FINAL REPORT

State of Wyoming
Department of Transportation

WY-20/05F

Assessment and Evaluations of I-80 Truck Loads and Their Load Effects: Phase 2: Service

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16. Abstract:

The research objective is to assess the performance (in terms of AASHTO design expectations for serviceability) of bridges along the Interstate 80 (I-80) corridor for Wyoming's truck traffic. Wyoming's I-80 carries a large volume of cross-continental and large energy industry trucks compared to many states. Moreover, frequent weather closures position trucks side-by-side and end-to-end for miles. These vehicles then travel as a convoy once the road opens. Wyoming's unique truck traffic and traffic patterns potentially create larger demands on bridges than those considered in the development of the AASHTO LRFD *Bridge Design Specifications*. These characteristics may also be true for other states that contain unique traffic features.

Reliability studies and live load factor calibration were performed in this study using a database of in-service Wyoming bridges. This database, consisting of 112 steel bridges and 60 prestressed concrete bridges, was used to determine modified Service II and Service III live load factors to maintain adequate reliability against exceeding serviceability limit states. The results confirm that the current live load factor of γ_L = 1.30 does not meet the serviceability expectations in the AASHTO Service II limit state (structural steel yielding) for Wyoming traffic on I-80. The results also confirm that the current live load factor of γ_L = 0.80 does not meet the serviceability expectations in the AASHTO Service III limit state (prestressed concrete cracking) for Wyoming traffic on I-80.

Based on the I-80 weigh-in-motion (WIM) vehicle load characteristics that create load effects for Service II and Service III limit states, the reliability indices do not meet the target reliability in the AASHTO LRFD Specifications. Raising the design live load factors, γ_L, directly and fairly uniformly increases reliability indices. An increase in γ_L for Service II to 1.45 (from 1.30) and an increase in γ_L for Service III to 1.00 (from 0.80) increases all of the reliability indices to more closely match the reliability indices expected with the AASHTO LRFD Specifications.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
ADTT	average daily truck traffic
Bias	β ratio of actual load effect to the nominal or design load
COV	coefficient of variation
CDF	cumulative density function
D	dead load effect
Gamma	γ load factor
GDF	girder distribution factor
GVW	gross vehicle weight
IM	dynamic load allowance (impact)
L	live load effect
LLF	live load factor
LRFD	Load and Resistance Factor Design
Mean	μ average of data
MPH	miles per hour
NCHRP	National Cooperative Highway Research Program
Nominal Resistance	strength required to meet limit state (optimal with Performance ratio = 1.0)
PDF	probability distribution function
RF	rating factor
S	girder spacing
WIM	weigh-in-motion
WYDOT	Wyoming Department of Transportation

EXECUTIVE SUMMARY

The research objective is to assess the performance (in terms of AASHTO design expectations for serviceability) of bridges along the Interstate 80 (I-80) corridor for Wyoming's truck traffic (WYDOT 2013). Wyoming's I-80 corridor carries a large volume of cross-continental and large energy industry trucks compared to many states. Moreover, frequent weather closures position trucks side-by-side and end-to-end for miles. These vehicles then travel as a convoy once the road opens. Wyoming's unique truck traffic and traffic patterns potentially create larger demands on bridges than those considered in the development of the AASHTO LRFD *Bridge Design Specifications* (AASHTO 2017). These characteristics may also be true for other states that contain unique traffic features.

A previous study was performed to assess the bridge safety (in terms of AASHTO Strength I design expectations) along the I-80 corridor (Barker and Puckett 2016 and 2019). Rational single- and multi-presence load cases were developed to model the traffic pattern characteristics thought to exist on I-80 across Wyoming. The 75-year design life live load model was applied for reliability studies. Reliability indices were computed using Monte Carlo simulation. The results indicated Wyoming's truck traffic and traffic patterns create larger demands than that considered in the AASHTO LRFD Specifications. Shorter, multi-span bridges are especially critical, leading to lower reliability indices. This led to recommendations for increasing the Strength I live load factor.

Similar reliability studies and live load factor calibration were performed in this study using a database of in-service Wyoming bridges. This database, consisting of 112 steel bridges and 60 prestressed concrete bridges, was used to determine modified Service II and Service III live load factors to maintain adequate reliability against exceeding serviceability limit states. The results confirmed that the current live load factor of $\gamma_L = 1.30$ did not meet the serviceability expectations in the AASHTO Service II limit state (structural steel yielding) for Wyoming traffic on I-80. The results also confirmed that the current live load factor of $\gamma_L = 0.80$ did not meet the serviceability expectations in the AASHTO Service III limit state (prestressed concrete cracking) for Wyoming traffic on I-80.

The study also shows that there are many steel bridges along the I-80 corridor that currently do not meet the Service II limit state. These are older bridges that were designed between the late 1950's and mid-1970's according to earlier specifications. However, it is expected that these in-service bridges may experience yielding and permanent set in excess of that allowed in the AASHTO LRFD Specifications. This would also be true if the prestressed concrete bridges used in the study were located on the I-80 corridor.

Based on the I-80 weigh-in-motion (WIM) vehicle load characteristics that create load effects for Service II and Service III limit states, the reliability indices do not meet the target reliability in the AASHTO LRFD Specifications. Raising the design live load factors, γ_L, directly and fairly uniformly increases reliability indices. An increase in γ_L for Service II to 1.45 (from 1.30) and an increase in γ_L for Service III to 1.00 (from 0.80) increases all of the reliability indices to more closely match the reliability indices expected with the AASHTO LRFD Specifications.

CHAPTER 1. INTRODUCTION

Wyoming's Interstate 80 (I-80) corridor carries a large volume of cross-continental and large energy industry trucks compared to many states (WYDOT 2013). Moreover, frequent weather closures position trucks side-by-side and end-to-end for miles. These vehicles then travel as a convoy once the road opens. Wyoming's unique truck traffic and traffic patterns potentially create larger demands on bridges than those considered in the development of the AASHTO LRFD *Bridge Design Specifications* (AASHTO 2017). These characteristics may also be true for other states that contain unique traffic features.

A previous study was performed to assess the bridge safety (in terms of AASHTO Strength I design expectations) along the I-80 corridor (Barker and Puckett 2016 and 2019). Rational single- and multi-presence load cases were developed to model the traffic pattern characteristics thought to exist on I-80 across Wyoming. The 75-year design life live load model was applied for reliability studies. Reliability indices were computed using Monte Carlo simulation. The results indicated Wyoming's truck traffic and traffic patterns created larger demands than that considered in the AASHTO LRFD Specifications. Shorter multi-span bridges were especially critical, leading to lower reliability indices.

To maintain target bridge safety, two important findings and recommendations were provided. The first pertains to not only Wyoming, but to all states.

1. AASHTO should incorporate the AASHTO commentary low-boy tandem load as part of the HL-93 loading specifications. The significantly lower reliability indices for shorter, multi-span bridges would be evident for all states because heavy trucks that straddle the interior support of short multi-span bridges causing relatively large negative moments are common; and

2. WYDOT (and other states with similar truck traffic conditions) should increase the live load factor, γ_L, for interstate bridges. The design live load factor, γ_L, can directly and fairly uniformly increase reliability indices. An increase in γ_L increases the nominal required capacity R_n, which increases the reliability indices fairly uniformly over a large range of bridge designs. The recommendation for WYDOT is to increase the live load factor to $\gamma_L = 2.00$ using a normal distribution for Wyoming truck traffic analysis, or $\gamma_L = 1.90$ using an upper-tail traffic characteristic distribution.

The "optional" low-boy tandem load presented in the AASHTO LRFD Commentary C.3.6.1.3.1 significantly increases the negative live load design moments for shorter spans. Using the low-boy tandem, the reliability indices for the shorter, two-span bridges increased to the range of reliability indices for the other length bridges.

If the commentary low-boy tandem loading is used, the reliability indices are fairly consistent. However, as a whole, they are below the target safety when using the Wyoming weigh-in-motion (WIM)-based truck data. Raising the design live load factor, γ_L, directly increases, with approximate uniformity, the reliability indices. An increase in γ_L to 2.00 (from 1.75) increases almost all of the reliability indices above the target safety, with only a few dipping slightly below.

The previous recommendation was determined using a slightly conservative normal distribution for the Wyoming truck data. The AASHTO LRFD Specifications were developed under NCHRP projects (Nowak 1999, Kulicki et al. 2007) that used truck database, raw data, upper-tail statistical procedures to estimate maximum truck load effects. An alternative to a live load factor increased to 2.00 is to consider the NCHRP method for the statistical properties of the live load model. When the NCHRP procedures were applied to the Wyoming WIM database, the required increase of the live load factor is to 1.90, smaller than the 2.00 noted above.

Incorporation of the above recommendations will raise the level of safety for Wyoming bridges on I-80 to that expected in the AASHTO LRFD Specifications. However, because the truck traffic on I-80 exceeds the demands represented in the AASHTO LRFD Specifications, additional concerns exist for performance and serviceability. The Service II design limit for steel bridges and the Service III design limit for prestressed concrete bridges are also of concern for the I-80 bridges.

The present objective is to determine if the Service II and III design limit states should be modified to account for the unique Wyoming truck traffic characteristics. The principles, dead load properties, and truck traffic characteristics from the Strength I design limit study are applied next to the Service II design limit for steel girders. In a subsequent section, the Service III design limit for prestressed concrete is addressed.

BRASS-GIRDER™

Load effects for Wyoming bridges are obtained using the Wyoming Department of Transportation's (WYDOT) BRASS-GIRDER™ analysis software (WYDOT 2019). BRASS-GIRDER™ is designed to assist the bridge engineer in the design review or rating of highway bridge girders for a variety of bridge types. This software provides for input of the bridge configuration and geometry, materials, noncomposite and composite dead loads, prestress strand geometry, and vehicular live loads. BRASS-GIRDER™ utilizes a direct stiffness solver to analyze the structure to obtain load effects. It also calculates section resistances and performs specification compliance in accordance with the AASHTO LRFD Specifications.

Existing bridge files were obtained from WYDOT and revised as necessary to create girder-system input files that contained the number of girders, girder spacing, deck cantilever lengths, and deck thickness. This study leverages the BRASS-GIRDER™ software for research that directly benefits WYDOT.

CHAPTER 2. SERVICE II DESIGN LIMIT

The AASHTO Service II design limit state controls yielding and permanent set in steel bridges. The design requirement is to limit the stress in the steel to $0.80F_y$ for noncomposite bridges, or to $0.95F_y$ for composite bridges subject to nominal dead load and factored live load, where F_y is the nominal yield stress of the steel.

The nominal load on a steel bridge girder is

$$D_n + D_{nw} + L_n(1 + IM)GDF \tag{1}$$

where:

- D_n = Nominal Noncomposite and Composite Dead Load Effect
- D_{nw} = Nominal Wearing Surface Load Effect
- L_n = Nominal Live Load Effect (AASHTO HL-93)
- IM = LRFD Design Dynamic Load Allowance (IM = 0.33)
- GDF = Lateral Distribution Factor

The load effects for Service II are stresses.

The optimized (meaning at the design limit where the rating factor (*RF*), or design performance ratio, is equal to 1.0) Service II design limit is

$$R_n = D_n + D_{nw} + \gamma_L L_n(1 + IM)GDF \tag{2}$$

where:

- R_n = Optimized (Service II Rating Factor = 1.0) Nominal Resistance
- γ_L = Live Load Factor (γ_L = 1.30 AASHTO Service II))

The Service II design limit is checked using stresses. The stress limit for composite beams ($0.95F_y$) shown is

$$0.95Fy = \frac{D_{nncp}}{S_x} + \frac{D_{ncp}}{S_{3n}} + \frac{D_{nw}}{S_{3n}} + \frac{\gamma_L L_n(1 + IM)GDF}{S_n} \tag{3}$$

where:

- D_{nncp} = Nominal Noncomposite Dead Load Moment
- D_{ncp} = Nominal Composite Dead Load Moment
- D_{nw} = Nominal Wearing Surface Load Moment
- L_n = Nominal Live Load Moment (AASHTO HL-93)
- S_x = Noncomposite Steel Section Modulus
- S_{3n} = Composite Long-Term Section Modulus
- S_n = Composite Short-Term Section Modulus

All of the ratios have units of stress.

In terms of moments, the limit state is

$$R_n = 0.95M_y = 0.95S_nF_y = \left(\frac{S_n}{S_x}\right)D_{nncp} + \left(\frac{S_n}{S_{3n}}\right)(D_{ncp} + D_{nw}) + \gamma_L L_n(1 + IM)GDF \tag{4}$$

where:

M_y = First-Yield Moment on the Composite Section

Assigning the ratios of the section moduli as *SR1* and *SR2*, the resistance becomes

$$R_n = 0.95M_y = 0.95S_nF_y = SR1D_{nncp} + SR2(D_{ncp} + D_{nw}) + \gamma_L L_n(1 + IM)GDF \tag{5}$$

For noncomposite bridges, the above equation simplifies to

$$0.80Fy = \frac{D_n}{S_x} + \frac{D_{nw}}{S_x} + \frac{\gamma_L L_n(1 + IM)GDF}{S_x} \tag{6}$$

and, because *SR1* and *SR2* are both equal to 1.0,

$$R_n = 0.80M_y = 0.80S_xF_y = D_n + D_{ws} + \gamma_L L_n(1 + IM)GDF \tag{7}$$

The design limit, and thus the reliability analyses, depend on the relative magnitude of the noncomposite dead, composite dead, wearing surface, and live loads. Therefore, the following ratios are computed to support the reliability analyses.

Let *W* be the ratio of nominal noncomposite and composite dead load to nominal live load

$$W = \frac{D_{nncp} + D_{ncp}}{L_n(1 + IM)GDF} \tag{8}$$

and *Y* be the ratio of composite dead load to nominal noncomposite and composite dead load

$$Y = \frac{D_{ncp}}{D_{nncp} + D_{ncp}} \tag{9}$$

and *X* be the ratio of wearing surface dead load to nominal noncomposite and composite dead load

$$X = \frac{D_{nw}}{D_{nncp} + D_{ncp}} \tag{10}$$

With substitution, the nominal load is

$$[(1 - Y)W + (YW) + (XW) + 1]L_n(1 + IM)GDF \tag{11}$$

and the optimized (rating factor equal 1.0) nominal resistance is

$$R_n = [SR1(1-Y)W + SR2(YW) + SR2(XW) + \gamma_L]L_n(1+I)GDF \tag{12}$$

where:

$D_{nncp} = (1-Y)WL_n(1+IM)GDF$
$D_{ncp} = \mathrm{YW}L_n(1+IM)GDF$
$D_{nncp} + D_{ncp} = \mathrm{W}L_n(1+IM)GDF$
$D_{nw} = \mathrm{XW}L_n(1+IM)GDF$
$D_{nncp} + D_{ncp} + D_{nw} = (1+\mathrm{X})\mathrm{W}L_n(1+IM)GDF$

SERVICE II RELIABILITY

In the limit state equation, Z represents the stresses exceeding the stress limits, R_n, of $0.95F_y$ for composite or $0.80F_y$ for noncomposite bridges. Because the design is optimized, the stress limit is determined by the design requirements (rating factor equal to 1.0).

$$\begin{aligned} Z &= R - \left(\frac{S_n}{S_x}\right)D_{nc} - \left(\frac{S_n}{S_{3n}}\right)D_c - \left(\frac{S_n}{S_{3n}}\right)D_w - LL(1+I)(GDF) \\ &= R - (SR1)D_{nc} - (SR2)D_c - (SR2)D_w - L \end{aligned} \tag{13}$$

Using the statistical properties for the random variables from the Phase I: Strength I reliability study:

R = Moment Strength – Lognormal Distribution

μ_R	$= \lambda_R R_n$	Mean
λ_R	= 1.12	Bias
R_n	$= [SR1(1-Y)W + SR2(YW) + SR2(XW) + \gamma_L]L_n(1+IM)GDF$	
COV_R	= 0.10	Coefficient of Variation

D_{nc} = Noncomposite Moment – Normal Distribution

μ_{Dnc}	$= \lambda_{Dnc} D_{nncp}$	Mean
λ_{Dnc}	= 1.05	Bias
D_{nncp}	=	Nominal Noncomposite Dead Load
COV_{Dnc}	= 0.10	Coefficient of Variation

D_c = Composite Moment – Normal Distribution

μ_{Dc}	$= \lambda_{Dnc} D_{ncp}$	Mean
λ_{Dc}	= 1.05	Bias
D_{ncp}	=	Nominal Composite Dead Load
COV_{Dc}	= 0.10	Coefficient of Variation (100 percent correlated with D_{nc})

D_w = Wearing Surface Moment – Normal Distribution

μ_{Dw}	$= \lambda_{Dw} D_{nw}$	Mean
λ_{Dw}	= 1.00	Bias
D_{nw}	=	Nominal Wearing Surface Load
COV_{Dw}	= 0.25	Coefficient of Variation

$L = LL(1+I)GDF$ = Live Load Moment on Girder

μ_L	$= \mu_{LL}(1+\mu_I)\mu_{GDF}$	Mean
COV_L	=	Determined by variables in L (below)

LL = Vehicle Moment on Bridge – Normal Distribution

μ_{LL}	$= \lambda_L L_n$	Mean
λ_L	=	Bias Determined by Live Load Model
L_n	=	HL-93 Nominal Live Load
COV_{LL}	= 0.06	Determined by Live Load Model (Phase I: Strength I study)

I = Dynamic Impact on Girder – Normal Distribution

μ_I	= 0.10	Mean
COV_I	= 0.80	Coefficient of Variation

GDF = Girder Distribution Factor – Normal Distribution

μ_{GDF}	= 1.0	Mean
GDF	=	Lateral Distribution Factor
COV_{GDF}	= 0.12	Coefficient of Variation

$SR1 = S_n/S_x$– Assumed Constant (variability assumed included in variability in dead load)
$SR2 = S_n/S_{3n}$– Assumed Constant (variability assumed included in variability in dead load)

The nominal AASHTO HL-93 loading used for L_n includes the controlling GDF factor (already distributed to the girder) for the reliability analyses. Therefore, the mean of the GDF is set to 1.00. However, the coefficient of variation for the GDF is used to determine the COV_L for the live load.

$$COV_L = \frac{\sqrt{(1+\mu_I^2)COV_{LL}^2 + (1+\mu_I^2)COV_{GDF}^2 + \mu_I^2 COV_I^2}}{(1+\mu_I)} \quad (14)$$

= 0.14 (from Phase I: Strength I study)

Consistent with the Phase I: Strength I study, for the reliability analyses, $\mu_I = 0.10$ is used for all cases, even when it is a road closure load case where impact is assumed to be zero. To account for this, the road closure (I = 0) live load bias values were divided by 1.1 (1+ μ_I) so that the live load mean in the reliability analyses did not include impact. In addition, the noncomposite dead load, D_{nc}, and the composite dead load, D_c, are assumed to be 100 percent correlated.

SERVICE II TARGET RELIABILITY

The NCHRP studies (Nowak 1999, Kulicki et al. 2007) developed AASHTO Strength I load factors based on dead and live load statistical models. The AASHTO live load factor for Strength I is $\gamma_L = 1.75$. The Phase I: Strength I study used the same process to recommend modified live load factors based on the live load statistical characteristics of Wyoming truck traffic on I-80. The recommended live load factor is either 2.00 or 1.90, depending on the method used for the live load characteristics. The recommended increase is due to the increase of the live load bias for the Wyoming truck traffic on I-80. The target reliability for the AASHTO Service II limit state can be determined by applying the NCHRP live load characteristics to the Service II limit state and the AASHTO Service II live load factor of γ_L = 1.30.

The Phase I: Strength I study NCHRP Example Bridges with Live Load Bias λ_L = 1.18 and COV_L = 0.18 (NCHRP live load model) demonstrate the target Service II reliability indices currently expected in the AASHTO Specifications. Because the example bridges are presented only as moments, *SR1* is assumed as 1.543 and *SR2* is assumed to be 1.098 as representative ratios. These ratios do not affect the results for using the NCHRP example bridges because the ratios are used consistently for all of the analyses. It is also assumed that the nominal composite dead load represents five percent of the total nominal dead load. Table 1 illustrates the three NCHRP Example bridges that were used in the Phase I: Strength I study.

Table 1. NCHRP Example Bridges (NCHRP 20-7/186 report, Updating the Calibration Report for AASHTO LRFD Code)

	NCHRP Example Bridges		
	Bridge 1	**Bridge 2**	**Bridge 3**
Actual M_y	23667	62188	26585
D_n	9071	27017	8496
D_{nw}	1247	3529	1493
L_n(1+I)GDF	5332	11521	7120
Optimized M_y	22540	59227	25320

For the reliability analysis of Example Bridge 1, the nominal variables become

D_{nncp} = 8617 ft-k (95 percent of total dead)
D_{ncp} = 454 ft-k (5 percent of total dead)
D_{nw} = 1247 ft-k
$L_n(1+IM)GDF$ = 5332 ft-k (includes IM = 0.33)
$SR1$ = 1.543
$SR2$ = 1.098

And for the optimized (Service II Rating Factor = 1.0)

$$R_n = 0.95M_y = \left(\frac{S_n}{S_x}\right)D_{nncp} + \left(\frac{S_n}{S_{3n}}\right)(D_{ncp} + D_{nw}) + 1.30L_n(1+IM)GDF \qquad (15)$$

M_y = 22540 ft-k
R_n = 22096 ft-k

The reliability statistical properties become

$\mu_R = \lambda_R R_n$ = 1.12(22096) = 24747 ft-k $\quad COV_R$ = 0.10
$\mu_{Dnc} = \lambda_{Dnc} D_{nncp}$ = 1.05(8617) = 9048 ft-k $\quad COV_{Dnc}$ = 0.10
$\mu_{Dc} = \lambda_{Dnc} D_{ncp}$ = 1.05(454) = 476 ft-k $\quad COV_{Dc}$ = 0.10 (100 percent correlated with D_{nc})
$\mu_{Dw} = \lambda_{Dw} D_{nw}$ = 1.00(1247) = 1247 ft-k $\quad COV_{Dw}$ = 0.25

To determine the live load, the design dynamic impact factor of $(1+IM) = 1.33$ must be removed from $L_n(1+IM)GDF$ so that the statistical dynamic impact factor, I, can be used in the reliability analyses.

$$\mu_L = \mu_{LL}(1+\mu_I)\mu_{GDF} = \lambda_L[L_n(1+IM)GDF]/(1+IM)](1+\mu_I)$$

$$\mu_L = 1.18[5332/1.33](1+0.10) = 5204 \text{ ft-k} \qquad COV_L = 0.18 \text{ (NCHRP)}$$

MONTE CARLO SIMULATIONS FOR RELIABILITY INDICES

To determine the reliability index for the AASHTO Service II limit state equation, statistical methods are used to predict the probability that the limit state equation is less than zero (probability that the strength is less than the combined load effect). Because algebraic sums and products are with mixed lognormal and normal variables in the limit state equation, Monte Carlo simulation was used to determine the reliability indices.

For one Monte Carlo trial, the limit state equation Z is computed by simulating the R, D_{nc}, D_c, D_w and L random variables according to their distributions. The definition of failure is if Z is less than zero. For the present work, 100,000 trials are used to determine how many failures occur, n_{Fail}. The probability of failure is $p_f = n_{Fail}$ / 100,000. The inverse cumulative density function of $-\Phi^{-1}(-p_f)$ results in the number of standard deviations failure is away from the mean of Z. The inverse cumulative density function $-\Phi^{-1}(-p_f)$ is the reliability index β. The number of trials of 100,000 was deemed a sufficiently large number for accuracy by experimenting with lower and larger values.

The Microsoft Excel™ random number generator, lognormal, and normal functions are used for the Monte Carlo simulations. The previous section defined the mean and coefficient of variation for the variables R, D_{nc}, D_c, D_w and L. However, because R is lognormally distributed, the mean of $\ln(R)$ and the standard deviation of $\ln(R)$ are required.

From statistics

$$\mu_{lnR} = \ln(\mu_R) - 0.5\sigma_{lnR}^2 \tag{16}$$

$$\sigma_{lnR} = \sqrt{\ln\,(1 + COV_R^2)} \tag{17}$$

The Monte Carlo simulation is demonstrated with the first NCHRP example from Table 1. Table 2 shows the Monte Carlo Excel analysis for Example Bridge 1. The number of failures in 100,000 trials is 10883, yielding a probability of failure of 10.88 percent. A probability of failure of 10.88 percent denotes a reliability index of $\beta = 1.23$. This represents the target reliability for the AASHTO Service II design limit for Example Bridge 1.

Table 2. Monte Carlo Excel Results for NCHRP Example Bridge 1

		Live Load	1.3		Live Load Bias		1.180					
		phi	1							Composite Factor		0.95
				Nominal	Bias	**Mean**	**COV**	**Std Dev**				
LogNormal	R	Resistance		22096	1.12	24746.99	0.1	2474.699	10.11148	0.099751		
Normal	D nc	Dead	95.00%	8617	1.05	9048.32	0.1	904.832				
Normal	D c		5.00%	454	1.05	476.23	0.1	47.623				
Normal	Dw	Wearing Surface		1247	1	1247.00	0.25	311.750				
Normal	(L+I)GDF	Live		4410	1.180	5203.71	0.18	936.668				
	(S3/S1)			1.543	1	1.543	0.000001	0.0000				
	(S3/S2)			1.098	1	1.098	0.000001	0.0000				
											Number	Percent
											Fail	Fail
										1.543	10883	10.883%
			24749.38284	9045.437	1244.211	5202.93327	1.543	1.098			**BETA =**	**1.23**
	100000	Trials	R	D_{nc}	D_w	(L+I)GDF	(S3/S1)	(S3/S2)				
		1	26681.66	7401.92	1882.41	4778.08	1.54	1.10				
		2	22446.79	8293.96	1033.11	2958.89	1.54	1.10				
		3	24007.90	7229.57	1583.24	5348.74	1.54	1.10				
		4	24286.91	8605.98	1175.20	4282.00	1.54	1.10				

Table 3 illustrates the three NCHRP Example bridge reliability results in addition to the respective dead and live load ratios *X*, *Y*, and *W*.

Table 3. Target Reliability for NCHRP Example Bridges

	NCHRP Example Bridges		
	Bridge 1	**Bridge 2**	**Bridge 3**
Actual M_y	23667	62188	26585
D_n	9071	27017	8496
D_{nw}	1247	3529	1493
L_n(1+I)GDF	5332	11521	7120
Optimized M_y	22540	59227	25320
$X = D_{nw}/(D_{nncp}+D_{ncp})$	0.137	0.131	0.176
Assumed $Y = D_{ncp}/(D_{nncp}+D_{ncp})$	0.05	0.05	0.05
$W = (D_{nncp}+D_{ncp})/L_n(1+IM)GDF$	1.701	2.345	1.193
Live Load Factor	1.3	1.3	1.3
Beta	1.23	1.08	1.41

The Beta values represent the target reliability indices associated with the AASHTO Service II limit state for a live load factor of γ_L = 1.30. The target reliability index is not uniform over ranges of nominal dead to live loads. As the ratio of dead load to live load increases, *W*, the target reliability index decreases. Figure 1 shows the three NCHRP Example bridge target reliability indices values labeled "NCHRP Example Bridges." As shown, the target reliability index varies with the nominal dead to live load ratio, W. This figure is further developed with additional analyses next.

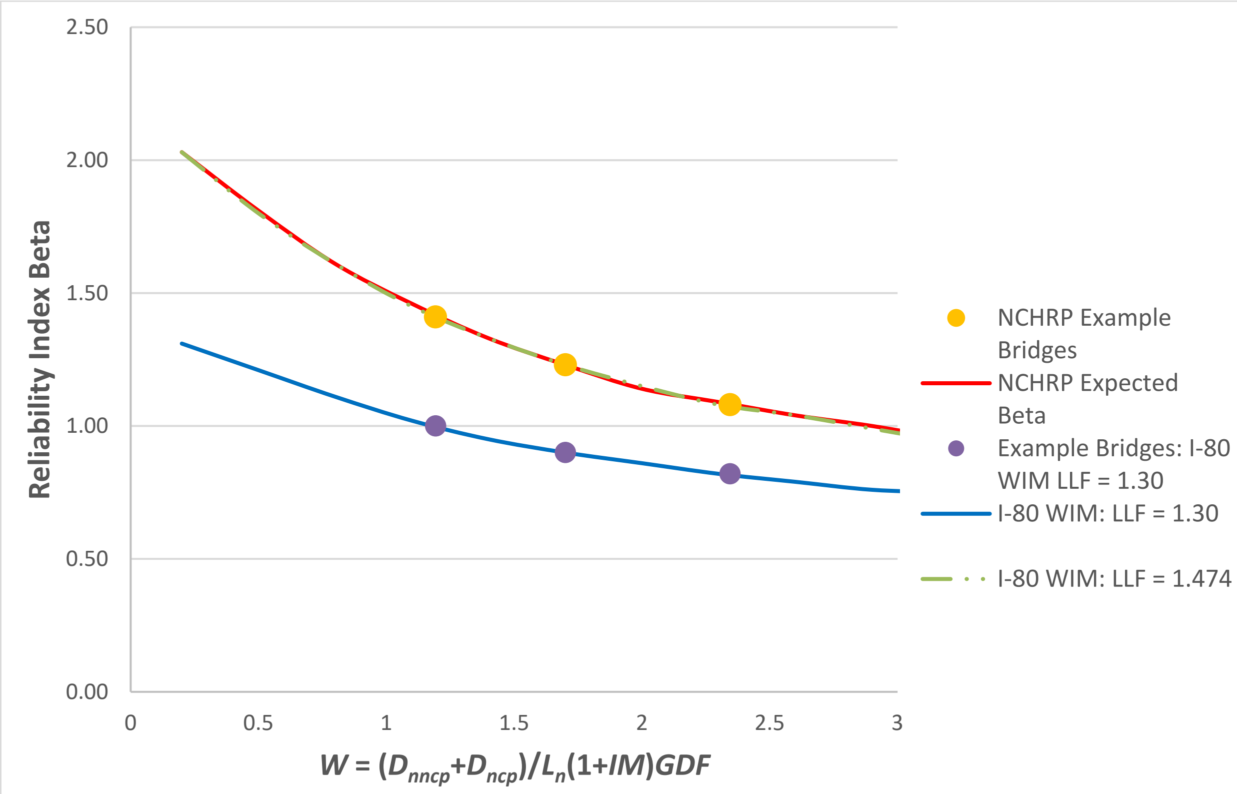

Figure 1. Service II Reliability Indices

To further develop the AASHTO Service II target reliability, Monte Carlo simulations were conducted while varying the nominal dead to live load ratio, W, for reasonable values of W. This is shown in the Figure 1 data series labeled "NCHRP Expected Beta." The target reliability varies according to the dead-to-live load ratio W. For the analysis, $X = 0.15$ (composite to total dead load ratio) and $Y = 0.05$ (wearing surface to total dead load ratio) were used.

CURRENT (USING LIVE LOAD FACTOR = 1.30) WYOMING TRUCK TRAFFIC SERVICE II RELIABILITY

The Phase I: Strength I study developed live load characteristics for Wyoming truck traffic on I-80 used to recommend modified Strength I live load factors. Those live load characteristics are used here to examine the Service II limit state. Table 4 shows the live load bias over various span lengths for positive and negative moment regions. Table 4 assumes that the first recommendation from the Phase I: Strength I study, that the "optional" low-boy tandem load presented in the AASHTO LRFD Commentary C.3.6.1.3.1, is used to determine the negative design live load moments for shorter spans, i.e., this load is not optional.

Table 4 shows the live load bias for both the normal distribution and the NCHRP tail-end methods. A rational live load bias of 1.40 (with a $COV_L = 0.14$) was chosen to represent the live load characteristics for the Service II analyses for this work. It is less than the maximums shown in Table 4. However, as is observed when comparing the database of actual steel bridges, there is considerable variability in the target reliability indices and a live load bias of 1.40 for Wyoming truck traffic is reasonable.

Table 4. Phase I: Strength I Live Load Bias

Bridge	Normal Method	NCHRP Method
Simple 30 ft	1.497	1.432
Simple 50 ft	1.365	1.247
Simple 100 ft	1.334	1.242
Simple 150 ft	1.389	1.239
Simple 200 ft	1.382	1.354
Two-Span 30 ft	1.168	1.159
Two-Span 50 ft	1.447	1.238
Two-Span 100 ft	1.386	1.268
Two-Span 150 ft	1.450	1.292
Two-Span 200 ft	1.356	1.237

The three NCHRP Example bridges are again used to determine the reliability indices for the Wyoming truck traffic characteristics using a live load bias λ_L = 1.40 (vs 1.18 for NCHRP) and COV_L of 0.14 (vs 0.18 for NCHRP). A live load factor, γ_L, of 1.30 shows the reliability for the Service II limit state currently expected along the I-80 corridor using the current specifications. Table 5 shows the three NCHRP Example bridge results.

Table 5. Wyoming Truck Traffic Reliability for NCHRP Example Bridges: γ_L = 1.30

	NCHRP Example Bridges		
	Bridge 1	Bridge 2	Bridge 3
Actual M_y	23667	62188	26585
D_n	9071	27017	8496
D_{nw}	1247	3529	1493
$L_n(1+I)GDF$	5332	11521	7120
Optimized M_y	22540	59227	25320
$X = D_{nw}/(D_{nncp}+D_{ncp})$	0.137	0.131	0.176
Assumed $Y = D_{ncp}/(D_{nncp}+D_{ncp})$	0.05	0.05	0.05
$W = (D_{nncp}+D_{ncp})/L_n(1+IM)GDF$	1.701	2.345	1.193
Live Load Factor	1.3	1.3	1.3
Beta	0.90	0.82	1.00

With the higher live load bias, the reliability indices are significantly lower than the target indices shown in Table 3. This is also demonstrated in the Figure 1 data series labeled "Example Bridges: I-80 WIM LLF = 1.30," where the reliability indices of the three example bridges are considerably lower than the targets. Therefore, the current live load factor of 1.30 does not meet the serviceability expectations in the AASHTO Service II limit state for Wyoming traffic on I-80.

To further demonstrate the deficiency of Service II reliability using a live load factor of 1.30 and the Wyoming traffic characteristics, Monte Carlo simulations were conducted while varying the nominal dead to live load ratio, *W*. This is shown in the Figure 1 data series labeled "I-80 WIM: LLF = 1.30." The reliability varies according to *W*, and with the same trend, but the reliability is considerably lower than the target. *X* and *Y* remained 0.15 and 0.05, respectively.

REQUIRED LIVE LOAD FACTOR TO ATTAIN TARGET RELIABILITY INDICES

To determine the appropriate Service II live load factor, γ_L, that would be required to meet the target reliability for the three NCHRP Example bridges with the Wyoming traffic characteristics, the Monte Carlo simulations were run to find the live load factor that produced the same reliability indices from Table 3. Table 6 shows the results.

Table 6. Wyoming Truck Traffic Target Reliability for NCHRP Example Bridges

	NCHRP Example Bridges		
	Bridge 1	**Bridge 2**	**Bridge 3**
Actual M_y	23667	62188	26585
D_n	9071	27017	8496
D_{nw}	1247	3529	1493
L_n(1+I)GDF	5332	11521	7120
Optimized M_y	22540	59227	25320
$X = D_{nw}/(D_{nncp}+D_{ncp})$	0.137	0.131	0.176
Assumed $Y = D_{ncp}/(D_{nncp}+D_{ncp})$	0.05	0.05	0.05
$W = (D_{nncp}+D_{ncp})/L_n(1+IM)GDF$	1.701	2.345	1.193
Live Load Factor	1.474	1.474	1.474
Beta	1.23	1.08	1.41

If a live load factor of 1.474 is used for the Service II design limit (instead of 1.30), the reliability indices match the target indices from Table 3 for the three NCHRP Example bridges. The three bridges are not shown in Figure 1 because they would overlay the "NCHRP Example Bridges" data. Also, using a live load factor of 1.474, the Wyoming traffic characteristics, *X* = 0.15 and *Y* = 0.05, and varying *W* overlaps the "NCHRP Expected Beta" target reliability on Figure 1 as shown with the label "I-80 WIM: LLF = 1.474." Load factors are typically rounded to the appropriate 0.05.

To maintain the expected reliability at the Service II limit state, bridges on I-80 should be designed with a live load factor in the range of 1.47. This represents a 13 percent increase in the live load factor for the current AASHTO Service II value. In comparison, the Phase I: Strength I study recommended between a 14 percent (2.00/1.75 – 1) and a 9 percent (1.90/1.75 – 1) increase in the Strength I live load factor due to the Wyoming traffic characteristics on I-80.

With the variability of target reliability, and dead and live load ratios as shown next, a Service II live load factor of 1.45 is recommended as a reasonable value for bridges on I-80. This value represents an 11.5 percent increase over the AASHTO live load factor of 1.30. Again, this increase is similar to, and consistent with, the recommended increase in the Phase I: Strength I live load factor increase.

BRIDGE DATABASE RELIABILITY RESULTS

A database of 112 in-service Wyoming steel bridges was compiled to examine the Service II limit state reliability. Both positive and negative moment regions were analyzed. These bridges included noncomposite and composite bridges, both with and without wearing surfaces, and designed with LRFD, LFD, and ASD methods. Ratios of wearing surface load to total dead load, X, range from 0 percent to 20.4 percent, and ratios of composite dead to total dead load, Y, range from -2.7 percent (wearing surface creates opposite moment) to 17.7 percent. The ratio of total dead load to live load, W, ranges from 19 percent to 272 percent. The span lengths ranged from 37 ft to 170 ft for positive moment, and from 34 ft to 138 ft for negative moment (negative moment span was average of adjacent spans). For the reliability analyses, the nominal design strength (resistance) was set to the sum of the factored load effect, i.e., it optimized the section consistent with previous work. Figure 2 and Figure 3 show the results for positive moment and negative moment, respectively.

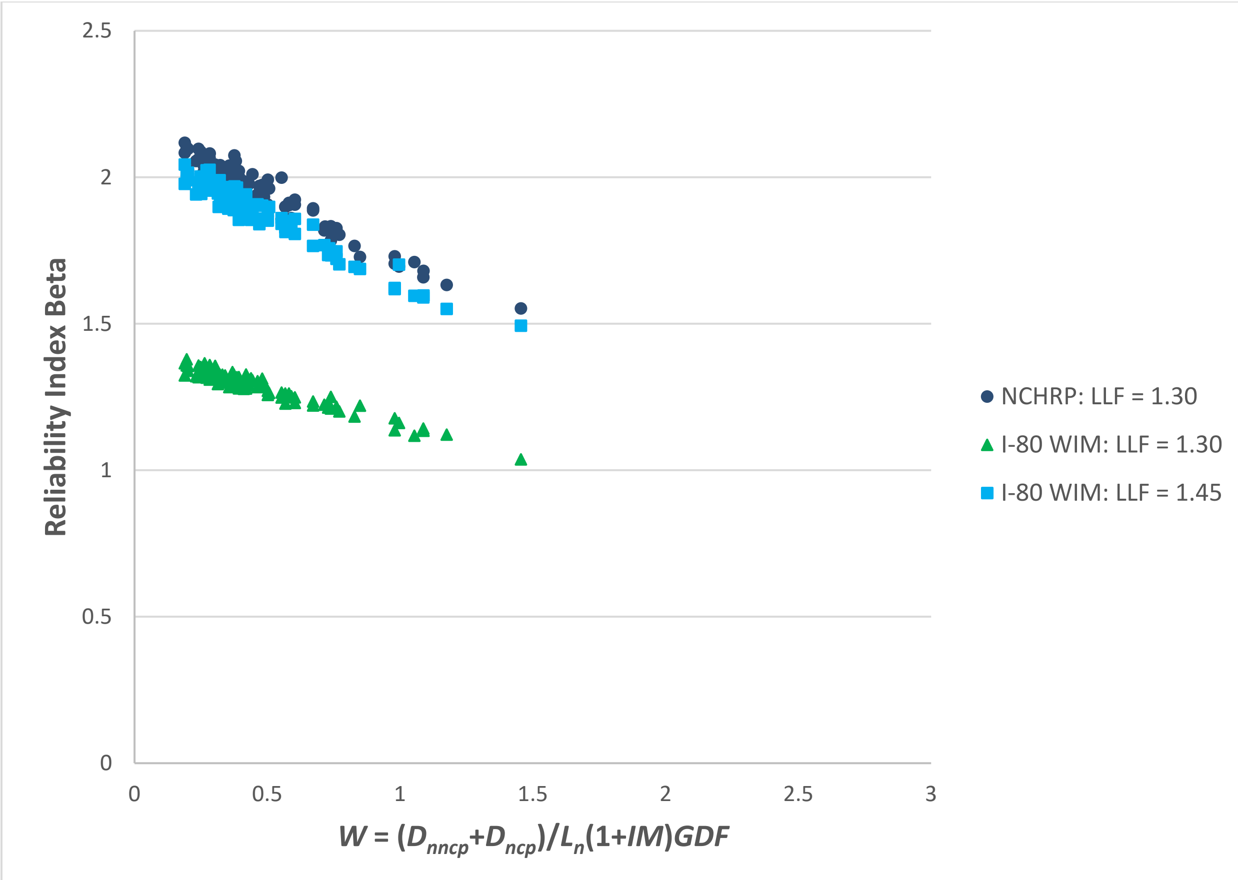

Figure 2. Service II Positive Moment Reliability (Optimized In-Service Bridges)

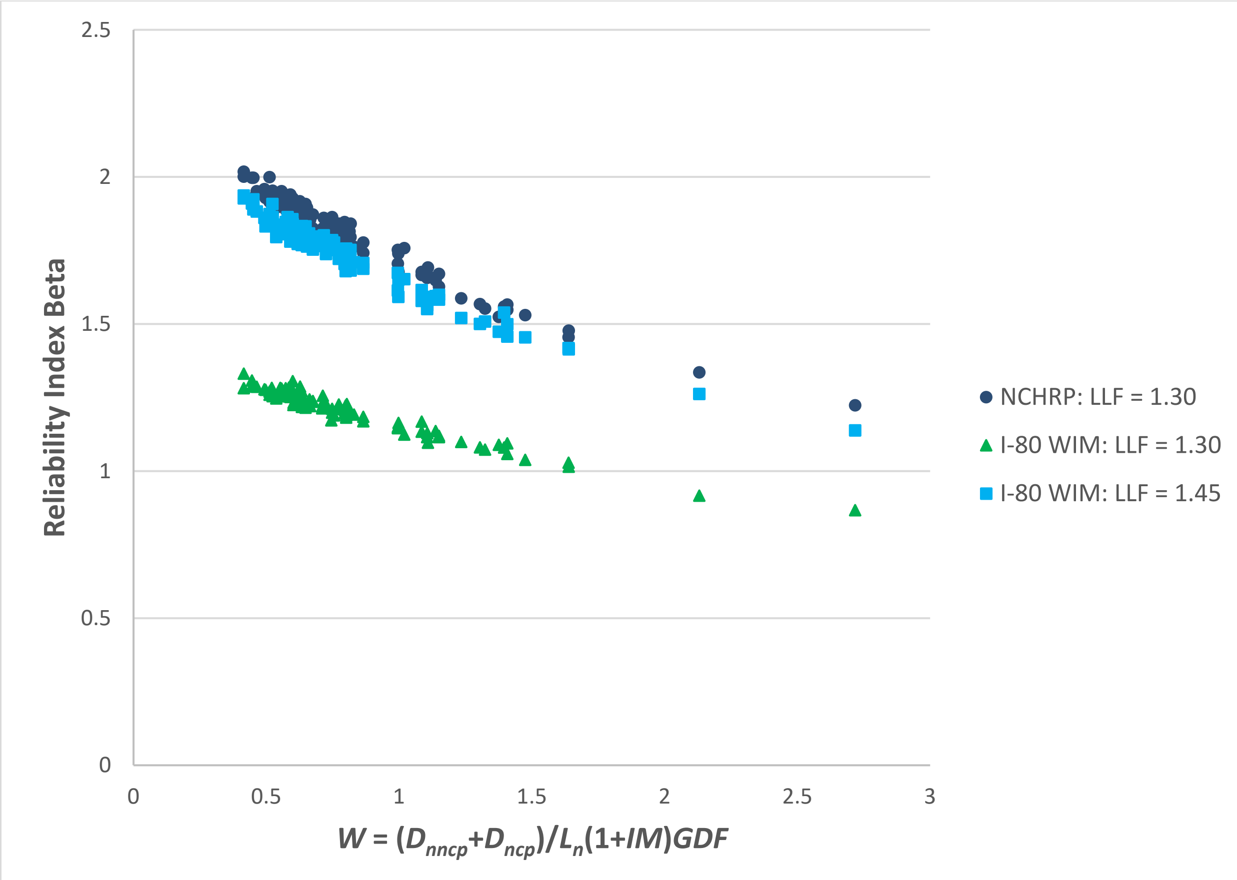

Figure 3. Service II Negative Moment Reliability (Optimized In-Service Bridges)

The data labeled "NCHRP: LLF = 1.30" represents the target reliability of the AASHTO Service II limit state according to the NCHRP AASHTO LRFD development work. The data labeled "I-80 WIM: LLF = 1.30" shows that, if the database of bridges were optimized for the Service II design requirement, the reliability against exceeding the Service II limit state given the Wyoming truck traffic characteristics on I-80 does not meet serviceability expectations using a Service II live load factor *LLF* = 1.30.

The data labeled "I-80 WIM: LLF = 1.45" demonstrates that if a Service II live load factor of 1.45 replaces the AASHTO factor of 1.30 for bridges designed for I-80 subject to the Wyoming truck traffic characteristics, those bridges would have similar reliability against exceeding the Service II limit state as target reliability indices expected in AASHTO requirements. Note that whether this is adequate, or not, assumes that the current AASHTO method is "correct."

A practical live load factor of 1.45, instead of the 1.474 determined previously, is reasonable and adequate because significant scatter exists in the target reliability indices for the range of variables for the database of bridges. Note that a live load factor of 1.5 could be justified as well.

Similar results and conclusions are illustrated for the reliability indices shown by the span length in Figure 4 for positive moment and Figure 5 for negative moment.

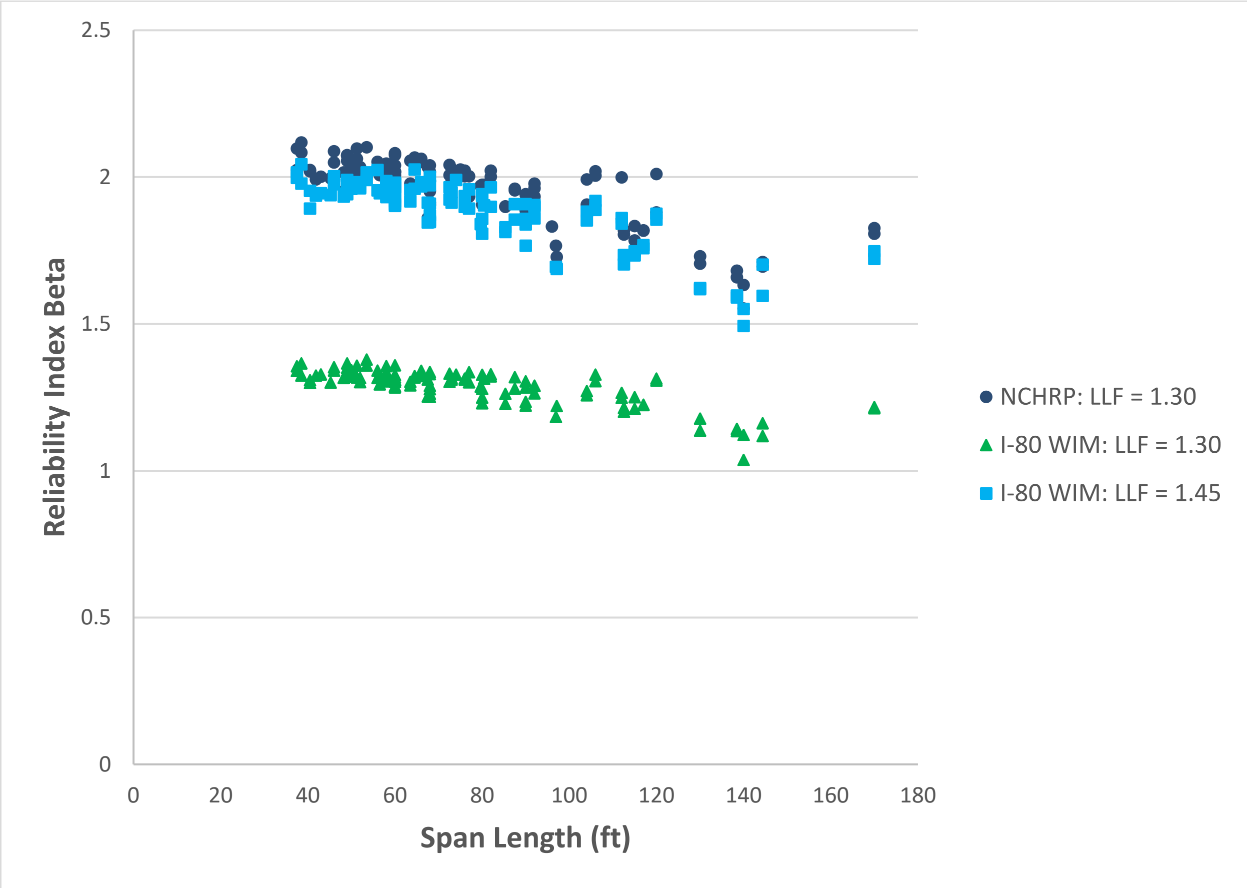

Figure 4. Service II Positive Moment Reliability vs Span Length (Optimized In-Service Bridges)

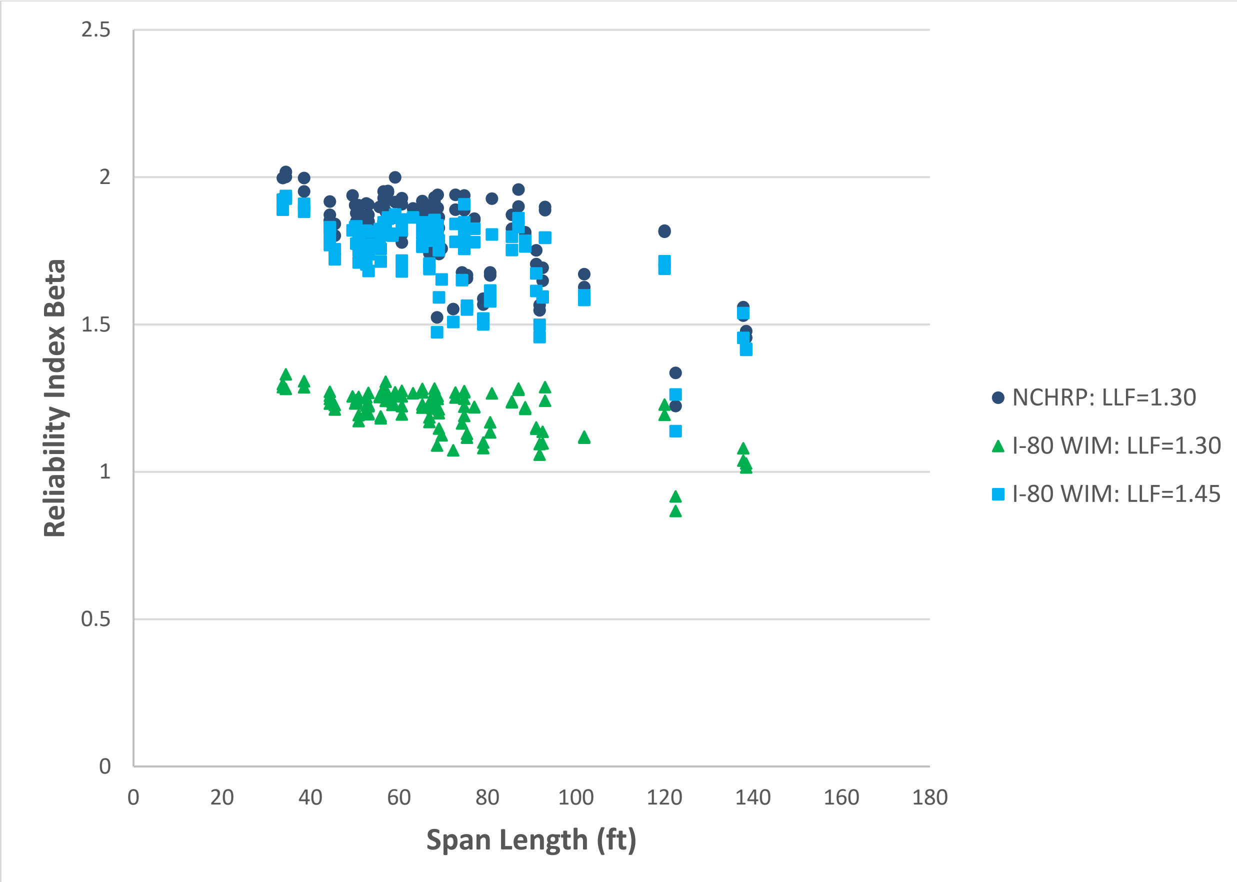

Figure 5. Service II Negative Moment Reliability vs Span Length (Optimized In-Service Bridges)

The variability of the target reliability indices data labeled "NCHRP: LLF = 1.30" supports the recommended use of a larger live load factor of 1.45.

SERVICE II ASSESSMENT OF STEEL BRIDGES ON I-80 CORRIDOR

The reliability analyses used an optimized strength, R_n, to calibrate the Service II live load factor. However, the bridges in the database represent in-service steel bridges on the I-80 corridor. Therefore, using the actual bridge sections, an assessment of the reliability against exceeding the Service II limit state for the steel bridge inventory on I-80 can be conducted. The reliability of each bridge is based on the proposed live load factor γ_L = 1.45, where the rating factor is:

$$RF = \frac{R_n - \left(\frac{S_n}{S_x}\right) D_{nncp} - \left(\frac{S_n}{S_{3n}}\right)(D_{ncp} + D_{nw})}{1.45 L_n (1 + IM) GDF} \tag{18}$$

In Figure 6 and Figure 7, the data labeled "I-80 WIM Optimized" illustrates the target reliability by using a live load factor of 1.45 and an optimized strength, R_n. The data labeled "I-80 WIM Actual" represents the reliability indices for the bridges using the actual strength of the bridge. Many steel bridges along the I-80 corridor do not meet expected reliability for the Service II limit state, as shown by the bridges that are below the target reliability data. These bridges all have a current rating factor of less than 1.0 using a live load factor of 1.45. Likewise, the bridges that have reliability indices above the target data represent bridges that have rating factors above 1.0. Several bridges have significantly high rating factors where the reliability index is shown capped at 4.0.

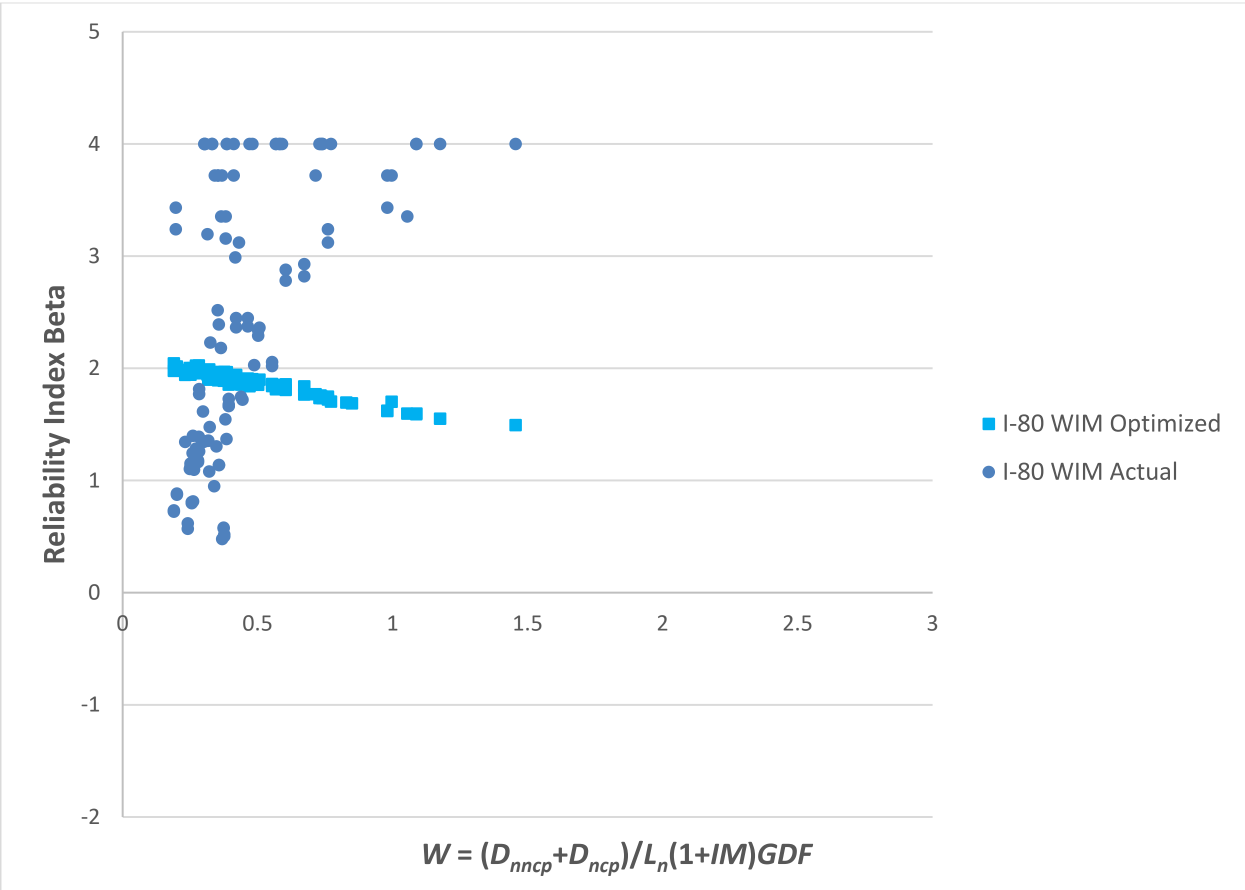

Figure 6. Service II Positive Moment Reliability Assessment (Actual R_n for In-Service Bridges Compared to Optimized Design)

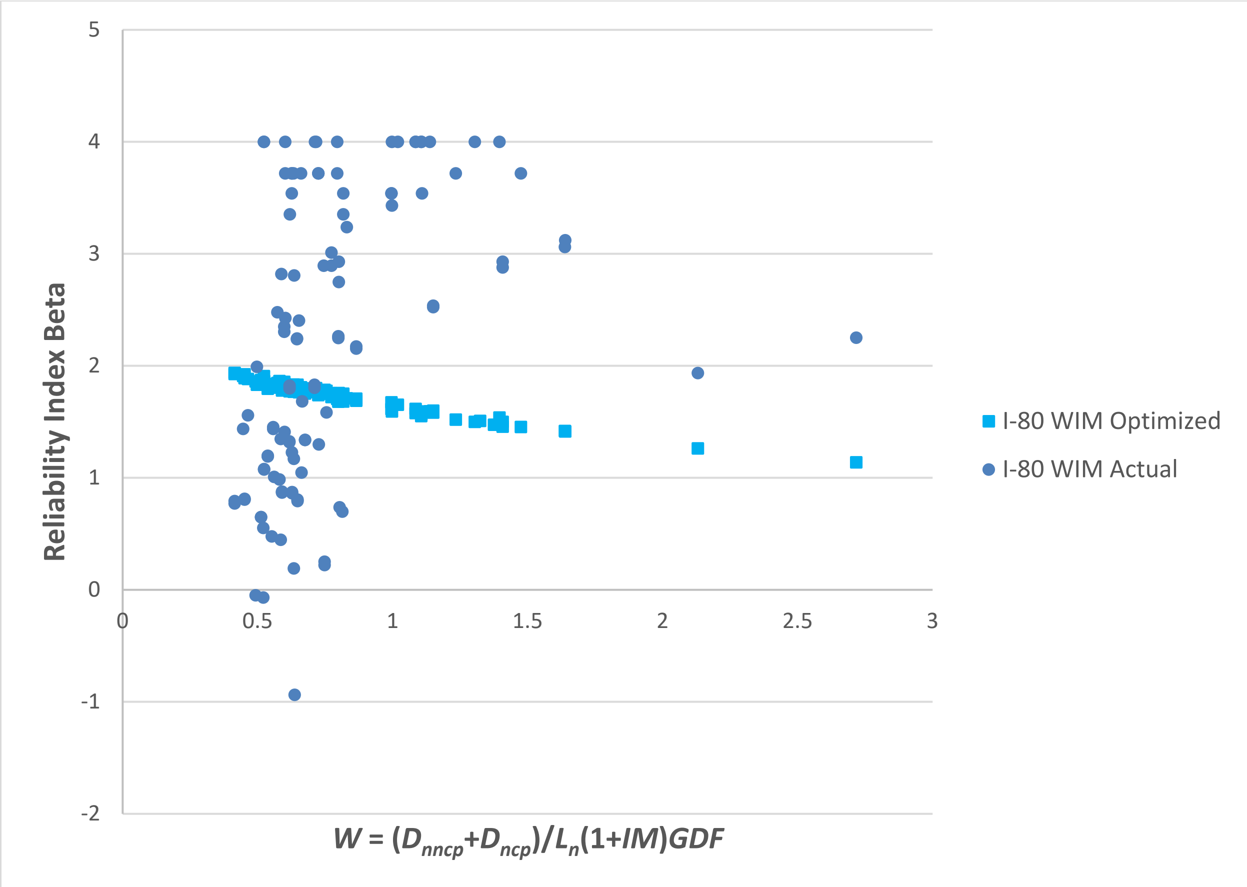

Figure 7. Service II Negative Moment Reliability Assessment (Actual R_n for In-Service Bridges Compared to Optimized Design)

Figure 8 and Figure 9 demonstrate the relation between the rating factor and exceeding the limit state and the rating factor and reliability, respectively.

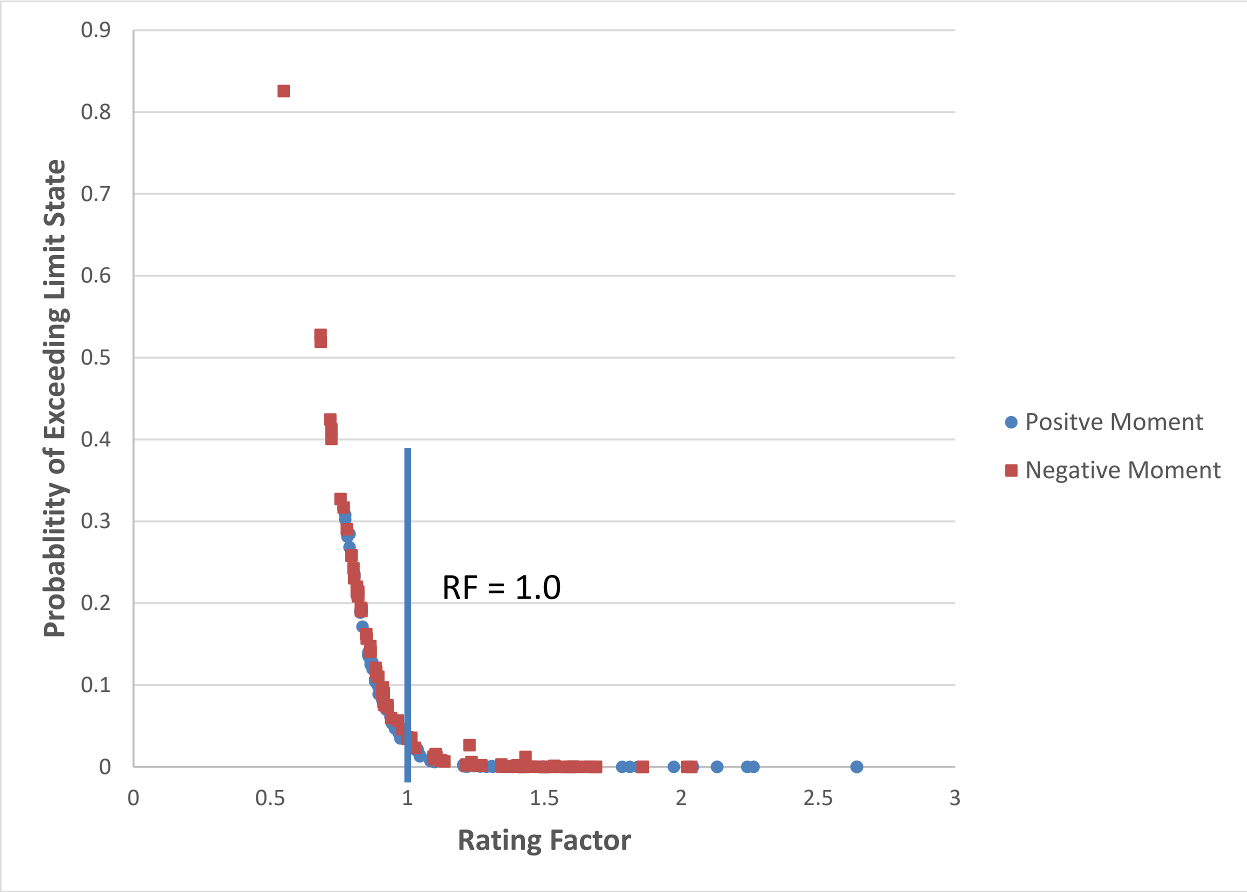

Figure 8. Service II Exceeding Limit State vs Rating Factor for γ_L = 1.45

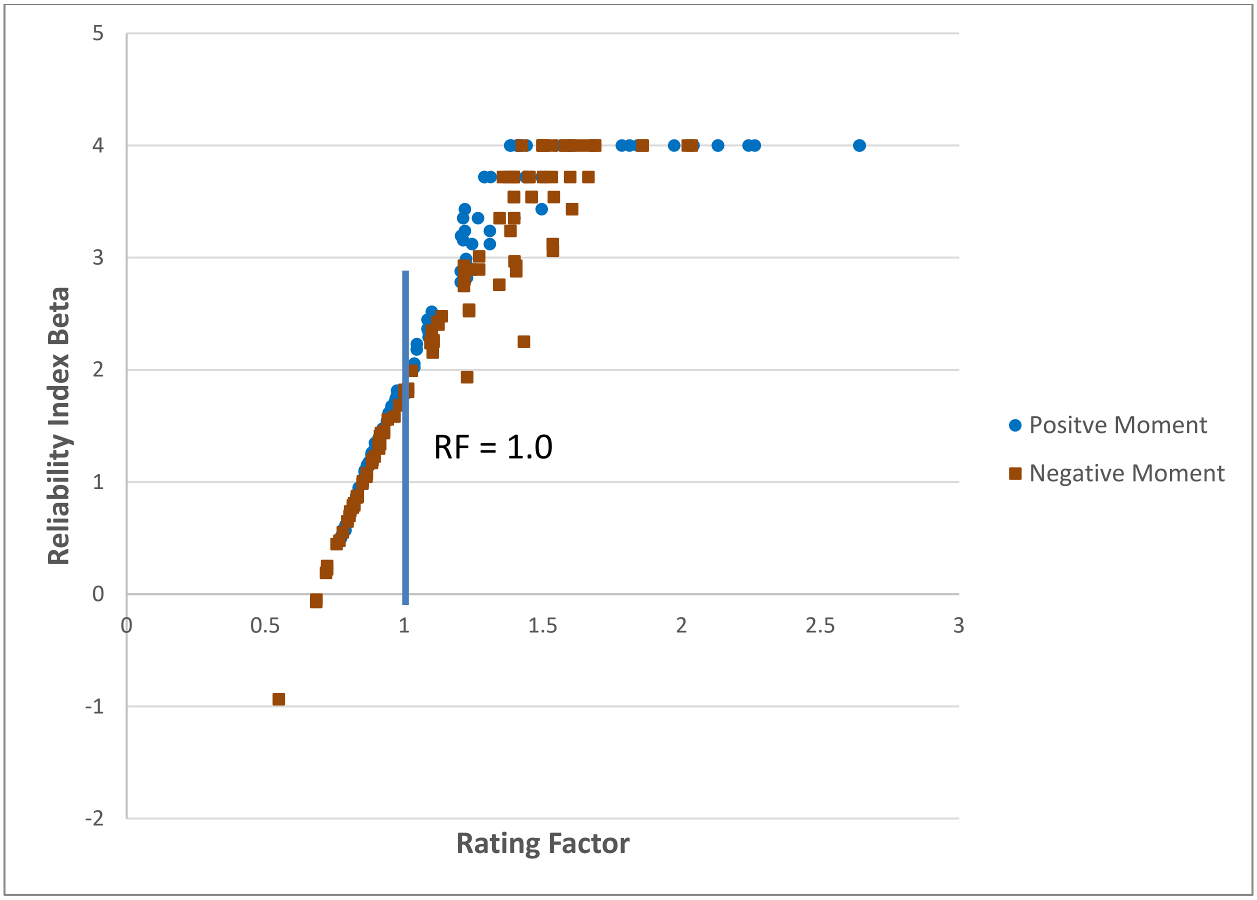

Figure 9. Service II Reliability vs Rating Factor for γ_L = 1.45

Figure 8 shows that when the rating factor falls below 1.0 using a Service II live load factor of 1.45, the probability of exceeding the Service II limit state transitions from small and consistent to unacceptable rather quickly. Figure 9 demonstrates that bridges with a rating factor equal or greater than 1.0 have reliability indices approximately 1.8 and larger, consistent with the target reliability shown in previous figures.

There are 47 positive-moment and 45 negative-moment cases in the 112 bridges in the database where the rating factor is less than 1.0. These represent bridges on I-80 where the reliability against exceeding the Service II limit state does not meet the expectation of the AASHTO design requirements. This indicates that these bridges should be expected to have more yielding and more permanent set than the AASHTO LRFD Specifications allow at the Service II limit state.

Figure 10 repeats Figure 8, but using the current Service II live load factor of 1.30. The results indicate that, for Wyoming truck traffic on the I-80 corridor, continued use of 1.30 results in bridges that do not meet expectations in terms of the Service II limit state. The small and consistent probability of exceeding the limit state transitions to unacceptable at rating factors well above 1.0.

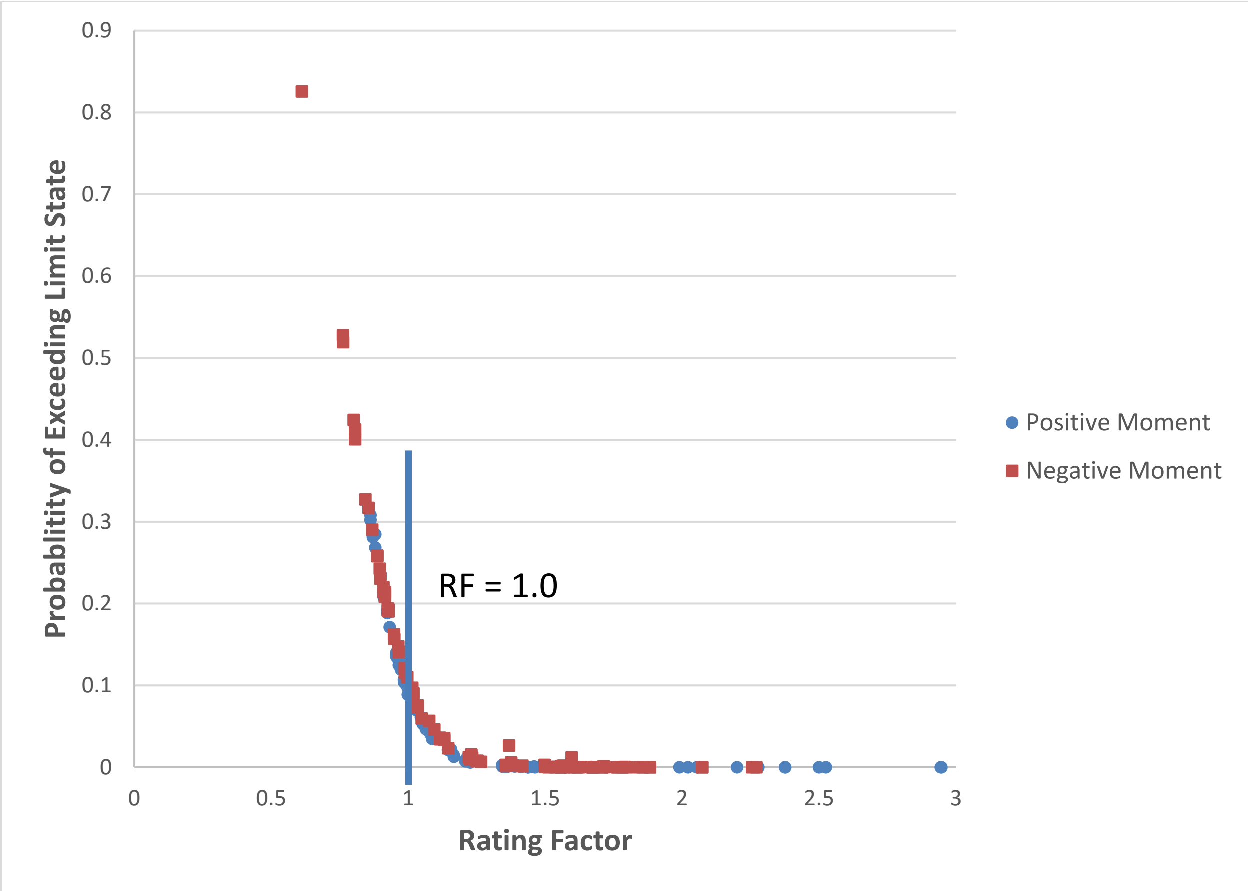

Figure 10. Service II Exceeding Limit State vs Rating Factor for γ_L = 1.30

CHAPTER 3. SERVICE III DESIGN LIMIT

The AASHTO Service III design limit state controls tension in prestressed concrete bridges with the objective of crack control. The design requirement is to limit the tensile stress in the outer fiber of the beam with bonded prestressing tendons. For components subjected to severe corrosive conditions, this limit is $0.0928\sqrt{f'_c}$, where f'_c is the compressive strength of the concrete. For components subjected to not worse than moderate corrosive conditions subject to nominal dead load and factored live load, this limit is $0.19\sqrt{f'_c}$.

The nominal load on a prestressed concrete bridge girder is

$$D_{ngw} + D_{nnc} + D_{nps} + D_{nc} + D_{nw} + L_n(1 + IM)GDF \tag{19}$$

where:

D_{ngw} = Nominal Girder Weight Effect
D_{nc} = Nominal Noncomposite Component Dead Load Effect
D_{nps} = Nominal Prestress Loads Effect
D_{nc} = Nominal Composite Component Dead Load Effect
D_{nw} = Nominal Wearing Surface Load Effect
L_n = Nominal Live Load Effect (AASHTO HL-93)
IM = LRFD Design Dynamic Load Allowance (IM = 0.33)
GDF = Lateral Distribution Factor

The load effects for Service III are stresses.

The prestress load effect is composed of a bending moment and an axial force

$$D_{nps} = D_{npsM} + D_{npsA} \tag{20}$$

where:

D_{npsM} = Nominal Prestress Load Stress due to Moment
D_{npsA} = Nominal Prestress Load Stress due to Axial Force

Rewritten in terms of stress, moment, and axial force

$$D_{npsM} = \frac{M_{nps}}{S_x} = \frac{P_{nps} \times e}{S_x} \tag{21}$$

$$D_{npsA} = \frac{P_{nps}}{A_x} \tag{22}$$

where:

M_{nps} = Nominal Prestress Loads Moment
P_{nps} = Nominal Effective Prestress Force
e = Eccentricity from N.A. to Prestress Force

S_x = Noncomposite Section Modulus
A_x = Noncomposite Section Area

The prestress force includes prestress losses, which can be shown as:

$$P_{nps} = \left(f_{pi} - \Delta f_s\right) A_{ps} \tag{23}$$

where:
f_{pi} = Initial Prestressing (percentage of ultimate)
Δf_s = Prestress Losses
A_{ps} = Area of Prestressing Reinforcement

Then, the optimized (meaning at the design limit where the rating factor, or design performance ratio, is equal to 1.0) Service III design limit is

$$R_n = D_{ngw} + D_{nncp} + D_{npsM} + D_{npsA} + D_{ncp} + D_{nw} + \gamma_L L_n (1 + IM) GDF \tag{24}$$

where:
R_n = Optimized (Service III Rating Factor = 1.0) Nominal Resistance
γ_L = Live Load Factor (γ_L = 0.80 AASHTO Service III))

The Service III design limit is checked using stresses. The stress limit for beams subjected to not worse than moderate corrosion condition ($0.19\sqrt{f'_c}$) is

$$0.19\sqrt{f'_c} = \frac{D_{ngw}}{S_x} + \frac{D_{nnc}}{S_x} + \frac{D_{nps}}{S_x} + \frac{D_{nc}}{S_c} + \frac{D_{nw}}{S_c} + \gamma_L \frac{L_n}{S_c} (1 + IM) GDF \tag{25}$$

where:
D_{ngw} = Nominal Girder Weight Moment
D_{nncp} = Nominal Noncomposite Dead Load Moment
D_{nps} = Nominal Prestress Loads Moment
D_{ncp} = Nominal Composite Dead Load Moment
D_{nw} = Nominal Wearing Surface Load Moment
L_n = Nominal Live Load Moment (AASHTO HL-93)
S_x = Noncomposite Section Modulus
S_c = Composite Section Modulus

All of the ratios have units of stress.

The design limit, and thus the reliability analyses, depend on the relative magnitude of girder weight, noncomposite dead, composite dead, wearing surface, and live loads. Therefore, the following ratios are computed to support the reliability analyses.

Let W be the ratio of nominal girder weight, noncomposite, and composite dead load to nominal live load

$$W = \frac{D_{ngw} + D_{nncp} + D_{ncp}}{L_n(1 + IM)GDF} \tag{26}$$

and Y be the ratio of composite dead load to nominal girder weight, noncomposite, and composite dead load

$$Y = \frac{D_{nc}}{D_{ngw} + D_{nncp} + D_{ncp}} \tag{27}$$

and X be the ratio of wearing surface dead load to nominal girder weight, noncomposite, and composite dead load

$$X = \frac{D_{nw}}{D_{ngw} + D_{nncp} + D_{ncp}} \tag{28}$$

For prestressed concrete, the stresses are calculated using the actual section properties for each real bridge.

PRESTRESSED CONCRETE BRIDGE DATABASE

A database of 60 in-service Wyoming prestressed concrete bridges was compiled to examine the Service III limit state reliability. These bridges were constructed with prestressed concrete beam shapes consisting mainly of AASHTO I-beams and AASHTO-PCI bulb-tees (PCI 2014) along with some additional tee- and box-beams. There are no prestressed concrete bridges on the I-80 corridor through Wyoming, so prestressed concrete bridges on other Wyoming highways were utilized in this study as if these bridges were on the I-80 corridor.

Positive moment regions were analyzed because these locations controlled. These bridges included noncomposite and composite bridges, simple spans, and simple spans made continuous, both with and without wearing surfaces and designed with LRFD, LFD, and ASD methods. Ratios of wearing surface load to total dead load, X, range from 0 percent to 25.0 percent, and ratios of composite dead to total dead load, Y, range from -7.1 percent (wearing surface creates opposite moment) to 21.9 percent. The ratio of total dead load to live load, W, ranges from 22.4 percent to 475 percent. The span lengths ranged from 34 ft to 160 ft. For the reliability analyses, the nominal design strength (resistance) was set to the sum of the factored load effect, i.e., it optimized the section consistent with previous work.

SERVICE III RELIABILITY

In the limit state equation, Z represents the stresses exceeding the stress limits, R_n, for $0.19\sqrt{f'_c}$ for beams subjected to not worse than moderate corrosion condition. Because the design is optimized, the stress limit is determined by the design requirements (rating factor equal to 1.0).

$$\begin{aligned} Z = {} & R - D_{gw} - D_{nc} - D_{ps} - D_c - D_w - LL(1+I)(GDF) \\ = {} & R - D_{gw} - D_{nc} - D_{ps} - D_c - D_w - L \end{aligned} \tag{29}$$

Using the statistical properties for the random variables from the Phase I: Strength I reliability study and National Academies of Sciences, Engineering, and Medicine (2014):

R = Stress Allowable – Lognormal Distribution

μ_R	$= \lambda_R R_n$	Mean
λ_R	= 1.05	Bias
R_n	$= D_{ngw} + D_{nncp} + D_{nps} + D_{ncp} + D_{nw} + \gamma_L L_n(1+IM)GDF$	
COV_R	= 0.075	Coefficient of Variation

D_{gw} = Girder Weight Stress – Normal Distribution

μ_{Dgw}	$= \lambda_{Dgw} D_{ngw}$	Mean
λ_{Dgw}	= 1.03	Bias
D_{ngw}	=	Nominal Girder Weight Load
COV_{Dgw}	= 0.08	Coefficient of Variation

D_{nc} = Noncomposite Stress – Normal Distribution

μ_{Dnc}	$= \lambda_{Dnc} D_{nncp}$	Mean
λ_{Dnc}	= 1.05	Bias
D_{nncp}	=	Nominal Noncomposite Dead Load
COV_{Dnc}	= 0.10	Coefficient of Variation

D_{ps} = Prestress Loads Stress – Normal Distribution

μ_{Dps}	$= \lambda_{Dps} D_{nps}$	Mean
λ_{Dps}	= 0.96	Bias
D_{nps}	=	Nominal Prestress Load
COV_{Dps}	= 0.10	Coefficient of Variation

D_c = Composite Stress – Normal Distribution

μ_{Dc}	$= \lambda_{Dnc} D_{ncp}$	Mean
λ_{Dc}	= 1.05	Bias
D_{ncp}	=	Nominal Composite Dead Load
COV_{Dc}	= 0.10	Coefficient of Variation

D_w = Wearing Surface Stress – Normal Distribution

μ_{Dw}	$= \lambda_{Dw} D_{nw}$	Mean
λ_{Dw}	= 1.00	Bias
D_{nw}	=	Nominal Wearing Surface Load
COV_{Dw}	= 0.25	Coefficient of Variation

$L = LL(1+I)GDF$ = Live Load Stress on Girder

μ_L	$= \mu_{LL}(1+\mu_I)\mu_{GDF}$	Mean
COV_L	= 0.14	Determined in Service II section

Prestressed concrete bridges were taken from the WYDOT bridge inventory for use in this study. Bias and COV values for the parameters used to calculate the prestressing force are shown in Table 7.

Table 7. Bias and COV for Prestress Parameters

Variable	Bias, λ	COV
f_{pi}	0.97	0.08
Δf_s	1.05	0.1
A_{ps}	1.01176	0.0125

The prestress bridges were individually evaluated to determine the effective bias and COV for the prestress force, which is used to determine both flexural and axial stresses. The resulting bias and COV are reported above under the D_{ps} component.

SERVICE III TARGET RELIABILITY

The target reliability for the AASHTO Service III limit state can be determined by applying the NCHRP live load characteristics to the Service III limit state and the AASHTO Service III live load factor of $\gamma_L = 0.80$.

As was done for Service II, the Live Load Bias $\lambda_L = 1.18$ and $COV_L = 0.18$ (NCHRP live load model) demonstrate the target Service III reliability indices currently expected in the AASHTO LRFD Specifications. The prestressed concrete bridges were analyzed with BRASS-GIRDER™ (2019) to obtain the critical rating factor along with the associated reliability index and failure probability.

MONTE CARLO SIMULATIONS FOR RELIABILITY INDICES

To determine the reliability index for the AASHTO Service III limit state equation, statistical methods are used to predict the probability that the limit state equation is less than zero (probability that the strength is less than the combined load effect). Because algebraic sums and products are with mixed lognormal and normal variables in the limit state equation, Monte Carlo simulation was used to determine the reliability indices.

The Monte Carlo simulations for prestressed concrete are performed similar to those for steel, except the limit state equation Z is computed by simulating the R, D_{gw}, D_{nc}, D_{ps}, D_c, D_w, and L random variables according to their distributions and 10,000 trials are used to determine how many failures occur.

The boost C++ library random number generator, lognormal, and normal functions are used for the Monte Carlo simulations. The previous section defined the mean and coefficient of variation for the variables R, D_{gw}, D_{nc}, D_{ps}, D_c, D_w, and L. However, because R is lognormally distributed, the mean of $\ln(R)$ and the standard deviation of $\ln(R)$ are required.

REQUIRED LIVE LOAD FACTOR TO ATTAIN TARGET RELIABILITY INDICES

To determine the appropriate Service III live load factor, γ_L, that would be required to meet the target reliability for Wyoming's prestressed concrete bridges with the Wyoming traffic characteristics, the Monte Carlo simulations were run to find the live load factor that produced the similar target reliability indices as with the NCRHP load model. Figure 11 shows the results for positive moment.

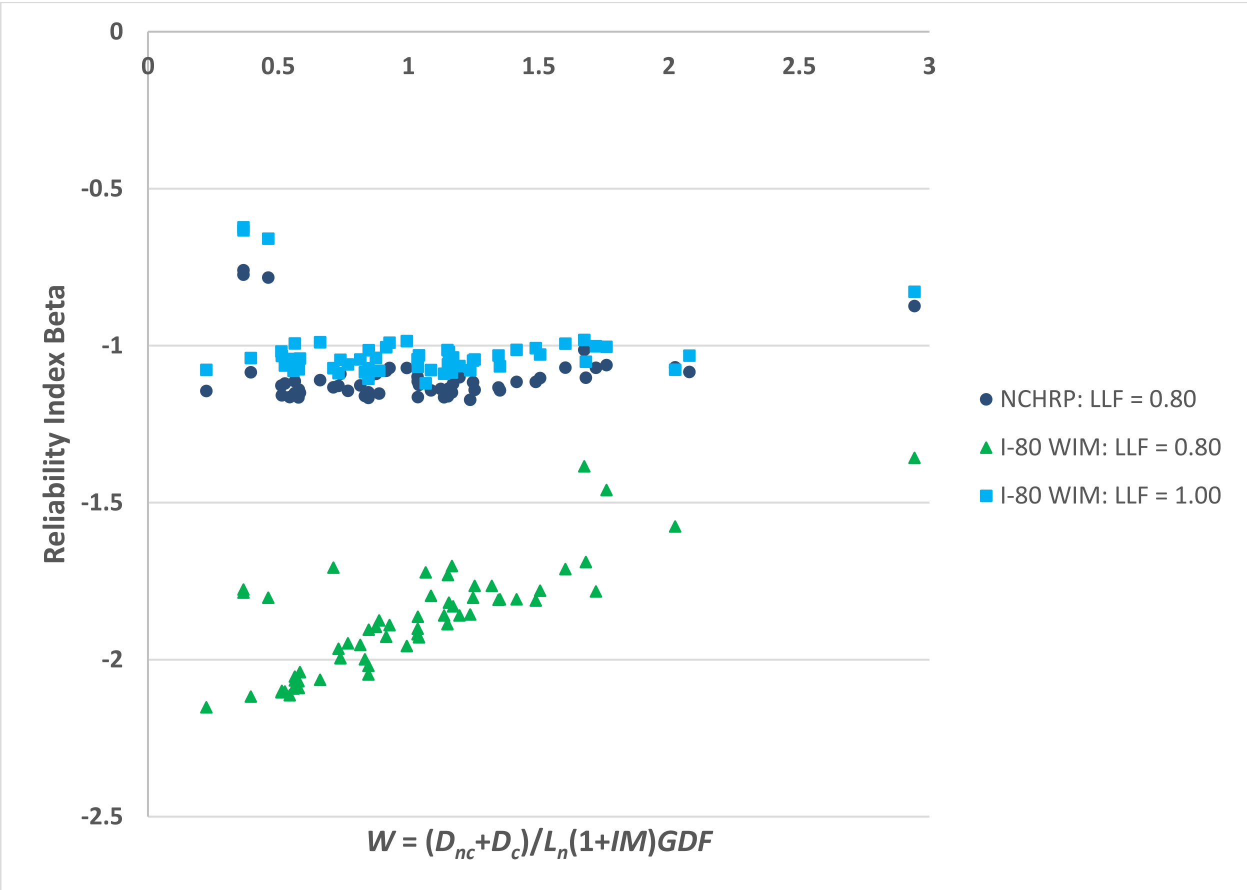

Figure 11. Service III Positive Moment Reliability (Optimized In-Service Bridges)

The reliability indices for the Service III limit state for prestressed concrete bridges are negative, which is a result of using a live load factor less than 1.0.

The data labeled "NCHRP: LLF = 0.80" represents the target reliability of the AASHTO Service III limit state according to the NCHRP AASHTO LRFD development work. The data labeled "I-80 WIM: LLF = 0.80" represents the reliability if the database of bridges were optimized for the current Service III design requirement given the Wyoming truck traffic characteristics on I-80. The optimized betas are much lower than the target betas, which indicates that the current live load factor of γ_L = 0.80 does not meet the serviceability expectations in the AASHTO Service III limit state for Wyoming traffic on I-80.

The data labeled “I-80 WIM: LLF = 1.00” demonstrates that if a Service III live load factor of 1.00 replaces the current AASHTO factor of 0.80 for bridges designed for I-80 subject to the Wyoming truck traffic characteristics, those bridges would have similar reliability against exceeding the Service III limit state as target reliability indices expected in AASHTO requirements. Note that whether this is adequate, or not, assumes that the current AASHTO method is “correct.”

To maintain the expected reliability at the Service III limit state, it is recommended that bridges on I-80 be designed with a live load factor of 1.0. This represents a 25 percent increase over the current AASHTO live load factor of 0.80. Again, this increase is similar to, and consistent with, the recommended increase in the Phase I: Strength I live load factor increase as well as the Service II live load factor increase recommendation.

Similar results and conclusions are illustrated for the reliability indices are shown by the span length in Figure 12 for positive moment.

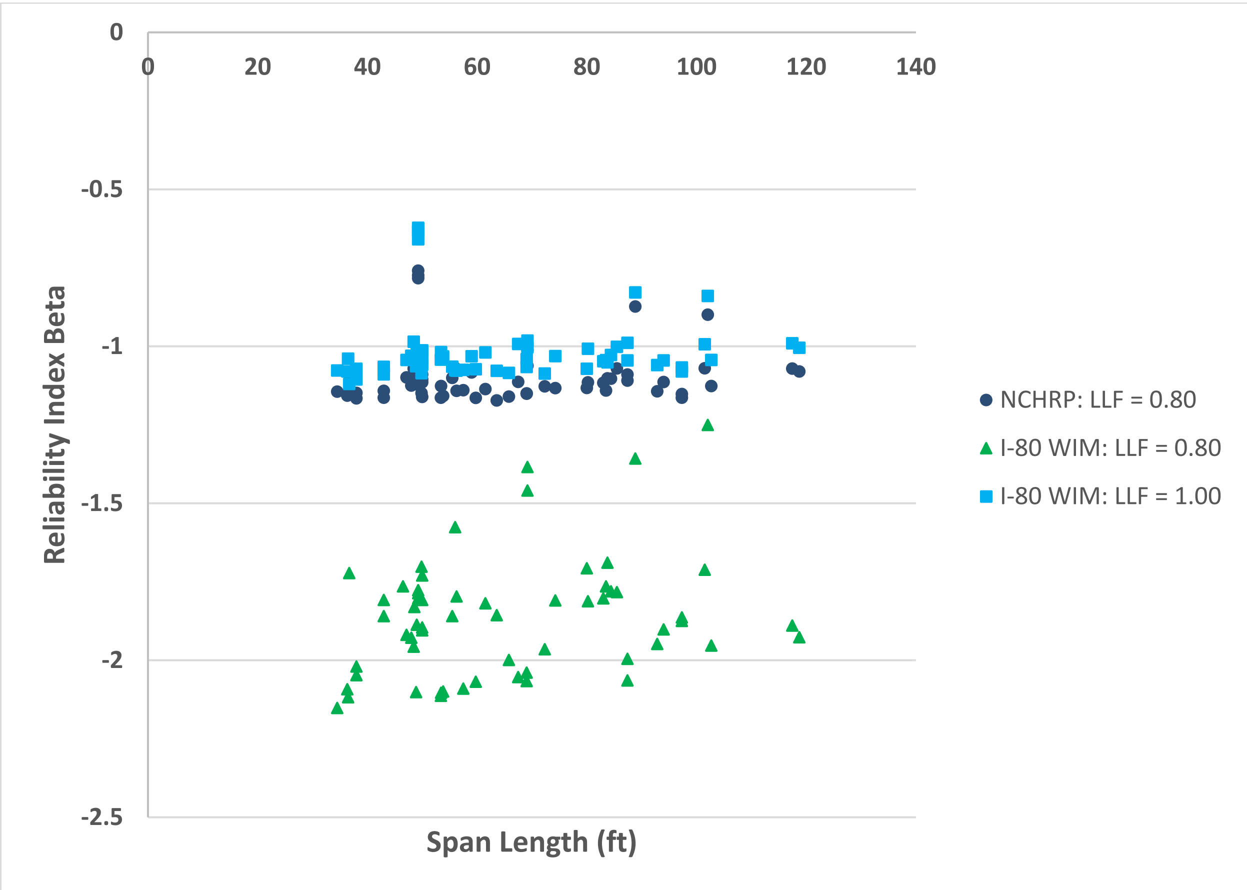

Figure 12. Service III Positive Moment Reliability vs Span Length (Optimized In-Service Bridges)

The variability of the target reliability indices data labeled “NCHRP: LLF = 0.80” supports the recommended use of a larger live load factor of 1.00.

SERVICE III ASSESSMENT OF PRESTRESSED CONCRETE BRIDGES IN WYOMING

The reliability analyses used an optimized strength, R_n, to calibrate the Service III live load factor. However, the bridges in the database represent in-service prestressed concrete bridges throughout Wyoming. Therefore, using the actual bridge sections, an assessment of the reliability against exceeding the Service III limit state for the prestressed concrete bridge inventory in Wyoming can be conducted assuming these bridges were on the I-80 corridor. The reliability of each bridge is based on the proposed live load factor γ_L = 1.00, where the rating factor is:

$$RF = \frac{R_n - D_{ngw} - D_{nncp} - D_{nps} - D_{ncp} - D_{nw}}{1.00 L_n (1 + IM) GDF} \tag{30}$$

In Figure 13, the data labeled "I-80 WIM Optimized" illustrates the target reliability by using a live load factor of 1.00 and an optimized strength, R_n. The data labeled "I-80 WIM Actual" represents the reliability indices for the bridges using the actual strength of the bridge. Many prestressed bridges in Wyoming do not meet expected reliability for the Service III limit state as shown by the bridges that are below the target reliability data. These bridges all have a current rating factor of less than 1.0 using a live load factor of 1.00. Likewise, the bridges that have reliability indices above the target data represent bridges that have rating factors above 1.0. The reliability index is shown capped at ±4.0.

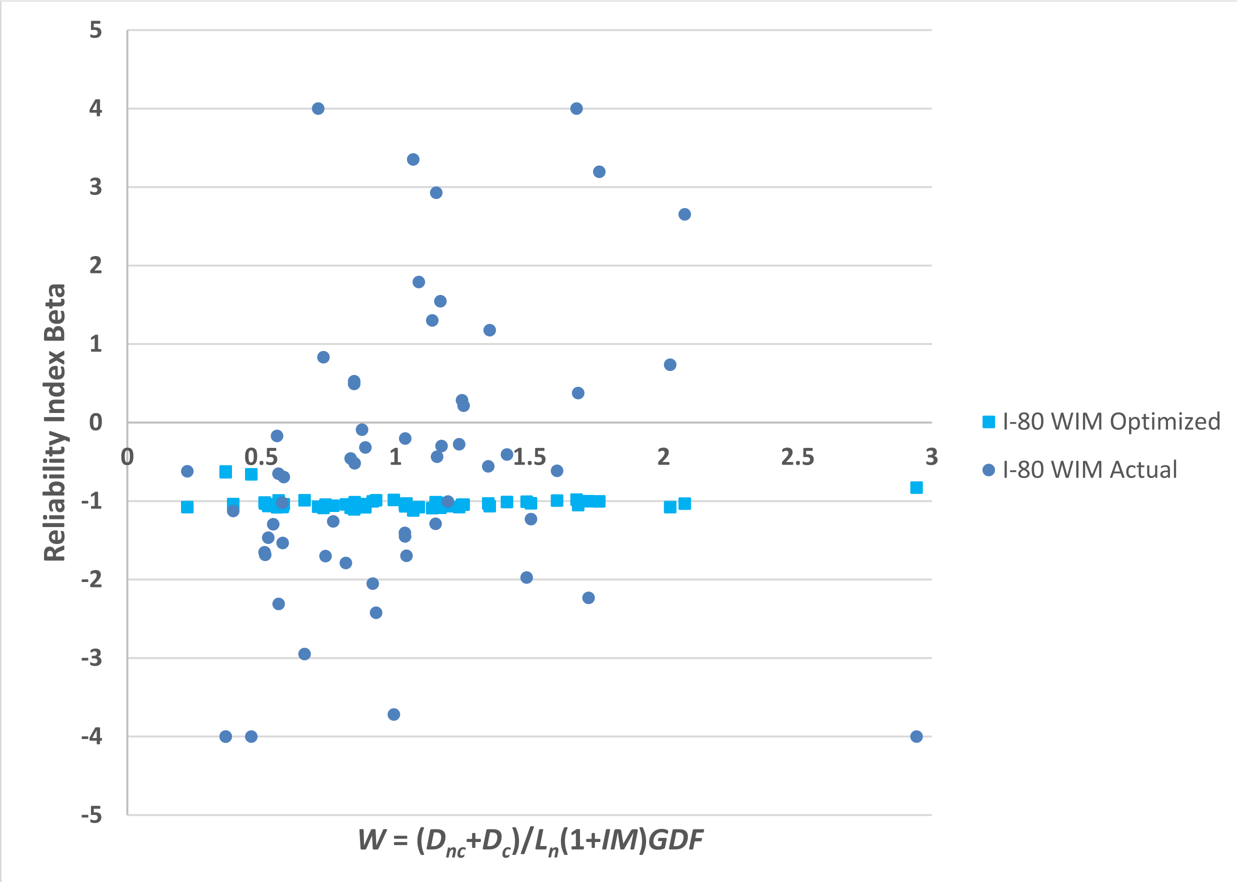

Figure 13. Service III Positive Moment Reliability Assessment (Actual R_n for In-Service Bridges Compared to Optimized Design)

Figure 14 and Figure 15 demonstrate the relation between the rating factor and exceeding the limit state and the rating factor and reliability, respectively.

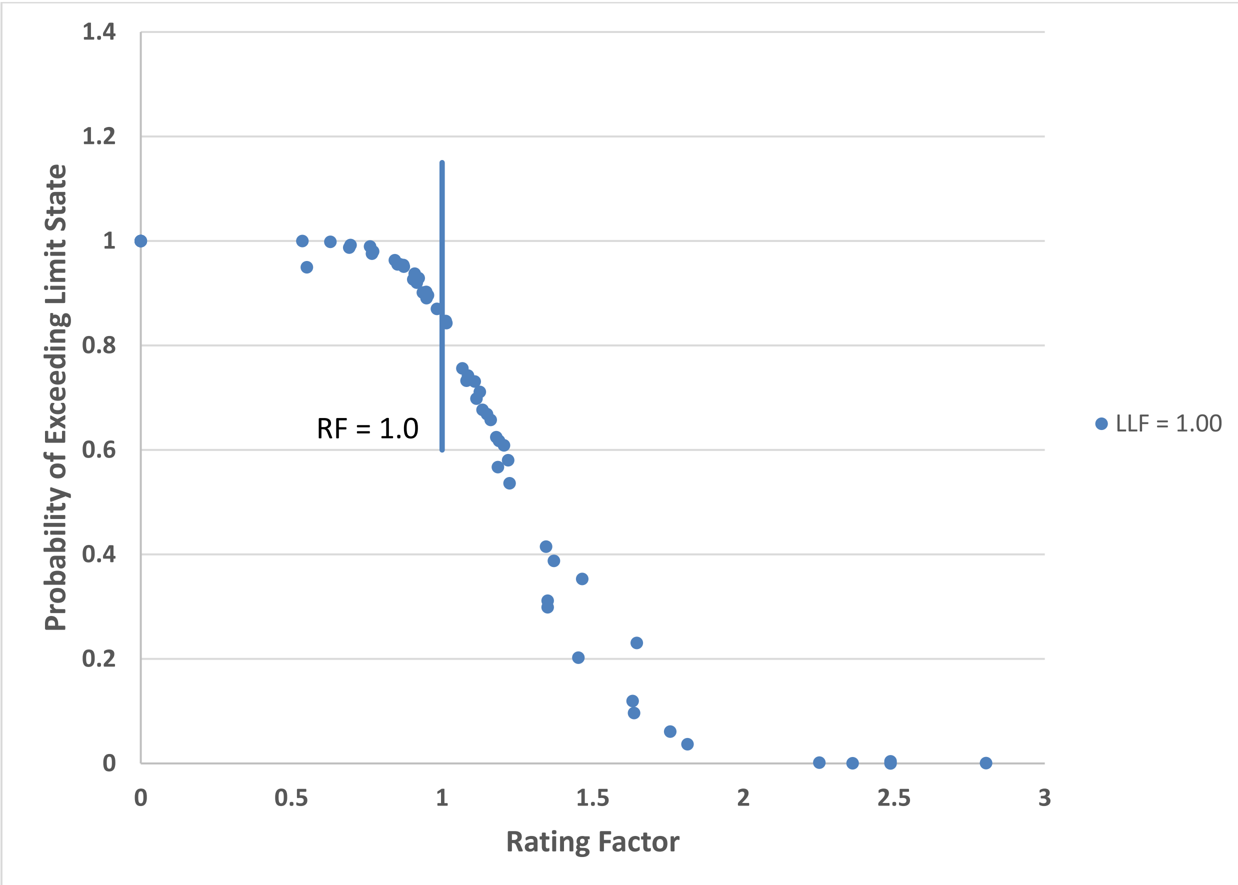

Figure 14. Service III Exceeding Limit State vs Rating Factor for γ_L = 1.00

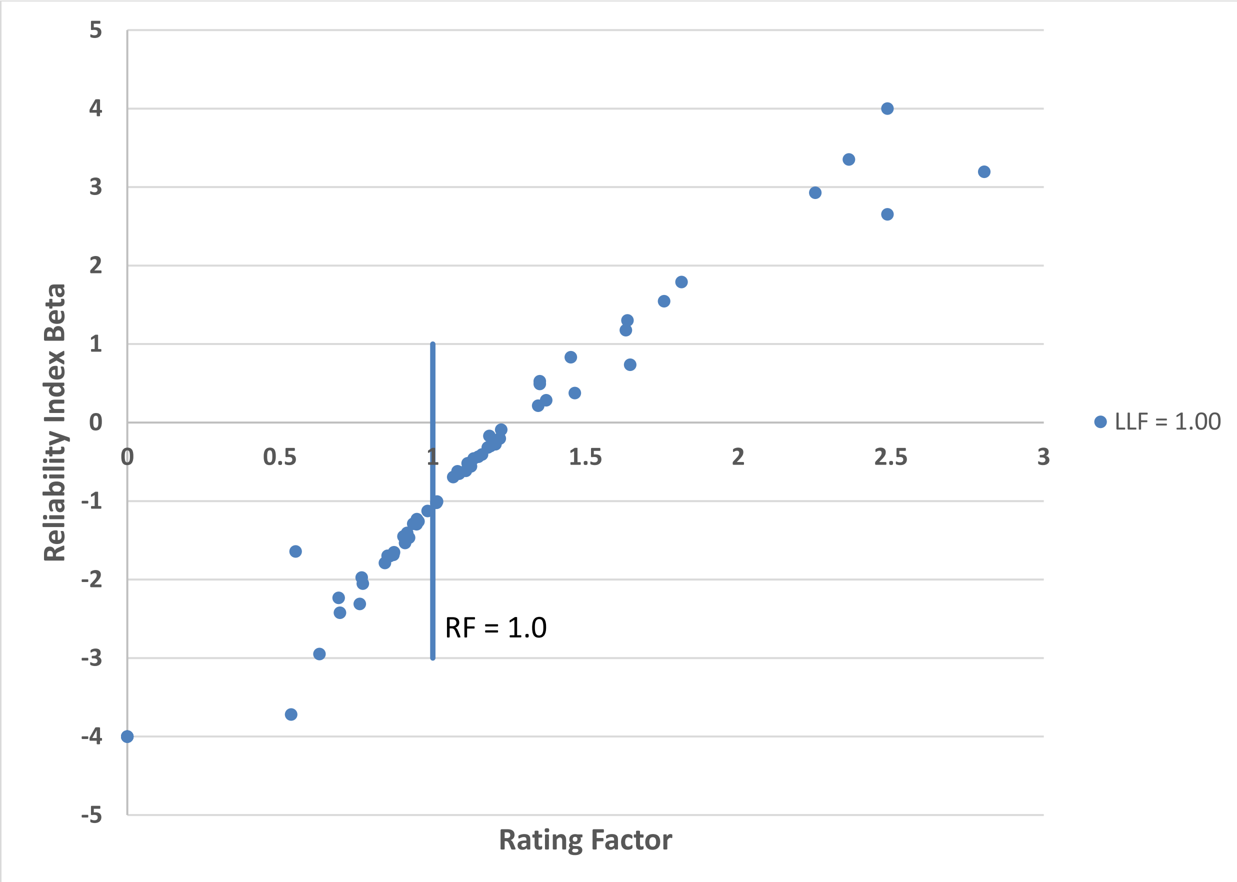

Figure 15. Service III Reliability vs Rating Factor for γ_L = 1.00

There are 26 positive-moment cases in the 60 bridges in the database where the rating factor is less than 1.0. These represent bridges throughout Wyoming where, if these bridges were on the I-80 corridor, the reliability against exceeding the Service III limit state does not meet the expectation of the AASHTO LRFD design requirements. This indicates that these bridges should be expected to have more cracking than the AASHTO LRFD Specifications allow at the Service III limit state.

Figure 16 repeats Figure 14, but using the current Service III live load factor of 0.80. The results indicate that, for Wyoming truck traffic, continued use of the 0.80 live load factor results in bridges that do not meet expectations in terms of the Service III limit state.

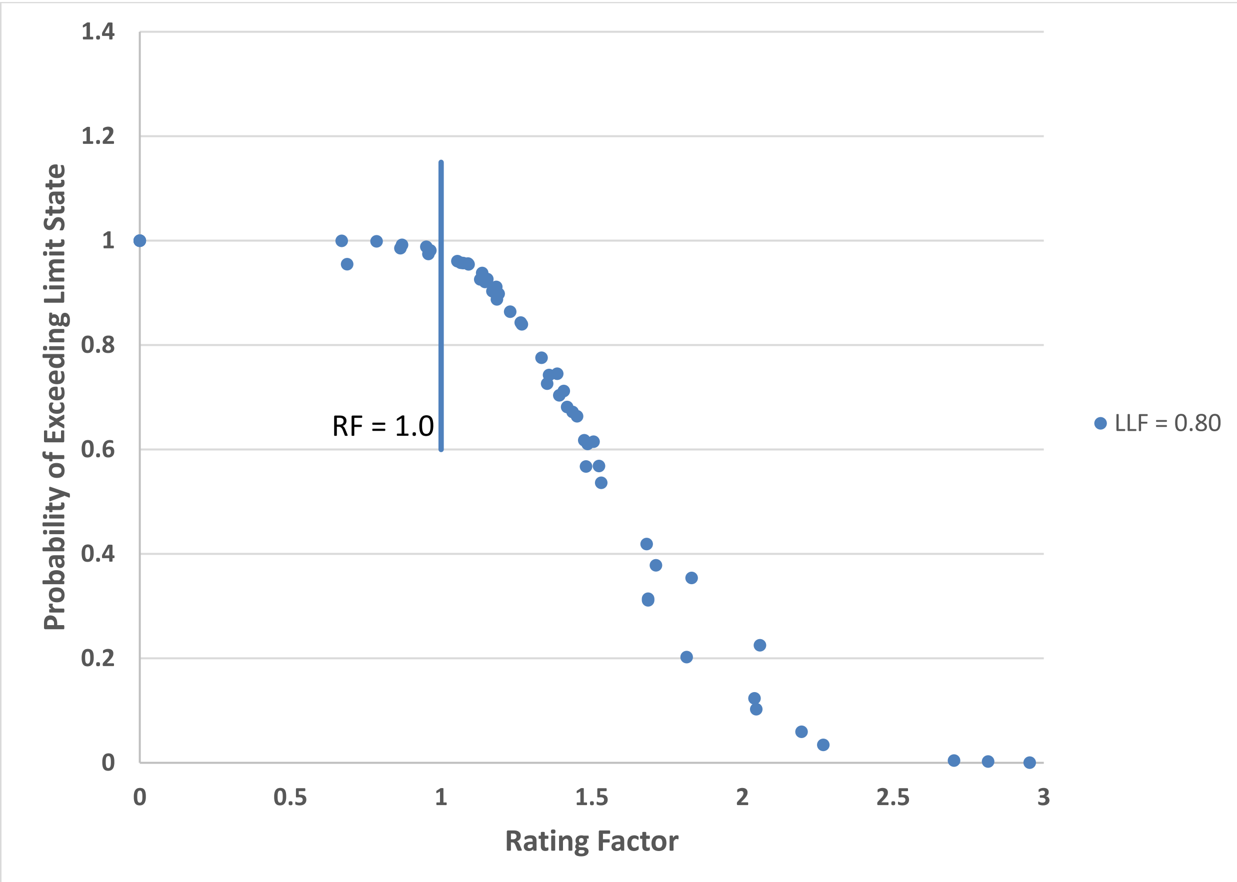

Figure 16. Service III Exceeding Limit State vs Rating Factor for γ_L = 0.80

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

The purpose of this work was to assess the performance (in terms of AASHTO design expectations for serviceability) of bridges along the Interstate 80 corridor for Wyoming's truck traffic. Wyoming's unique truck traffic and traffic patterns create larger demands on bridges than that considered in the development of the AASHTO LRFD Specifications (AASHTO 2017). These characteristics may also be true for other states that contain unique traffic features. The Phase I: Strength I study (Barker and Puckett, 2016 and 2019) recommended increasing the Strength I live load factor from 1.75 to 1.90 or 2.00 (8.6 or 14.3 percent increase, respectively) to maintain adequate reliability for the safety of bridges on I-80. However, as expected, the larger demand also has an impact on the performance and serviceability of bridges on the I-80 corridor. The present work documents these load effects for the Service II limit state for steel bridges and Service III limit state for prestressed concrete bridges.

FINDINGS

This study applied similar reliability studies and calibration to those used in a previous study (Barker and Puckett 2016 and 2019) using a database of in-service Wyoming bridges. This database, consisting of 112 steel bridges and 60 prestressed concrete bridges, was used to determine modified Service II and Service III live load factors to maintain adequate reliability against exceeding serviceability limit states. The results confirm that the current live load factor of γ_L = 1.30 does not meet the serviceability expectations in the AASHTO Service II limit state (structural steel yielding) for Wyoming traffic on I-80. The results also confirm that the current live load factor of γ_L = 0.80 does not meet the serviceability expectations in the AASHTO Service III (prestressed concrete cracking) limit state for Wyoming traffic on I-80.

The study also shows that there are many steel bridges along the I-80 corridor that currently do not meet the Service II limit state. These are older bridges that were designed between the late 1950's and mid-1970's according to earlier specifications. However, it is expected that these in-service bridges may experience yielding and permanent set in excess of that allowed in the AASHTO LRFD Specifications. This would also be true if the prestressed concrete bridges used in the study were located on the I-80 corridor.

RECOMMENDATIONS

Based on the I-80 WIM vehicle load characteristics that create load effects for Service II and Service III limit states, the reliability indices do not meet the target reliability in the AASHTO LRFD Specifications. Raising the design live load factors, γ_L, directly and fairly uniformly increases reliability indices. An increase in γ_L for Service II to 1.45 (from 1.30) and an increase in γ_L for Service III to 1.00 (from 0.80) increases all of the reliability indices to more closely match the target reliability indices. These changes represent an 11.5 percent increase for Service II and a 25 percent increase for Service III.

Recommendation – WYDOT increases the Service II live load factor, γ_L, to 1.45.

Recommendation – WYDOT increases the Service III live load factor, γ_L, to 1.00.

FUTURE CONSIDERATIONS

Consistent with increases to the Strength I, Service II, and Service III live load factors to maintain the safety and serviceability, the third design concern for steel bridges is fatigue damage and fracture based on the load characteristics of the Wyoming I-80 WIM data. Note the Specifications recently increased Fatigue I and II load factors to 1.75 (from 1.5) and 0.8 (from 0.75) based upon National Academies of Sciences, Engineering, and Medicine (2014) considering typical traffic. Whether this is sufficient to address Wyoming I-80 loads is an open question. Certainly, these loads could change the behavior from an infinite design life to a finite design life with associated operational concerns and public safety.

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