1.62         F vehicle =       (2720-101.43)/1008       RF vehicle =       2.60       1.77       1.31	M <sub>LL</sub>		576.0	576.0	576.0 k-in				
W16x36       W16x26       W14x22         Inventory       Inventory       Inventory       Inventory         F ped =       (2720-101.43)/816.48       RF ped =       3.21       2.19       1.62         F vehicle =       (2720-101.43)/1008       RF vehicle =       2.60       1.77       1.31	γ <sub>LL</sub> LL		1008	1008	1008 k-in				
Inventory         Inventory         Inventory           F ped =         (2720-101.43)/816.48         RF ped =         3.21         2.19         1.62           F vehicle =         (2720-101.43)/1008         RF vehicle =         2.60         1.77         1.31						W16x36	W16x26	W14x22	
F ped =       (2720-101.43)/816.48       RF ped =       3.21       2.19       1.62         F vehicle =       (2720-101.43)/1008       RF vehicle =       2.60       1.77       1.31						Inventory	Inventory	Inventory	-
F vehicle = (2720-101.43)/1008 RF vehicle = 2.60 1.77 1.31	RF pe	d = (2720-101.4	3)/816.48		RF ped =	3.21	2.19	1.62	
F vehicle = (2720-101.43)/1008 RF vehicle = 2.60 1.77 1.31		( · ·	,						
	RF ve	hicle = (2720-1014	13)/1008		BE vehic	le = 2.60	1 77	1.31	
		(2/20 1011				2.00			
	RF pe	rd = (2720-101.4 hicle = (2720-101.4	43)/816.48 43)/1008		RF ped =	W16x36 Inventory 3.21 le = 2.60		W16x26 Inventory 2.19 1.77	W16x26         W14x22           Inventory         Inventory           2.19         1.62           1.77         1.31
			W16x36 W	16x26	W14x22				
W16x36 W16x26 W14x22	C = φ <sub>0</sub>	√ <sub>n</sub>	128.67	108.82	86.91 kip				

V <sub>DC</sub>	1.13	1.01	0.96 kip			
γ <sub>DC</sub> DC	1.41	1.26	1.20 kip			
Pedestrian						
V <sub>LL</sub>	6.48	6.48	6.48 kip			
γ <sub>LL</sub> LL	11.34	11.34	11.34			
Vehicle						
V <sub>LL</sub>	8.83	8.83	8.83 kip			
γ <sub>LL</sub> LL	15.46	15.46	15.46			
				W16x36	W16x26	W14x22
				Inventory	Inventory	Inventory
RF ped = (128.67-1.4	41)/11.34		RF ped =	11.22	9.49	7.56
RF vehicle = (128.67-1.4	41)/15.46		RF vehicle =	8.23	6.96	5.54

Control products in the product of	knff	1000 MIL 10	Project	Riverfront Park Bridges	Inspection & Analysis	By	M. Frymoyer	Sheet No.
The network particular particula	KPII Consu	ilting Engineers	Location	Spokane		Date	9/9/2014	7 of 20
$ \begin{array}{c} \begin{array}{c} \begin{array}{c} \begin{array}{c} \begin{array}{c} \begin{array}{c} \begin{array}{c} \begin{array}{c}$	1601 Fifth Avenue, Suite 1600	Seattle, WA 98101	Client Woodon Brid	City of Spokane				JOD NO.
laorban $ \begin{array}{ccccccccccccccccccccccccccccccccccc$	(206) 622-5822 fax (206) 622	-8130	Wooden Brid	ige East Load hatting				114170
Let us the set of the	Floorbeam							
size W1626 ead Laads $Size = Size =$			L =	17.47 ft				
ead Loads       Singles Singl			Size	W16x26				
ead Loods $ \begin{array}{ccccccccccccccccccccccccccccccccccc$								
$ \begin{array}{c} \mbox{label{eq:line}} & \mbox{line} $	Dood Loodo		Stringero - F	)ook	P _	1 10 kin	Chaor reaction from and a	otringor
Proce         2.25 kp         W1050, $v_{02} = 1.13 kp$ W1020S         w - 28 brt           Monent         M = V1/9 forobeam           M = P a         -2256 -           a =         6 ft           Munce         -2256 -           Munce         -225 kp           Vel (28 w) V2         -227 08 kp           W16x28 - 292 r         -225 kp           Value         -227 08 kp           W16x28 - 292 r         -225 kp           Value         -27 08 kp           Shear from stringers         -227 08 kp           W16x28 - 292 r         -228 kp           Value         -12.967 6 -           W18x28 - V2 / 2         -27 12.962 -           2 thinger storecontection         PL	Deau Luaus		Stringers + L	Jeck	DC –	1.13 KIP	Shear reaction from end of	sunger
Forbeam       Wiede $w = 0$ fb         Moment $W = w^2 / (2 \text{ forotherm})$ $w = 0$ fb $M = P =$ $0$ fb $0.99 \text{ fb}/(3.26 + 20)$ fb $0.91 \text{ fb}/(3.26 + 20)$ fb $M = P =$ $0$ fb $0.99 \text{ fb}/(3.26 + 20)$ fb $0.99 \text{ fb}/(3.26 + 20)$ fb $M = P =$ $0$ fb $0.99 \text{ fb}/(3.26 + 20)$ fb $0.99 \text{ fb}/(3.26 + 20)$ fb $M = P =$ $0.1$ $0.99 \text{ fb}/(3.26 + 20)$ fb $0.23 \text{ fb}/(3.26 + 20)$ fb         Shear $-122.297 \text{ fb}/(3.26 + 20)$ fb $-227.09 \text{ fb}/(3.26 + 20)$ fb $0.23 \text{ fb}/(3.26 + 20)$ fb         We Loads       Note: posterian bad and vhole load do not ad concurrent?       Note: posterian bad and vhole load do not ad concurrent?       Note: posterian momenta and are concerving them are loaded fb       Note: Store fb $0.23 \text{ fb}/(3.26 + 20)$ fb         We Loads       Note: posterian momenta field concurrent?       Note: posterian momenta field concurrent?       Note: Store fb $0.23 \text{ fb}/(3.26 + 20)$ fb         Memmer $M_{1} \text{ fro: 0}$ $0.10 \text{ fb}/(3.26 + 20)$ fb $0.20 \text{ fb}/(3.26 + 20)$ fb       Note: Store fb         Memmer $M_{2} \text{ for 2}$ $0.776 \text{ fb}/(3.26 + 20)$ fb $0.00 \text{ fb}/(3.26 + 20)$ fb $0.00 \text{ fb}/(3.26 + 20)$ fb         Memer $M_{2} = 0.77.6 \text{ fb}/(3.26 $				2 stringers/connection	$P_{DC} =$	2.25 kip	$v_{DC} = 1.13 \text{ kp}$	
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			Floorbeam					
$\begin{array}{cccccccccccccccccccccccccccccccccccc$			W16x26		W =	26 lb/ft		
Moment       M = wf. / 8 loonbaam       991.700989 lb.b       991.81000 =       0.99 lb.kp         M = P a       = 2576 =       13.52 lb.kp         a =       0 ll       -0.99113.52+2°0       14.52 lb.kp         Shear       V = (2 P + v) (2 Wink 28 = 2817.488762 =       227.09 lb       = 227.091000 =       0.23 kp         Shear       V = (2 P + v) (2 Wink 28 = 2817.488762 =       227.09 lb       = 227.091000 =       0.23 kp         We Loads       Note : protection and other accurs where all shear ac								
W16x26 - (28)'17.47'2/8 -       991'20096 ft-b       91.41000 -       0.91 H4p         M = P a       =       0.1       =       13.52 H4p         M = C       =       0.91'3.52+2'0 =       14.52 ft.Mp         Shear       V = (2 P + v) V =       22.00 b       -227.09 H000 -       0.23 kp         Shear       V = (2 P + v) V =       22.00 b       -227.09 H000 -       0.23 kp         Shear       V = (2 P + v) V =       22.00 b       -227.09 H000 -       0.23 kp         Shear       V = (2 P + v) V =       24.8 kp       0.24 kp         Voc =       2.44 kp       0.10 H000 -       0.23 kp         Vac =       2.44 kp       0.10 H000 -       0.23 kp         2 atringer/connection       PL =       0.46 kp       W16.36, V_L = 6.48 kp         2 atringer/connection       PL =       0.46 kp       W16.36, V_L = 6.48 kp         Morrert       M = P a       -       12.96'S =       7.76 ft.Mp         Advect       M_L + es a       -       -       12.36 kp         Shear       V = 2 P / 2       -       -       12.36 kp         ML + es a       10'G a       10 kp       -       12.36 kp         Mu + P a       -       -       12.96'		Moment	$M = w l^2 / 8 f l$	oorbeam				
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			W16x26	=(26)*17.47^2/8 =	991.760986 ft-lb	991.8/1000 =	0.99	ft-kip
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$								
$a = 6 \text{ ft}$ $M_{01} = - 6 \text{ ft}$ $M_{02} = $			M = P a			=2.25*6 =	13.52	ft-kip
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$			a =	6 ft				
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$ \begin{array}{cccccccccccccccccccccccccccccccccccc$								
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$								
$M_{CC} = - 0.99.13.52.2^{\circ} (14.52 \text{ ft:} kp$ $Shear : V = (2 + w)/2 \\ W 16/26 = 26'1.468'5/2 = 2.27.09 \text{ is } -227.09 \text{ is } -227.09 \text{ is } 0.23 \text{ kp}$ $Shear itom attingers = 2P/2 = 2.25 \text{ kp}$ $V_{CC} = 2.48 \text{ kp}$ $W 16v36, V_{LL} = 6.48 \text{ kp}$ $W 16v36, V_{LL} = 0.48 \text{ kp}$ $W 16v36, V_{LL} = 0.48 \text{ kp}$ $W 16v36, V_{LL} = 0.48 \text{ kp}$ $W 16v36, V_{LL} = 0.00 \text{ ft kp}$ $W 1000  to the the the the the the the the the the$								
$\begin{aligned} & \text{Piere} & \mathbb{P} = [2 \ Pi \ W \ M \ Size \ 2 \ 2 \ 2 \ 2 \ 2 \ 2 \ 2 \ 2 \ 2 \ $			$M_{DC} =$			=0.99+13.52+2	2*0= 14.52	ft-kip
Shear $V = (2 + wi)/2$ $V = 28 + 77.488752 =$ $227.09$ ib $-227.09 + 20 + 0.000 =$ $0.23$ kip         Shear from stringers = $2P/2 =$ $2.25$ kip $V_{00} =$ $2.48$ kip         Vec Loads       Note: pedestrian load and vehicle load do not act concurrently       Maximum moment and shear occurs when all stringers are loaded $P_{1L} =$ Shear reaction from end of stringer W16x86, $V_{L1} = 6.48$ kip $2 stringers/connection       Shear reaction from end of stringerW16x86, V_{L1} = 6.48 kip2 stringers/connection       Shear reaction from end of stringerW16x86, V_{L1} = 6.48 kip2 stringers/connection       Shear reaction from end of stringerW16x86, V_{L1} = 6.48 kip2 stringers/connection       Shear reaction from end of stringerW16x86, V_{L1} = 6.48 kip2 stringers/connection       Shear reaction from end of stringerW16x86, V_{L1} = 6.48 kip2 stringers/connection       P_{L1} = 6.48 kipP_{L2} =       Shear reaction from end of stringerW16x86, V_{L1} = 6.48 kip2 stringers/connection       P_{L1} = 12.96^{\circ} 6 = 77.76 ft-kip12.96 kip         Moment       M = P a       = 12.96^{\circ} 6 = 77.76 ft-kipM_{L1} = 77.76 ft-kip       77.76 ft-kip         Mut       = 0.77.76 ft-kip       max moment at end of braced length (i.e. at stringer connection)       M_{L1} = 77.76 ft-kip       M_{L2} = 60.00 ft-kip         Mut       M = P a       $							-	-
$W16x26 = 28^{\circ}17.46875/2 = 27.09 \text{ b} = -227.091000 = 0.23 \text{ kjp}$ Shear from stringers = 2P/2 = = 2:2.55/2 = 2.25 \text{ kjp} $V_{\infty} = \frac{2.2.25/2}{2.2812} = 2.25 \text{ kjp}$ $V_{\infty} = \frac{2.4.28 \text{ kjp}}{2.2.25 \text{ kjp}}$ $V_{\infty} = \frac{2.2.25/2}{2.2.812} = 2.25 \text{ kjp}$ $V_{\infty} = \frac{2.4.28 \text{ kjp}}{2.2.25 \text{ kjp}} = \frac{2.2.25/2}{2.2.810} = 2.25 \text{ kjp}$ $\frac{V_{\infty} = \frac{2.2.25/2}{2.2.810} = 2.25 \text{ kjp}}{2.2.25 \text{ kjp}}$ $\frac{V_{\infty} = \frac{2.2.25/2}{2.2.810} = 2.25 \text{ kjp}}{2.2.25 \text{ kjp}} = \frac{2.2.25/2}{2.2.810} = 2.25 \text{ kjp}}$ $\frac{V_{\infty} = \frac{2.2.25/2}{2.2.810} = 2.25 \text{ kjp}}{2.2.25 \text{ kjp}} = \frac{2.2.25/2}{2.2.810} = 2.25 \text{ kjp}}{2.2.9676} = 77.76 \text{ ft-kip}}{2.2.9676} = 77.76 \text{ ft-kip}}{2.2.96 \text{ kjp}}{2.2.9676} = 77.76 \text{ ft-kip}}{2.2.96 \text{ kjp}}{2.2.9676} = 77.76 \text{ ft-kip}}{2.2.96 \text{ kjp}}{2.2.96 \text{ kjp}}{2.2.9676} = 77.76 \text{ ft-kip}}{2.2.96 \text{ kjp}}{2.2.96 \text{ kjp}$		Shear	V = (2 P +w I	)/ 2				
$\begin{aligned} & \text{But run stringers } = 2^{\mu} / 2 & 2^{\mu} \\ & \psi_{0} = & 2^{\mu} / 2^{\mu} \\ & \text{int Lots'} \end{aligned}$ Note: potential and vehicle load on out at concurrently $\begin{aligned} & \text{Perform } & Maximum moment and shear occurs where all simpler are loaded in the the load on the the load on the load $			W16x26	= 26*17.46875/2 =	227.09 lb	=227.09/1000	= 0.23	kip
Shear from stringers = $2P/2 =$ = 22.25/2 =       2.25 kp         Voc =       2.48 kp         Voc =       2.48 kp         Vec Loads       Note: pedestrian load and vehicle load do not act concurrently         Pedestrian       Maximum moment and shear occurs when all stringers are loaded Stringer Live Load       Pi, =       6.48 kp       Shear reaction from end of stringer W16x36, Vi, = 6.46 kp         Moment       M = P a       = 12.96°6 =       77.76 ft-kip         a =       6 ft       77.76 ft-kip       12.96 kp         Shear       V = 2 P / 2       = 2*12.96/2 =       12.96 kp         for calculating Cb:       M2 =       77.76 ft-kip       max moment at ond of braced length (i.e. at stringer connection)         Vehicle       Assume full wheel line load at both stringers       Pi, =       10 kip         Moment       M = P a       = 10° 6 =       60.00 ft-kip         Mu = P a       = 10° 6 =       60.00 ft-kip       60.00 ft-kip         Mu = P a       = 00+0 =       60.00 ft-kip       60.00 ft-kip         Nu = $2 = 2/10/2 =$ 10.00 kp       10.00 kp         Mu = 0.00 ft-kip       max moment at ond obraced length       60.00 ft-kip         Mu = 0.00 ft-kip       max moment at end of braced length       60.00 ft-kip <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td>								
Vic =       248 kp         Ive Loads       Not: pedestrian load and vehicle load do not act concurrently       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load       Series action from end of stringer Stringer Leo Load         Morener       M = P a       =       12.96°.6 =       77.76 ft-kip         Shear       V = 2 P / 2       = 2°12.96°.2 =       12.96 kp         Shear       V = 2 P / 2       = 2°12.96°.2 =       12.96 kp         Shear       V = 2 P / 2       = 2°12.96°.2 =       12.96 kp         Mut.reso =       P1 =       77.76 ft-kip       max moment at ond of braced length (i.e. at stringer connection)         Vehicle       M2 =       77.76 ft-kip       max moment at ond of braced length (i.e. at stringer connection)         Morent       M = P a $= 10°.6 =       60.00 ft-kip         Morent       M = P a       = 00·0 -       60.00 ft-kip         Morent       M = P a       = 00·0 -       60.00 ft-kip         Mut.resce =       $	S	hear from string	gers = 2P / 2 =			= 2*2.25/2 =	2.25	kip
Vec =       2.48 kp         Vec =       2.48 kp         Vec =       2.48 kp         Vec =       2.48 kp         Pedestrian       Maximum moment and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more and shear occurs when all stringers are loaded more are are are are are are are are are a								
Ive Loads       Not: setedestrian load and vehicle load do not act concurrently         Pedestrian       Maximum moment and shear occurs when all stringers are loaded stringer live Load       Shear occurs when all stringers are loaded regulation from end of stringer live Load         Moment       M = P a a = $\mu_{L}$ = $6.48 \text{ kpr}$ Shear occurs when all stringers are loaded regulation from end of stringer live Load         Moment       M = P a a = $0.1 \text{ erg}$ $12.96^{\circ} \text{ fr}$ $77.76 \text{ fr.klp}$ Shear       V = 2 P / 2 $= 2^{\circ} 12.96^{\circ} \text{ fr}$ $77.76 \text{ fr.klp}$ $77.76 \text{ fr.klp}$ Shear       V = 2 P / 2 $= 2^{\circ} 12.96^{\circ} \text{ fr}$ $77.76 \text{ fr.klp}$ $77.76 \text{ fr.klp}$ Mut_rece       Mut_rece $77.76 \text{ fr.klp}$ max moment at opposite end of braced length (i.e. at stringer connection)         Vehicle       M1 = $77.76 \text{ fr.klp}$ max moment at opposite end of braced length (i.e. at stringer connection)         Vehicle       M2 = $77.76 \text{ fr.klp}$ max moment at opposite end of braced length (i.e. at stringer connection)         Moment       M = P a $= 0^{\circ} 0.00 \text{ fr.klp}$ $0.00 \text{ fr.klp}$ $0.00 \text{ fr.klp}$ Mark       M = P a $= 0^{\circ} 0.00 \text{ fr.klp}$ $0.00 \text{ fr.klp}$ $0.00 \text{ fr.klp}$ Mark <td></td> <td></td> <td><math>V_{DC} =</math></td> <td></td> <td></td> <td></td> <td>2.48</td> <td>kip</td>			$V_{DC} =$				2.48	kip
Viet : pedestrian       Nate: pedestrian       Maximum moment and shear occurrently         Pedestrian       Maximum moment and shear occurrently $P_{LL} = 6.48$ kip       Shear reaction from end of stringer W16x36, $V_{LL} = 6.48$ kip         Moment       M = P a a = $P_{LL} = 6.48$ kip       Shear reaction from end of stringer W16x36, $V_{LL} = 6.48$ kip         Moment       M = P a a = $e^{12}$ $e^{12}$ .96% fe       Shear reaction from end of stringer W16x36, $V_{LL} = 6.48$ kip         Moment       M = P a a = $e^{12}$ $e^{12.96\%}$ $e^{-77.76}$ ft-kip         Metric pedestrian       M2 = $77.76$ ft-kip $77.76$ ft-kip         Shear       V = 2 P / 2 $e^{-212.96/2} =$ $12.96$ kip         M1 = $77.76$ 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 $P_{U_L} =$ $10$ kip         Moment       M = P a $e^{10}$ fe = $60.00$ ft-kip         Mut_ut_venue.e = $e^{20}$ $e^{21}$ $10$ kip         Shear $e^{2} P / 2$ $e^{21}$ $10.00$ kip         Shear $e^{2} P / 2$ $e^{21}$ $10.00$ kip </td <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td>								
PedestrianMaximum moment and shear occurs when all stringers are loaded Stringer Live Load 2 stringers connection $P_{L} = 6.48 \text{ kip}$ Shear reaction from end of stringer Stringer Live Load 1 (5x36, VL = 6.48 kip)Moment $M = P a$ $a = 6 \text{ ft}$ $= 12.96'6 = 77.76 \text{ ft-kip}$ Moment $M = P a$ $a = 6 \text{ ft}$ $= 12.96'6 = 77.76 \text{ ft-kip}$ Shear $V = 2 P / 2$ $= 2'12.96/2 = 12.96/2 = 12.96 \text{ kip}$ for calculating Cb: $M2 = 77.76 \text{ ft-kip}$ max moment at ond of braced lengthM1 = $77.76 \text{ ft-kip}$ max moment at opposite end of braced length (i.e. at stringer connection)Vehicle $M1 = 77.76 \text{ ft-kip}$ max moment at opposite end of braced length (i.e. at stringer connection)Moment $M = P a$ $= 10'6 = 60.00 \text{ ft-kip}$ $M_{L, VEHCLE} = $ $= 60+0 = 60.00 \text{ ft-kip}$ $M_{L, VEHCLE} = $ $= 60+0 = 60.00 \text{ ft-kip}$ $M_{L, VEHCLE} = $ $= 2'10'2 = 10.00 \text{ kip}$ $M_{L, VEHCLE} = $ $60.00 \text{ ft-kip}$ $M_{L, VEHCLE} = $ $= 0.00 \text{ ft-kip}$ $M_{L, VEHCLE} = $ $M_{L, VHCLE} = 10 \text{ kip}$	Live Loads	Note: pedes	strian load and	vehicle load do not act co	oncurently			
PedestrianMaximum moment and shear occurs when all stringers are loaded Stringer Live Load 2 stringers/connectionShear reaction from end of stringer W16:36, $V_{LL} = 6.48$ kip 12.96 kipShear reaction from end of stringer W16:36, $V_{LL} = 6.48$ kip 12.96 kipMomentM = P a a == 12.96 Kep77.76 ft-kipMult_PED =								
Stringer Live Load $P_{LL} = 6.48 \text{ kp}$ wito 36, $V_{LL} = 5.46 \text{ kp}$ $2 \text{ stringers:connection}$ $P_{LL} = 12.96 \text{ kp}$ Moment $M = P aa = 6  ftM_{LL PED} = 77.76 \text{ ft-kip}M_{LL PED} = 77.76 \text{ ft-kip}M_{LL PED} = 2^{+}12.96/2 = 12.96 \text{ kp}for calculating Cb: M2 = 77.76 \text{ ft-kip} max moment at end of braced lengthM1 = 77.76  ft-kip$ max moment at opposite end of braced length (i.e. at stringer connection) Vehicle $M_{LL VEHOLE} = 10 \text{ kip}$ $M_{LL VEHOLE} = 60.00 \text{ ft-kip}$ $M_{LL VEHOLE} = -60+0 = 60.00 \text{ ft-kip}$ for calculating Cb: $M2 = 60.00 \text{ ft-kip}$ max moment at end of braced length $M = P a$ $= 10^{+}6 = 60.00 \text{ ft-kip}$ $M_{LL VEHOLE} = -2^{+}10/2 = 10.000 \text{ kp}$	Pedestria	an	Maximum mo	oment and shear occurs v	when all stringers are l	oaded	Shear reaction from end of	stringer
$Vehicle = 2 tringers/connection P_{LL} = 12.96 kp$ $Moment M = P a = 6 ft = 12.96 kp = 77.76 ft-kip$ $M_{LL PED} = 77.76 ft-kip = 2'12.96/2 = 12.96 kip$ for calculating Cb: M2 = 77.76 ft-kip max moment at end of braced length $M1 = 77.76 ft-kip max moment at opposite end of braced length (i.e. at stringer connection)$ $Vehicle Assume full wheel line load at both stringers P_{LL} = 10 kip$ $Moment M = P a = 10^{\circ}6 = 60.00 ft-kip$ $M_{LL VEHICLE} = 60+0 = 60.00 ft-kip$ for calculating Cb: M2 = 60.00 ft-kip max moment at end of braced length $M = P a = 10^{\circ}6 = 60.00 ft-kip$ $M_{LL VEHICLE} = 2 P/2 = 10.00 kip$			Stringer Live	Load	P <sub>LL</sub> =	6.48 kip	$VV 16x36, V_{LL} = 6.48 \text{ KIP}$	
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$				2 stringers/connection	P <sub>LL</sub> =	12.96 kip		
$Moment = P a = 6 ft$ $M_{L, PED} = \frac{77.76 ft kip}{12.96 fs} = 77.76 ft kip$ $M_{L, PED} = \frac{77.76 ft kip}{12.96 kip}$ for calculating Cb: M2 = 77.76 ft kip max moment at end of braced length $M1 = 77.76 ft kip max moment at opposite end of braced length (i.e. at stringer connection)$ $Vehicle M1 = 77.76 ft kip max moment at opposite end of braced length (i.e. at stringer connection)$ $M1 = 77.76 ft kip max moment at opposite end of braced length (i.e. at stringer connection)$ $M1 = 77.76 ft kip max moment at opposite end of braced length (i.e. at stringer connection)$ $M1 = 77.76 ft kip max moment at opposite end of braced length (i.e. at stringer connection)$ $M1 = 10^{\circ} 6 = 60.00 ft kip$ $M_{L, VEHCLE} = -60+0 = 60.00 ft kip$ $V = 2 P/2 = -2^{\circ} 10.2 = 10.00 kip$ for calculating Cb: M2 = 60.00 ft kip max moment at end of braced length (i.e. at stringer connection)								
$a = 6 \text{ ft}$ $M_{LL PED} =                                   $		Moment	M = P a			=12.96*6 =	77.76	ft-kip
$M_{LL PED} = \frac{77.76 \text{ ft-kip}}{22 \text{ ft} 296 \text{ ft} 297 \text{ ft-kip}}$ Shear $V = 2 P / 2 = 2^{2} 2 2^{2} = 2^{2} 12.96 \text{ kp}}$ for calculating Cb: $M2 = 77.76 \text{ ft-kip} \text{ max moment at only braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length}$ $M1 = 77.76 \text{ ft-kip} $			a =	6 ft				
$M_{L, PED} = \frac{77.76 \text{ ft-kip}}{12.96 \text{ kp}}$ Shear $V = 2 P/2$ $= 2^{+}12.96/2 = 12.96 \text{ kp}$ for calculating Cb: $M2 = 77.76 \text{ ft-kip}$ max moment at end of braced length (i.e. at stringer connection) $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ M $I = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ M $I = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ M $I = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length (i.e. at stringer connection)}$ M $I = 10^{+} \text{ ft-kip} \text{ max moment at opposite end of braced length}$ M $I = 60.00 \text{ ft-kip} \text{ max moment at end of braced length}$ M $I = 60.00 \text{ ft-kip} \text{ max moment at end of braced length}$								
$M_{LL PED} = \frac{77.76 \text{ ft-kip}}{2}$ $Shear V = 2 P / 2 = 2^{+} 2 = 12.96 / 2 = 12.96 / 4p$ for calculating Cb: $M2 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length} (i.e. at stringer connection)$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length} (i.e. at stringer connection)$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length} (i.e. at stringer connection)$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length} (i.e. at stringer connection)$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at opposite end of braced length} (i.e. at stringer connection)$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length} (i.e. at stringer connection)$ $M1 = 60.00 \text{ ft-kip} \text{ max moment at end of braced length} (i.e. at stringer connection)$								
$M_{LL PED} = \frac{77.76 \text{ ft-kip}}{77.76 \text{ ft-kip}}$ Shear $V = 2 P / 2 = 2^{2} 2 2^{2} 2$								
$M_{LL PED} = \frac{77.76 \text{ ft-kip}}{77.76 \text{ ft-kip}}$ Shear $V = 2 P/2$ $= 2*12.96/2 = 12.96 \text{ kip}$ for calculating Cb: M2 = 77.76 ft-kip max moment at end of braced length $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 77.76 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 60.00 \text{ ft-kip} \text{ max moment at end of braced length}$ $M2 = 60.00 \text{ ft-kip} \text{ max moment at end of braced length}$ $M1 = 60.00 \text{ ft-kip} \text{ max moment at end of braced length}$								
Shear $V = 2 P/2$ $= 2*12.96/2 =$ $12.96 \text{ kip}$ for calculating Cb:M2 =77.76 ft-kipmax moment at end of braced lengthM1 =77.76 ft-kipmax moment at opposite end of braced length (i.e. at stringer connection)VehicleAssume full wheel line load at both stringers $P_{LL} =$ 10 kipMomentM = P a $=10^{\circ}6 =$ $60.00 \text{ ft-kip}$ Shear $V = 2 P/2$ $=2*102 =$ $10.00 \text{ kip}$ for calculating Cb:M2 = $60.00 \text{ ft-kip}$ max moment at end of braced lengthM1 = $60.00 \text{ ft-kip}$ max moment at end of braced length			$M_{LL PED} =$				77.76	ft-kip
Shear $V = 2 P / 2$ $= 2^{*} 12.96 / 2 =$ $12.96 \text{ kip}$ 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)         Vehicle       Assume full wheel line load at both stringers $P_{LL} =$ $10 \text{ kip}$ Moment       M = P a $= 10^* 6 =$ $60.00 \text{ ft-kip}$ MLL VEHICLE = $= 60 + 0 =$ $60.00 \text{ ft-kip}$ Shear $V = 2 P / 2$ $= 2^* 10/2 =$ $10.00 \text{ kip}$ for calculating Cb:       M2 = $60.00 \text{ ft-kip}$ max moment at end of braced length         M1 = $60.00 \text{ ft-kip}$ max moment at end of braced length $(i.e. at stringer connection)$								
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for calculating Cb:M2 =77.76 ft-kipmax moment at end of braced lengthM1 =77.76 ft-kipmax moment at opposite end of braced length (i.e. at stringer connection)VehicleAssume full wheel line load at both stringers $P_{LL} =$ 10 kipMomentM = P a= 10*6 =MLL VEHICLE == 60+0=ShearV = 2 P/2= 2*10/2 =V = 2 P/2= 2*10/2 =for calculating Cb:M2 =M1 =60.00 ft-kipM1 =60.00 ft-kipmax moment at end of braced lengthM1 =60.00 ft-kipmax moment at opposite end of braced lengthM1 =60.00 ft-kipmax moment at opposite end of braced lengthM1 =60.00 ft-kipmax moment at opposite end of braced lengthM1 =60.00 ft-kip								
M1 =77.76 ft-kipmax moment at opposite end of braced length (i.e. at stringer connection)VehicleAssume full wheel line load at both stringers $P_{LL} =$ 10 kipMomentM = P a= 10°6 =60.00 ft-kipM_{LL VEHICLE} == 60+0= $60.00 \text{ ft-kip}$ Shear $V = 2 P/2$ $= 2*10/2 =$ $10.00 \text{ kip}$ for calculating Cb:M2 = $60.00 \text{ ft-kip}$ max moment at end of braced length max moment at opposite end of braced length (i.e. at stringer connection)	for calcu	lating Cb:	M2 =	77.76 ft-kip	max moment at end	d of braced length		
M1 =77.76 ft-kipmax moment at opposite end of braced length (i.e. at stringer connection)VehicleAssume full wheel line load at both stringers $P_{LL} =$ 10 kipMomentM = P a $=10^{\circ}6 =$ $60.00$ ft-kipML_L VEHICLE = $=60+0=$ $60.00$ ft-kipShear $V = 2 P/2$ $=2^{\circ}10/2 =$ $10.00$ kipfor calculating Cb:M2 = $60.00$ ft-kipmax moment at end of braced lengthM1 = $60.00$ ft-kipmax moment at opposite end of braced length (i.e. at stringer connection)								
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Vehicle       Assume full wheel line load at both stringers $P_{LL} = 10$ kip         Moment       M = P a $=10^{\circ}6 = 60.00$ ft-kip         M <sub>LL</sub> vehicle $=60+0=$ $60.00$ ft-kip         Shear $V = 2 P/2$ $= 2^{\circ}10/2 =$ $10.00$ 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)			M1 –	77 76 ft-kin	max moment at on	nosite and of braced le	nath (i e, at stringer conne	ction)
VehicleAssume full wheel line load at both stringer $P_{LL} = 10$ kipMomentM = P a $=10^{\circ}6 = 60.00$ ft-kipM_{LL VEHICLE} = $=60+0= 60.00$ ft-kipShear $V = 2 P/2$ $=2^{\circ}10/2 = 10.00$ kipfor calculating Cb:M2 = 60.00 ft-kipmax moment at end of braced length M1 = 60.00 ft-kip			1011 -	77.70 ft tup	max moment at opp		ngtir (i.e. at stringer conner	
VenicieAssume full wheel line load at both stringers $P_{LL} = 10 \text{ kip}$ MomentM = P a $= 10^{\circ}6 = 60.00 \text{ ft-kip}$ M_{LL VEHICLE} = $= 60+0=$ $60.00 \text{ ft-kip}$ Shear $V = 2 P / 2$ $= 2^{\circ}10/2 =$ $10.00 \text{ kip}$ for calculating Cb:M2 = $60.00 \text{ ft-kip}$ max moment at end of braced length max moment at opposite end of braced length (i.e. at stringer connection)								
$M_{LL} = 10 \text{ kip}$ $Moment  M = P \text{ a} = 10^{\circ}6 = 60.00 \text{ ft-kip}$ $M_{LL \text{ VEHICLE}} = = 60+0 = 60.00 \text{ ft-kip}$ $Shear  V = 2 \text{ P}/2 \qquad = 2^{\circ}10/2 = 10.00 \text{ kip}$ for calculating Cb: M2 = 60.00 ft-kip max moment at end of braced length $M1 = 60.00 \text{ ft-kip} \qquad \text{max moment at opposite end of braced length (i.e. at stringer connection)}$	venicie	2	Assume full v	wheel line load at both stri	ingers			
MomentM = P a $=10^{\circ}6 =$ $60.00 \text{ ft-kip}$ $M_{LL VEHICLE} =$ $=60+0=$ $60.00 \text{ ft-kip}$ Shear $V = 2 P/2$ $= 2^{\circ}10/2 =$ $10.00 \text{ kip}$ for calculating Cb:M2 = $60.00 \text{ ft-kip}$ max moment at end of braced lengthM1 = $60.00 \text{ ft-kip}$ max moment at opposite end of braced length (i.e. at stringer connection)					P <sub>LL</sub> =	10 kip		
WomentM = P a= 10 °6 =60.00 ft-kip $M_{LL VEHICLE} =$ = 60+0=60.00 ft-kipShear $V = 2 P / 2$ = 2*10/2 =10.00 kipfor calculating Cb:M2 =60.00 ft-kipmax moment at end of braced lengthM1 =60.00 ft-kipmax moment at opposite end of braced length (i.e. at stringer connection)			м. р.			10*0	00.00	6 L.
$M_{LL VEHICLE} = = 60+0 = 60.00 \text{ ft-kip}$ $Shear$ $V = 2 P / 2 = 2^* 10/2 = 10.00 \text{ kip}$ for calculating Cb: $M2 = 60.00 \text{ ft-kip}$ max moment at end of braced length $M1 = 60.00 \text{ ft-kip}$ max moment at opposite end of braced length (i.e. at stringer connection)		Woment	M = P a			=10"6 =	60.00	п-кір
$M_{LL \ VEHICLE} = = 60+0= 60.00 \ \text{ft-kip}$ Shear $V = 2 \ \text{P} / 2 = 2^*10/2 = 10.00 \ \text{kip}$ for calculating Cb: M2 = 60.00 \ \text{ft-kip} max moment at end of braced length} $M1 = 60.00 \ \text{ft-kip} max moment at opposite end of braced length (i.e. at stringer connection)}$								
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$\frac{1}{10.00 \text{ ft-kip}} = \frac{1}{10.00 \text{ ft-kip}}$ Shear $V = 2 \text{ P} / 2 = \frac{1}{10.00 \text{ kip}}$ for calculating Cb: M2 = 60.00 ft-kip max moment at end of braced length $M1 = 60.00 \text{ ft-kip} max moment at opposite end of braced length (i.e. at stringer connection)$			M			. 60.0	00.00	ft kin
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)			IL VEHICLE =			=60+0=	60.00	п-кір
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)								
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)		Shear						
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)			V = 2 P / 2			= 2*10/2 =	10.00	kip
Tor 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)	<b>.</b> .	<del>.</del> .						
M1 = 60.00 ft-kip max moment at opposite end of braced length (i.e. at stringer connection)	for calcu	lating Cb:	M2 =	60.00 ft-kip	max moment at end	d of braced length		
M1 = 60.00 ft-kip max moment at opposite end of braced length (i.e. at stringer connection)								
			M1 =	60.00 ft-kip	max moment at op	posite end of braced le	ngth (i.e. at stringer conne	ction)

kpf	f Consulting Engineers	Project Location	Riverfront Park	Bridges Inspection & Analysis	6	By	M. Frymoyer	Sheet No.
Крт		Client	Spokane City of Spokane			Date	9/9/2014	8 of 20
1601 Fifth Aven (206) 622-5822	ue, Suite 1600 Seattle, WA 98101 fax (206) 622-8130	Wooden Brid	ge East Load Ra	ating				114176
1200/ 022 0022	144 1200) 022 0100		0	Ū				
	E 29000	ksi						
	Fyc / Fyt 50	ksi						
	Sx 38.4	in						
	Zx 44.2	in°						
Capacity of Si	tringer							
	Local Buckling Resistance	e						
	$\lambda_{\rm r} = \rm hfc / (2 tfc)$	<u>, , , , , , , , , , , , , , , , , , , </u>		W16v26			0822-3)	
	$b_{c_0} = compression flance w$	ridth		5.5 in		0.0101110-0.1	0.0.2.2 0)	
	$t_{f_0} = compression flance this$	ickness		0.345 in				
	d = depth of member			15.7 in				
	D <sub>c</sub> = depth of web in compl	ression		7.6775 in				
	t <sub>w</sub> = web thickness			0.25 in				
		// - ! / -						
	$\lambda_f =$	=5.5/(2*0.345	o) =	7.97				
	Λ <sub>pf</sub> = 0.38*V(E/FyC) =	=0.38"SQRT	(29000/50) =	9.15		(AASHIO 6.)	0.8.2.2-4)	
	λι < λ_ι			VES				
	rd = rpr			120				
	if $\lambda_t \leq \lambda_{nt}$ then Fnc = Rb Rh Evo	C				(AASHTO 6 1	0.8.2.2-1)	
		-				(, , , , , , , , , , , , , , , , , , ,	0101212 1)	
	R <sub>h</sub> = 1.0	for rolled sha	pes			(AASHTO 6.1	0.1.10.1)	
	R <sub>b</sub> = 1.0	constructabili	ty does not need	to be checked		(AASHTO 6.1	0.1.10.2)	
	<b>-</b>							
	$F_{nc} = =1*1*50=$	50	ksi					
	0*D - /h	04.4						
	2 DC/IW	61.4						
	$\lambda_{m} = 5.7 \sqrt{(E/Evc)}$	137.3						
	· · · · · · · · · · · · · · · · · · ·	107.10						
	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	5.2)	
		0010						
	$M_{nc} = R_{pc} M_{yc} =$	2210	k-in			(AASHTO A6	.3.2-1)	
	Lateral Torsional Bucklin	ng Resistance	4	W16x26		(AASHTO 6 1	0823)	
	$L_p = 1.0 r_t v(E/F_{vc})$	ig neolotanot	<u>-</u>	33.07 in		0.0101110-0.1	0.0.2.0)	
	p ( ( )0)			2.76 ft				
	$r_t = b_{fc} / (\sqrt{12^*(1+1/3^*D_c^*t_w)})$	/ b <sub>fc</sub> / t <sub>fc</sub> )		1.373 in				
	$L_b = unbraced length =$	6	ft	72 in				
	L <sub>b</sub> ≤ L <sub>p</sub>			NO				
	$L_r = \pi r_t v(E/F_{vr})$			124.16 in				
	,			10.35 ft				
	$F_{yr} = 0.7 F_{yc}$	35	ksi					
				VEC				
	- <u>p</u> - <u>r</u>			TE3				
	if L <sub>b</sub> < L <sub>r</sub> F <sub>nc</sub> = C <sub>b</sub> (1-(1-F <sub>ur</sub> /I	R <sub>h</sub> /F <sub>vc</sub> )*(L <sub>h</sub> -L <sub>n</sub> )	/(L <sub>r</sub> -L <sub>p</sub> )R <sub>h</sub> R <sub>h</sub> F <sub>vr</sub> ≤	R <sub>b</sub> R <sub>b</sub> F <sub>vc</sub>				
	D ( ( · y))	,o, ( b -p/	., р, о н ус-	,~				
	F <sub>nc</sub> =		pedestrian	43.59 ksi				
			vehicle	43.59 ksi				
			-	-				
	if $L_b < L_r M_{nc} = C_b (1 - (1 - F_{yr})^2)$	S <sub>xc</sub> /R <sub>pc</sub> /M <sub>yc</sub> )*(L	- <sub>b</sub> -L <sub>p</sub> )/(L <sub>r</sub> -L <sub>p</sub> )R <sub>pc</sub> N	l <sub>yc</sub> ≤ R <sub>pc</sub> M <sub>yc</sub>				
	M <sub>nc</sub> =		pedestrian	1839.9 k-in				
			vehicle	1839.9 k-in				

knff		Project	Riverfront Park Bridges Inspection & A	Analysis	By	M. Frymoyer	Sheet No.	
KPTT Cons	sulting Engineers	Location	Spokane		Date	9/9/2014	9 of 20	
1601 Fifth Avenue, Suite 1600	0 Seattle, WA 98101	Client Wooden Brid	City of Spokane		l		Job No.	
(206) 622-5822 fax (206) 62	2-8130		iye Lasi Luau naliiiy				1141/0	
E	2900	) ksi						
Fyw	5	) ksi						
Capacity of Floorbeam	(continued)							
C <sub>b</sub> = 1.75	6-1.05(M1/M2)+0	.3(M1/M2) <sup>2</sup> ≤ 2	2.3	(AASHTO A6	6.3.3-6)			
for pedes	strian load	=1.75-1.05*(	77.76/77.76)+0.3*(77.76/77.76)^2 =	1.00				
for vehic	le load	=1.75-1.05*(	60/60)+0.3*(60/60)^2 =	1.00				
if L.S.L. F								
$F_{rr} = C_{b}B$	$\pi c = 1 cr \leq 1 (b th)$ $\pi 2F/(1 /r)^2$	/c						
. <sub>G</sub> = <b>O</b> <sub>D</sub> .								
Tension	Flange Flexura	Resistance						
	_							
$F_{nt} = R_h F$	yt		50 ksi					
Nominal	Shear Resistar	ce of Unstiffe	ned Web					
Norman	Oncar ricsistar							
$\phi_v V_n = \phi_v C$	CVp				(AASHTO	6.10.9.2-1)		
		W16x26						
V <sub>p</sub> = 0.58	F <sub>yw</sub> D t <sub>w</sub>	108.82	kip		(AASHTO	6.10.9.2-2)		
D = deptr	n of web	15.01	in					
D/t –		60.04						
D/t <sub>w</sub> =		00.04						
k =	5.0	) (given in AAS	SHTO 6.10.9.2)					
1.12 v(E I	k/F <sub>yw</sub> )	60.31						
if D/t <sub>w</sub> ≤ 1	.12 √(E k/F <sub>yw</sub> )	YES			(AASHTO	6.10.9.3.2-4)		
C -	1.	h						
0 =	1.	5						
C V <sub>p</sub>		108.82	kip					

10 - 66		Project	Riverfront Park Bridges Inspection & Analysis	Ву	M. Frymoyer	Sheet No.
	Consulting Engineers	Location	Spokane	Date	9/9/2014	10 of 20
1601 Eifth Avenue, Suite	1600 Seattle WA 08101	Client	City of Spokane			Job No.
(206) 622-5822 fax (20	16) 622-8130	Wooden Brid	ge East Load Rating			114176

Rating Equation (LRFR Method)

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

where:

RF	= Rating Factor			
С	$= \phi_c \phi_s \phi_n R_n$	for strength li	mit state	$\phi_c \phi_s \ge 0.85$
$\tilde{C}$	$= f_R$	for service lin	nit state	
$\phi_{c}$	= Condition fac	tor		
$\phi_{s}$	= System facto	r		
$\phi_n$	= Resistance fa	ictor based on	construction m	aterial
$R_n$	= Nominal Cap	acity of membe	er	
$J_R$	= Allowable Str	ess per LRFD	Specs.	
$\gamma_{DC}$	= Dead load fa	ctor for structur	al components	and attachments
DC	= Dead load du	e to structural	components ar	nd attachments
γ	- Dood lood for	tor for wooring	curface and u	tilition
		to wearing s	urface and utilit	inues
211		e to weating a		165
${\gamma}_p$	= Load factor for	or permanent lo	ad	
P	= Permanent lo	ad other than o	dead loads	
ν	- Dischard from			
	= Live load fact	Or		
	= Live load effe	CI Lellewanaa (Im	(h = = t)	
11/1	= Dynamic load	allowance (im	ipaci)	
Ø	= 1.00	Good or Satis	sfactory BMS (	Condition 1 or 2
$\phi^{\tau c}$	= 1.00	0000 01 000	, 2	
7 s	$\phi_c \phi_s =$	- 0.85	Μ	BE Table 6A.4.2.4-1
Live Loads	:			
	Pedestrian Lo	bad =	90	) pst
	Vehicle Load	=	H5 or H10	

#### Summary Dead and Live Load Factors for Prestressed Concrete Bridge

Load Combination Limit State	$\gamma_{\scriptscriptstyle DC}$	$\gamma_{\scriptscriptstyle DW}$	$\gamma_{\scriptscriptstyle P}$	$\gamma_{\prime L}$
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

0%

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

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knff		Project	Riverfront Park Bridge	s Inspection & Analysis	Ву	M. Frymoyer		Sheet No.	
KPII Consulting Eng	gineers	Location	Spokane		Date	9/9/2014		11 of 20	
1601 Fifth Avenue, Suite 1600 Seattle, W	VA 98101	Client	City of Spokane					Job No.	
(206) 622-5822 fax (206) 622-8130		vvouaen Brid	aye Easi Load Rating					1141/0	
								REVISION	J 1
					Stre	enath I			
,					Voo	1 25			
C –	$\gamma_{DC}DC$	$C - \gamma_{DW} DW$	$V \pm \gamma_p P$ )		PDC	1.20		Inventori	
RF = -	1/	$LL(1 \pm IM)$	)		YLL	1.75		nivenilory	
	1 IL	$LL(1 \pm 1)VI$	/		YLL	1.35		Operating	
					φ <sub>f</sub> =	1.0		(AASHTO 6.5.	.4.2)
		<i>φφ</i> =	0.85		ф., =	1.0			
Eloorbeam - El	ovuro	ΨcΨs	0.00		τv	1.0			
	entre C	aacad an aar	trolling and of local bu	okling and lateral torgional k	huokling				
Cap	acity, C, I	Jaseu on cor	itrolling case of local bu	ching and lateral torsional a	Ducking				
			W16v06						
G -	љf М		W 10X20				$\phi_c \phi_s =$	1.00	
	etrian		1564 k-in				1830.0	r.00	
pede	olo		1564 k in				1920.0	k in	
veni			1004 K-III				1039.9	N-111	
	ч								
dead	u		174.0 k in						
M <sub>DC</sub>	DC		1/4.2 K-III						
YDC I			217.7 K-IN						
ped			<b>666 1 1 1</b>						
M <sub>LL</sub>			933.1 k-in						
γ <sub>LL</sub> L	L invento	ory	1633 k-in						
γ <sub>LL</sub> L	L operati	ng	1260 k-in						
vehi	cle								
M <sub>LL</sub>			720.0 k-in						
γ <sub>LL</sub> L	L invento	ory	1260 k-in						
γ <sub>LL</sub> L	L operati	ng	972.0 k-in						
					$\phi_{c}\phi_{s}$	= 0.85	$\phi_c \phi_s =$	1.00	
					1013				
					W	16x26	W16	x26	
		Inventory			Inventory	Operating	Inventory	Operating	
RF p	ed =	(1563.9-217	.73)/1632.96	RF ped =	0.8	2 1.07	0.99	1.29	
RF v	vehicle =	(1563.9-217	.73)/1260	RF vehicle =	1.0	7 1.38	1.29	1.67	
Floorbeam- Shea	ar i								
			W16x26						
C = ¢	¢ <sub>v</sub> V <sub>n</sub>		108.82 kip						
V <sub>DC</sub>			2.48 kip						
Noo I	DC		3.10 kin						
YDC	- •		0.10 Mp						
Ped	<u>estrian</u>								
V <sub>LL</sub>			12.96 kip						
γ <sub>LL</sub> L	L.		22.68 kip						
Vehi	icle								
V <sub>LL</sub>			10.00 kip						
γ <sub>LL</sub> L	.L		17.50 kip		W16x26				
			•		Inventorv	-			
RF p	ed =	(108.82-3.1)	/22.68	RF ped =	4.66				
		,							
RF v	vehicle -	(108 82-3 1)	/17.5	RF vehicle =	6.04				
		(100.0 <u>2</u> -0.1)			0.04				

knff		Project	Riverfront Park Bridges Ir	nspection & Analysis	By	M. Frymoyer	Sheet No.
KPII Consulting E	ngineers	Location	Spokane		Date	9/9/2014	12 of 20
1601 Fifth Avenue, Suite 1600 Seattle,	, WA 98101	Ulient Wooden Bride	City of Spokane				JOD NO. 114176
(206) 622-5822 fax (206) 622-8130			Jo Lasi Luau naliny				1170
Floorbeam							
		L =	17.47 ft				
		Size	W16x26				
						Shear reaction from end o	f stringer
Dead Loads		Stringers + De	eck	P <sub>DC</sub> =	1.13 kip	W16x36, V <sub>DC</sub> = 1.13 kip	
			2 stringers/connection	P <sub>DC</sub> =	2.25 kip		
		Floorbeam	Ū				
		W16v26		W -	26 lb/ft		
		VV 10×20		W -	20 10/10		
Мо	ment	$M = w l^2 / 8 flo$	orbeam				
		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
She	ear	V = (2 P + w I)	/ 2				
		W16x26	= 26*17.46875/2 =	227.09 lb	=227.09/1000	) = 0.23	kip
01					0*0.05/0	0.05	1.d.a
Snear fr	om string	ers = 2P / 2 =			= 2"2.25/2 =	2.25	кір
		V <sub>DC</sub> =				2 48	kip
		50				2.10	
Live Loads Not	te: pedest	rian load and v	ehicle load do not act cor	ncurently			
Pedestrian		Maximum mo	ment and shear occurs w	hen all stringers are l	oaded	Shear reaction from end o	f stringer
		Stringer Live	Load	P <sub>LL</sub> =	6.48 kip	W16x36, $V_{LL} = 6.48$ Kip	
			2 stringers/connection	P <sub>LL</sub> =	12.96 kip		
Mo	mont	M – R o			_12.06*6 _	77 76	ft kin
IVIO	ment	м = г а а =	6 ft		=12.90 0 =	11.10	п-кр
		ŭ	0.11				
		$M_{LL PED} =$				77.76	ft-kip
She	ear	V = 2 P / 2			= 2*12.96/2 =	12.96	kip
for a louisting	0		77 70 (1.1.)				
for calculating	g Cb:	M2 =	77.76 ft-kip	max moment at end	d of braced length		
		M1 =	//./6 ft-kip	max moment at opp	posite end of braced I	ength (i.e. at stringer conne	ction)
		_					
Vehicle		Assume full w	heel line load at both strir	ngers	10.11		
				P <sub>LL</sub> =	10 kip		
140	ment	M – P a			_10*e _	60.00	ft-kin
IVIO	inent.	w = r a			= 10 0 =	60.00	ιι-κιμ
		$M_{LL VEHICLE} =$			=60+0=	60.00	ft-kip
She	ear						
		V = 2 P / 2			= 2*10/2 =	10.00	kip
	•						
for calculating	g Cb:	M2 =	60.00 ft-kip	max moment at end	d of braced length		
		M1 _	60.00 ft-kip	max moment at on	nosite end of braced l	enoth (i.e. at stringer conno	ction)
			00.00 it NP	max moment at opp	Source on or braced I	Sugar (no. at sumger confile	

kpff	neulting Engineers	Project	Riverfront Park	Bridges Inspection &	Analysis	By	M. Frymoyer	Sheet No.
	isaning Englieels	Client	City of Snokane	9		Dale	31312014	Job No.
1601 Fifth Avenue, Suite 16 (206) <u>622-5822</u> fax (206)	00 Seattle, WA 98101 622-8130	Wooden Brid	ge East Load R	ating		·		114176
E	29000	) ksi						
Fyc / Fy	t 50	JKSI ≰in <sup>3</sup>						
5X 7v	38.4	+ ''' > in <sup>3</sup>						
ZX	44.2	2 111						
Capacity of Stringer								
Local B	uckling Resistan	nce						
$\lambda_{f} = bfc /$	(2 tfc)			W16x26		(AASHTO 6.1	10.8.2.2-3)	
b <sub>fc</sub> = cor	npression flange v	width		5.5 in				
t <sub>fc</sub> = con	pression flange th	hickness		0.345 in				
a = aepi D = der	in of member	oression		15./ IN 7.6775 in				
t <sub>w</sub> = web	thickness	510331011		0.25 in				
•••								
$\lambda_f =$		=5.5/(2*0.345	i) =	7.97				
$\lambda_{\rm pf} = 0.38$	8*√(E/Fyc) =	=0.38*SQRT	29000/50) =	9.15		(AASHTO 6.1	10.8.2.2-4)	
$\lambda_f \leq \lambda_{pf}$				YES				
16.2								
If $\lambda_f \leq \lambda_{pf}$	then $rnc = Rb Rh Fy$	ус				(AASHIU 6.1	10.8.2.2-1)	
B. =	1 (	) for rolled sha	bes			(AASHTO 6	10 1 10 1)	
- 11						(		
R <sub>b</sub> =	1.0	)				(AASHTO 6.1	10.1.10.2)	
F <sub>nc</sub> =	=1*1*50=	50	ksi					
2^DC/tw		61.4						
λ = 5.7	V(F/Fvc)	137.3						
NW SI	(2)	107.10						
if 2*Dc/t	w ≤ λrw	YES						
$R_{pc} = M$	$_{p}/M_{yc} = Z_{x}/S_{x} =$	1.15				(AASHTO A6	5.2.1-4)	
$M_{yc} = F_y$	<sub>c</sub> S <sub>x</sub> =	1920	k-in			(AASHTO DE	5.2)	
M _ P	М —	2210	k in				2 2 1)	
IVI <sub>nc</sub> = n	<sub>pc</sub> IVI <sub>yc</sub> =	2210	K-111					
Lateral	Torsional Buckli	na Resistance		W16x26		(AASHTO 6.1	10.8.2.3)	
$L_{p} = 1.0$	$r_t v(E/F_{yc})$		-	33.07 in		(	,	
				2.76 ft				
$r_t = b_{fc} /$	(v(12*(1+1/3*D <sub>c</sub> *t <sub>v</sub>	<sub>w</sub> / b <sub>fc</sub> / t <sub>fc</sub> )		1.373 in				
L <sub>b</sub> = unb	praced length =	0	ft	0 in		New deck co	nnections ensure tha	t compression
						flange is fully	braced	
$L_b \le L_p$				YES				
if L <sub>b</sub> < L	$_{p} F_{nc} = R_{b}R_{h}F_{yc}$			50 ksi				
if L <sub>b</sub> < L	$_{\rm p} M_{\rm nc} = R_{\rm pc} M_{\rm yc}$			2210 k-in				
$I = \pi r$	V(E/E)			124 16 in				
$\mathbf{L}_{r} = \mathcal{H}_{t}$	v(L/Tyr)			10.35 ft				
F <sub>yr</sub> = 0.7	′ F <sub>yc</sub>	35	ksi					
$L_b \leq L_r$				YES				
if L <sub>b</sub> < L	$F_{nc} = C_b (1 - (1 - F_{yr}))$	/H <sub>h</sub> /F <sub>yc</sub> )*(L <sub>b</sub> -L <sub>p</sub> )	/(L <sub>r</sub> -L <sub>p</sub> )R <sub>b</sub> R <sub>h</sub> F <sub>yc</sub> :	≤ K <sub>b</sub> R <sub>h</sub> F <sub>yc</sub>				
F =			nedestrian	50 00 kei				
nc —			vehicle	50.00 ksi				
				-				

1 m f	6	Project	Riverfront Park Bridges Inspection & A	nalysis	Ву	M. Frymoyer	Sheet No.	
крг	Consulting Engineer	S Location	Spokane		Date	9/9/2014	14 of 20	
1601 Fifth Aven	ue, Suite 1600 Seattle, WA 9810	Client Woodon Brid	City of Spokane				Job No.	
(206) 622-5822	fax (206) 622-8130	Wooden Brid	ge Last Load Hating				114170	
	E 290	00 ksi						
	Fyw	50 ksi						
Capacity of S	tringer (continued)							
	C <sub>b</sub> = 1.75-1.05(M1/M2)+	$-0.3(M1/M2)^2 \le 2$	2.3	(AASHTO A6	.3.3-6)			
	for pedestrian load	<del>=1 75-1 05*(</del>	<del>77 76/77 76)+0 3*(77 76/77 76)^2 =</del>	<del>1 00</del>				
		-1.10 1.00 (		1.00				
	for <b>vehicle</b> load	<del>=1.75-1.05*(</del>	<del>60/60)+0.3*(60/60)^2 =</del>	<del>1.00</del>				
		_						
	if $L_b > L_r$ , $F_{nc} = F_{cr} \le R_b R_h$	F <sub>yc</sub>						
	$F_{cr} = C_b R_b \pi 2 E / (L_b / r_t)$							
	Tension Flange Flexur	al Resistance						
	renelen Hunge Hexu	<u>ur ricoloturioo</u>						
	$F_{nt} = R_h F_{yt}$		50 ksi					
	Nominal Shear Resista	ance of Unstiffe	ned Web					
	ф.V. – ф.CV					10 9 2-1)		
	$\Psi_v \Psi_n = \Psi_v \Psi_p$	W16x26			(////////01/10/0.	10.5.2 1)		
	$V_p = 0.58 F_{yw} D t_w$	108.82	kip		(AASHTO 6.	10.9.2-2)		
	D = depth of web	15.01	in					
	D#	~~~~						
	$D/t_w =$	60.04						
	k =	5.0 (aiven in AA	SHTO 6 10 9 2)					
		(groot arrow						
	1.12 √(E k/F <sub>yw</sub> )	60.31						
	$   D/l_w \leq 1.12 \ V(E \ K/F_{yw})$	YES			(AASHTO 6.	10.9.3.2-4)		
	_							
	C =	.0						
	C V <sub>n</sub>	108.82	kip					
	- p	100.02						

10 - 66		Project	Riverfront Park Bridges Inspection & Analysis	Ву	M. Frymoyer	Sheet No.
κριι	Consulting Engineers	Location	Spokane	Date	9/9/2014	15 of 20
1601 Eifth Avenue, Suite	e 1600 Seattle WA 08101	Client	City of Spokane			Job No.
(206) 622-5822 fax (20	06) 622-8130	Wooden Brid	ge East Load Rating			114176

Rating Equation (LRFR Method)

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

where:

RF	= Rating Factor					
С	$= \phi_c \phi_s \phi_n R_n$	for strength lin	mit state	$\phi_c \phi_s \ge 0.85$		
С	$= f_R$	for service lim	nit state			
$\phi_{c}$	= Condition fact	or				
$\phi_s$	= System factor					
$\phi_n$	= Resistance fa	ctor based on	construction n	naterial		
$R_n$	<ul> <li>Nominal Capa</li> </ul>	acity of membe	r			
$f_R$	= Allowable Stre	ess per LRFD :	Specs.			
$\gamma_{\scriptscriptstyle DC}$	= Dead load fac	tor for structur	al components	s and attachments		
DC	= Dead load du	e to structural	components a	nd attachments		
$\gamma_{DW}$	= Dead load fac	tor for wearing	surface and u	utilities		
DW	W = Dead load due to wearing surface and utilities					
$\gamma_p$	= Load factor fo	r permanent lo	ad			
Р	= Permanent lo	ad other than o	lead loads			
$\gamma_{\scriptscriptstyle LL}$	= Live load fact	or				
LL	= Live load effe	ct				
IM	= Dynamic load	allowance (Im	pact)			
ø	= 1.00	Good or Satis	factory BMS	Condition 1 or 2		
$\phi_c$	= 1.00	0000 01 0000	nactory, Divic			
7 s	$\phi_c \phi_s =$	0.85	Ν	/IBE Table 6A.4.2.4-1		
l ive l oads						
LIVE LOOUS	Pedestrian Lo	ad =	q	0 psf		
	Vohiolo Lood	_	H5 or H10	- P-1		
	VEHICLE LUQU	-				

#### Summary Dead and Live Load Factors for Prestressed Concrete Bridge

Load Combination Limit State	$\gamma_{\scriptscriptstyle DC}$	$\gamma_{\scriptscriptstyle DW}$	$\gamma_{\scriptscriptstyle P}$	$\gamma_{\prime \perp}$
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

0%

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

IM

koff Consulting Fr	naineers	Project	Riverfront Park Bridges	Inspection & Analysis	By	M. Frymoyer	Sheet No.
	9,,,,,,,,,	Client	City of Spokane		Dale	5/5/2014	Job No.
(206) 622-5822 fax (206) 622-8130	WA 98101	Wooden Brid	ge East Load Rating		L		114176
					Ct	anath I	
			,		<u>Stre</u>	1 25	
С -	$-\gamma_{DC}DC$	$\gamma - \gamma_{DW} DW$	$V \pm \gamma_p P$ )		r DG	1.20	Inventory
<i>KF</i> =	γ I	LL(1 + IM)	<u> </u>		YLL	1.70	Operating
	111	-(	,		YLL A	1.35	
					φ <sub>f</sub> =	1.0	(AASHTO 6.5.4.2)
		$\phi_c \phi_s$ =	0.85		φ <sub>v</sub> =	1.0	
Floorbeam - F	lexure						
Cap	bacity, C, b	based on cor	trolling case of local buc	kling and lateral torsional b	ouckling		
			MICHOC				
С =	фf М.,		W 16X26				
ped	lestrian		1879 k-in				
veh	icle		1879 k-in				
dea	d						
M <sub>DC</sub>	<b>;</b>		174.2 k-in				
YDC	DC		217.7 k-in				
ped							
M <sub>LL</sub>			933.1 k-in				
YLL	LL invento	ry	1633 k-in				
Υ <sub>LL</sub>	LL operatii	ng	1260 K-IN				
veh	icle						
Mu			720 0 k-in				
Vil	LL invento	rv	1260 k-in				
YLL	LL operati	ng	972.0 k-in				
	·	0					
					W	16x26	
		Inventory			Inventory	Operating	
KF	ped =	(18/8.5-21/	/3)/1632.96	R⊢ ped =	1.0	2 1.32	
BE	vehicle –	(1878 5-217	73)/1260	RF vehicle =	1.3	2 1 71	
		(1070.0 217	10)/1200		1.0		
Floorbeam- She	ar						
			W16x26				
C =	φ <sub>v</sub> V <sub>n</sub>		108.82 kip				
V			2 48 kin				
V DC	DC		2.40 Mp				
YDC	20		5.10 KIP				
Dea	loctrice						
<u>Pec</u>	iestrian						
V <sub>LL</sub>			12.96 kip				
YLL	LL		22.68 kip				
Vat	nicle						
			10.00 kip				
YLL	LL		17.50 kip		W16x26		
					Inventory		
RF	ped =	(108.82-3.1)	/22.68	RF ped =	4.66		
RF	vehicle =	(108.82-3.1)	/17.5	RF vehicle =	6.04		

knff		Project	Riverfront Pa	ark Bridges Inspecti	on & Analysis	By	M. Frymoyer	Sheet No.
<b>KPII</b> Consult	ing Engineers	Location	Spokane			Date	8/19/2014	17 of 20
1601 Fifth Avenue, Suite 1600	Seattle, WA 98101	Client	City of Spok	ane				Job No.
(206) 622-5822 fax (206) 622-8	130	Wooden Brid	lge West Loa	d Rating				114176
Timbor Plank Dooking		C -	0.55	Specific Gravity		11 2 24)		
3 <sup>3</sup> / <sub>8</sub> "x7 <sup>3</sup> / <sub>8</sub> " net dimensio	ns	G =	62.4	b/ft <sup>3</sup>	(INDS TADIE	e 11.3.2A)		
Southern Pine		r water	34 32	b/ft <sup>3</sup>				
Accume Southern Ding	No 2	σ	0.65	nof				
Assume Southern Fine	e INU. 2	Utimber	9.00	psi				
Chaoli timbor planka a	o oimplo opo	na hatwaan	ataal atrinaa		Timber Die	ak Dimanaia	20	
Check limber planks a	s simple spa	ns between	steel stringe	is .	Timber Plan	ik Dimensio	ns	
L =	6	tt	Stringer sp	acing	d = depth	3.375	in	
L clr =	6-5.5/12				w = width	7.375	in	
L clr =	5.54	ft						
Width of n	ailer	5.5	in					
Dead Loads		deck	= 7.375/121	t * 9.6525 psf w =	5.930	lb/ft		
	Moment	M = w I <sup>2</sup> / 8	=5.93*5.54^2	2/8 =	22.75 ft-lb			
	Shear	V = w I / 2	= 5.93*5.54/	2="	16.43 lb			
Live Loads	Note: pedest	trian load and	vehicle load o	to not act concurent	lly			
Pedestrian			90	/ psf	(LFRD Ped E	Bridge 3.1)		
		W =	=90*7.375/1	2 =	55.31 lb/ft			
	Moment	M = w F / 8	=55.3125*5.	54^2/8 =	212.20 ft-lb			
	Shear	V = w I / 2	= 55.3125*5	.54/2 =	153.2 lb			
Vehicle			H10		(I FRD Pod F	Rridge 3 2)		
venicie			wheel 1	8 kin	(LIND Fed L 8000	lb	Use maximum of wheel loa	ads
			wheel 2	2 kip	2000	lb		
				·				
			for planks <	10" wide, reduce wh	neel 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 m	ioment occurs	when wheel is at m	nidspan			
			distribute wh	eel load over 20" =	1.67	ft		
			W =	3532.93413 lb/ft				
			K =	2950 -2050*5 54/2 252'		ringer	6940 ft-lb	
				-2950 5.54/2-5552	2.33 1.07/2 1.07/4 =		00+011-10	
	Shoar	for calculatio	a max desia	n shear live load sh	all be place at dist. Fr	om support -	min(3d 1/4 1)	
	Snear	IUI Calculatiii	3d =	0.84 ft	all be place at ulst. I h	om support =	(AASHTO 4.6.2.2.2a)	
			1/4L =	1.39 ft			(, , , , , , , , , , , , , , , , , , ,	
			min =	0.84 ft				
			sum momen	ts at opposite suppo	ort			
			"b" = I <sub>cir</sub> - mir	n(3d, 1/4L)	4.70 ft			
			$V_{LD} = Pb / I$	!	5001.42 lb			
			$V_{LU} =$	678	1.58845 =5001*8000/	5900		
			$V_{LL} =$	$0.5[.6V_{LU}+V_{LD})$			(AASHTO 4.6.2.2.2a-1)	
			$V_{LL} =$	4535 lb				

knff		Project R	verfront Park	Bridges Inspection	& Analysis	Ву	M. Frymoyer	Sheet No.
K PII Con	sulting Engineers	Location S	ookane			Date	8/19/2014	18 of 20
1601 Fifth Avenue, Suite 160	0 Seattle, WA 98101	Client C	ty of Spokane	)				Job No.
(206) 622-5822 fax (206) 6	22-8130	Wooden Bridge	West Load R	ating				114176
Capaci	ty of Timber Pla	ank (Deck)						
Mr = φ	Mn				(AASHT	O 8.6.1-1)		
Mn = F	bSC⊾				(AASHT	O 8.6.2-1)		
φ =	0.85	flexure						
	C <sub>L</sub> = 1.0 w	vhen depth < v	vidth	1.0	(AASHT	O 8.6.2)		
	S = sectio	n modulus						
	$S = bd^2 / 6$	) = =	7.375*3.375 14.00 in	5^2/6 1 <sup>3</sup>				
	$F_b = F_{bo} C$	<sub>KF</sub> C <sub>M</sub> C <sub>F</sub> C <sub>fu</sub> (	$C_i C_d C_\lambda$					
	F <sub>bo</sub> =	1.2 k	si		(AASHT	O Table 8.4	4.1.1.4-1)	
	C <sub>KF</sub> =	2.5/φ =	2.94				format conversion	on factor
	C <sub>M</sub> =	0.85					wet service facto	or
	C <sub>F</sub> =	1	a	Iready adjusted	for southern p	oine	size factor for vis	sually graded lumber
	C <sub>fu</sub> =	1.05					flat use factor	, ,
	C <sub>i</sub> =	0.8					incising factor	
	C <sub>d</sub> =	1 15					deck factor	
	$C_{\lambda} =$	0.8	fc	or Strength I			time effect factor	r
	F <sub>bo</sub> =	2.3184 k	si					
Mn =	32.46 2705	i kip-in i ft-lb						

1		Project	Riverfront Park Bridges Inspection	on & Analysis	By	M. Frymoyer	Sheet No.
кртт	Consulting Engineers	Location	Spokane		Date	8/19/2014	19 of 20
1601 Fifth Avenue, Su	ite 1600 Seattle, WA 98101	Client	City of Spokane				Job No.
(206) 622-5822 fax	(206) 622-8130	Wooden Brid	ge West Load Rating				114176
φ =	- 0.75	shear		(AASHTO	8.5.2.2)		
	Shear para	allel to grain					
	$F_v = F_{vo} C_b$	$\langle F C_M C_i C_\lambda \rangle$		(AASHTO	8.4.4.1)		
	F <sub>vo</sub> =	0.175	ksi	AASHTO	Table 8.4.1	.1.4-1	
	C <sub>KF</sub> =	2.5/φ =	3.33	(AASHTO	8.4.4.2)		
	C <sub>M</sub> =	0.97		(AASHTO	8.4.4.3)		
	C <sub>i</sub> =	0.8		(AASHTO	8.4.4.7)		
	$C_{\lambda} =$	0.8		(AASHTO	8.4.4.9)		
				Υ.	,		
	F <sub>v</sub> =	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				

Project Riverfront Park Bridges Inspection & Analysis By M. Frymoyer Shee	eet No.
K OTT         Consulting Engineers         Location         Spokane         Date         8/19/2014         20 o	of 20
1601 Fifth Avenue, Suite 1600 Seattle, WA 98101 Client City of Spokane Job	b No.
(206) 622-5822 fax (206) 622-8130 Wooden Bridge West Load Rating 114	4176
$C = \phi_c \phi_s \phi_n R_n$ for strength limit state $\phi_c \phi_s \ge 0.85$	
$(C, r, DC, r, DW + r, p)$ $Y_{DC}$ 1.25	
$RF = \frac{(C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{p}F)}{V(L - VC)}$ VLL 1.75 Inventory	
$\gamma_{LL}LL(I + IM)$ YLL 1.35 Operating	
$\Phi_s$ = 1.0 Deck is redundant system $\Phi_f$ = 0.85	
$\phi_{\rm v} = 0.75$	

#### Deck - Flexure

			φ <sub>c</sub> =	condition fa	actor
			Good	Fair	Poor
			1	0.95	0.85
			C =	≡φ <sub>c</sub> φ <sub>s</sub> φfM <sub>n</sub> (f	t-lb)
	$\gamma_{DC}$ DC	28.44 ft-lb	2299.24	2184.28	1954.35
	Pedestrian			RF	
Inventory	$\gamma_{LL} \; LL$	371.36 ft-lb	6.11	5.81	5.19
Operating	$\gamma_{LL}$ LL	286.47 ft-lb	7.93	7.53	6.72
	Vehicle			RF	
Inventory	$\gamma_{LL}$ LL	12144.8 ft-lb	0.19	0.18	0.16
Operating	$\gamma_{LL}$ LL	9368.8 ft-lb	0.24	0.23	0.21

for vehicles
RF x wheel load = maximum wheel for RF = $1.0$
Inventory
Operating

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

#### Floorbeam- Shear

			φ <sub>c</sub> =	condition fa	actor
			Good	Fair	Poor
			1	0.95	0.85
			С	$= \phi_c \phi_s \phi_v V_n$	(lb)
	$\gamma_{DC}$ DC	16.43 lb	4506.86	4281.52	3830.83
	Pedestrian			RF	
Inventory	$\gamma_{LL}$ LL	268.13 lb	16.75	15.91	14.23
Operating	$\gamma_{LL}$ LL	206.84 lb	21.71	20.62	18.44
	Vehicle			RF	
Inventory	$\gamma_{LL}$ LL	7936.6 lb	0.57	0.54	0.48
Operating	$\gamma_{LL}$ LL	6122.5 lb	0.73	0.70	0.62
	for vehicles				
	RF x wheel load	I = maximum wheel for RF = 1.0	Max	Wheel Load	d (lb)

 $RF \times wheel load = maximum wheel for <math>RF = 1.0$ Inventory Operating

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

# APPENDIX D

UNDERWATER INSPECTION REPORT - SEE SEPARATE FILE

# APPENDIX E

### PHOTOGRAPH LOG PHOTOGRAPH CONTACT SHEET

<b>kpff</b> Consulting Engineer	s Location	Riverfror	It Park Bridges Inspection	By	MLF 8/14/2014	Sheet No.	
Consulting Engineer	Client	City of S	ookane	Dale	0/14/2014	Job No	
1601 Fifth Avenue, Suite 1600 Seattle, WA 98101 (206) 622-5822 fax (206) 622-8130	Inspection P	hoto Log	Sonarie			114176.12	
Bridge Name:	Wooden B	ridge Eas	t				
Date of Inspection:	8/13/2014						
Photo No.	Location		Notes				Bv
							_,
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 F	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	1812 Deck		Deck, looking north				TW
1986	Elevation		Elevation, looking east				MF
1987	Elevation		Elevation, looking east				MF
1988	Elevation		Elevation, looking east				MF
<b>├</b> ─── <b>├</b> ───							
			l				
<u>├</u> ───┤──							
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		-					

### East Wooden Bridge Photographs



IMG\_1805.JPG

IMG\_1810.JPG

![](_page_54_Picture_3.jpeg)

![](_page_54_Picture_4.jpeg)

![](_page_54_Picture_6.jpeg)

IMG\_1811.JPG

![](_page_54_Picture_8.jpeg)

![](_page_54_Picture_9.jpeg)

IMG\_1812.JPG

![](_page_54_Picture_11.jpeg)

IMG\_1808.JPG

IMG\_1986.JPG

![](_page_54_Picture_13.jpeg)

IMG\_1809.JPG

IMG\_1987.JPG