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Assessment and Evaluations of I-80 Truck Loads and Their Load Effects

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16. Abstract: The research objective is to examine the safety of Wyoming bridges on the I-80 corridor considering the actual truck traffic on the interstate based upon weigh in motion (WIM) data. This was accomplished by performing statistical analyses to determine reliability indices for a set of archetype bridges. This set of bridges include simple-span bridges with lengths between 30 ft and 200 ft (positive moments) and two-span bridges with equal spans lengths of 30 ft to 200 ft (negative moments). Adequate safety, as defined by AASHTO Bridge Specifications, is a reliability index of at least 3.50. Wyoming DOT has several years of truck characteristics that were used to develop a live load model in a manner similar to the NCHRP used to calibrate the LRFD Specifications. The results are the live load bias values and coefficients of variations for the different bridge archetypes that are used to determine the 75-year maximum load statistical properties for the reliability analyses.

Three optimally designed steel bridges from the NCHRP 20-7/186 report with varying ratios of dead, wearing surface, and live loads are used to perform the reliability analyses and assess safety. Truck traffic along I-80 creates more demand than that assumed in the AASHTO LRFD bridge design procedures. The greater demand results in reliability indices that do not meet target safety levels and have reliability indices significantly less than 3.5. Two issues should be addressed: (1) the unacceptably low reliability indices for short multi-span bridges (2) the overall low reliability indices for all span lengths.

The "optional" (low-boy) dual tandem load where there is a tandem in adjacent spans in the AASHTO LRFD commentary significantly increases the negative design live load moments. Using the dual tandem, the reliability indices for the shorter two-span bridges increase to 3.00 and above, placing this bridge type into the range of the reliability indices for the other bridge span lengths. However, indices are below the target. Raising the design live load factor, γ_L, directly and fairly uniformly increases reliability indices. An increase in γ_L to 2.00 (from 1.75) increases almost all of the reliability indices above 3.50.

In summary, the "optional" low boy load should be used for design and will control shorter multiple-span bridges in the negative moment region. Alternatives to address this situation are outlined.

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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
L – live load effect
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
S – S girder spacing
WYDOT – Wyoming Department of Transportation

EXECUTIVE SUMMARY

The research objective is to examine the safety of Wyoming bridges on the I-80 corridor considering the actual truck traffic on the interstate based upon weigh in motion (WIM) data. This was accomplished by performing statistical analyses to determine reliability indices for a set of archetype bridges. This set of bridges include simple-span bridges with lengths between 30 ft and 200 ft (positive moments) and two-span bridges with equal spans lengths of 30 ft to 200 ft (negative moments). Adequate safety, as defined by AASHTO Bridge Specifications, is a reliability index of at least 3.50.

Wyoming DOT has several years of truck characteristics that were used to develop a live load model in a manner similar to the NCHRP used to calibrate the LRFD Specifications. However, projections for maximum single vehicle effects assume a normal distribution (different than the NCHRP work) and rational load combinations are developed that consider the traffic patterns in Wyoming. The results are the live load bias values and coefficients of variations for the different bridge archetypes that are used to determine the 75-year maximum load statistical properties for the reliability analyses.

Three example steel bridges from the NCHRP 20-7/186 report with varying ratios of dead, wearing surface, and live loads are used to perform the reliability analyses and assess safety. However, instead of using the nominal resistance of the actual example bridges (bridges are usually over-designed), the nominal resistance is determined assuming the bridges are optimally designed (performance ratio = 1). The performance ratio is a state where the design load effect is equal to the resistance for the strength limit state, which yields a reliability index that represents the design equation.

Truck traffic along I-80 creates more demand than that assumed in the AASHTO LRFD bridge design procedures. The greater demand results in reliability indices that do <u>not</u> meet target safety levels. For shorter two-span bridges, Wyoming's truck traffic creates significantly more demand on bridges and results in unacceptably low reliability indices.

The target reliability index is $\beta = 3.50$. For the medium and longer span bridges, the analyses show the minimum β for positive moments is approximately 3.25 and for the negative moments it is just below 3.0. For shorter spans in positive moment, the minimum reliability index is approximately 3.0. However, for shorter two-span bridges in negative moment, the reliability index approaches 2.00. This is an important finding as many truck have axles spacing spectra that match up with the span-lengths of two-span bridges and consequently match up with peak influences for negative moment.

Two issues should be addressed: (1) the unacceptably low reliability indices for short multi-span bridges (2) the overall low reliability indices for all span lengths.

The "optional" (low-boy) dual tandem load where there is a tandem in adjacent spans in the AASHTO LRFD commentary significantly increases the negative design live load moments. Using the dual tandem, the reliability indices for the shorter two-span bridges increase to 3.00 and above, placing this bridge type into the range of the reliability indices for the other bridge span lengths. However, indices are below the target. Raising the design live load factor, γ_L, directly and fairly uniformly increases reliability indices. An increase in γ_L to 2.00 (from 1.75) increases almost all of the reliability indices above 3.50.

An alternative to a live load factor increase to 2.00 is to consider a different method for the statistical properties of the live load model. The AASHTO LRFD Bridge Specifications were developed under

NCHRP projects that used truck database raw data upper tail statistical procedures to estimate maximum truck load effects. When these procedures were applied to the Wyoming 1000-truck database, the required increase of the live load factor is 1.90, smaller than the 2.00 noted above.

In summary, the “optional” low boy load should be used for design and will control shorter multiple-span bridges in the negative moment region. Alternatives to address this situation are outlined.

CHAPTER 1: INTRODUCTION

The purpose of this section is to assess the safety (in terms of AASHTO design expectations) of bridges along the Interstate 80 (I-80) corridor for Wyoming's truck traffic. The concern is that the truck traffic in Wyoming may produce demands on bridges that exceed those that were considered in the development of the AASHTO LRFD Bridge Specifications (AASHTO 2014). Wyoming's I-80 carries a large volume of cross-continental trucks and large energy industry trucks compared to many other states. Another concern is that I-80 is often closed during severe weather and truck traffic tends to gather in large groups waiting for the road to open. When the road opens, this large group of trucks travels as a tight convoy (See Figure 1).

Figure 1. Typical Truck Spacing after Roadway Closure I-80

NCHRP Report 368 and the follow-up NCHRP Project 20-7/186 Report developed the loading model and Strength I safety equations for AASHTO LRFD bridge design. Researchers used a sample of truck traffic from an Ontario truck survey to model the live load effects. The sample included 9250 trucks that represent approximately two weeks of truck traffic. The probabilistic characteristics of the truck sample were projected to estimate the statistical properties for maximum predicted load effects over a 75-year design life. For the 75-year design life, researchers considered single vehicle, multi-presence of vehicles in the same lane, and multi-presence of vehicles in adjacent lanes as load combinations that may produce the largest load effect. Applying the live load model in a reliability study with the statistical characteristics of lateral distribution factors, dynamic impact of trucks, dead load, and strength resistance for a suite of bridges, the NCHRP reports 368 and 186 (hereafter NCHRP) calibrated dead and live load factors and impact factors to obtain a consistent reliability index (level of safety) of $\beta = 3.5$ over a wide range of design variables.

NCHRP Live Load Model

For the original reliability study, the NCHRP work needed to find the 75-year design life statistical properties of the maximum live load effect. For bending moments, this includes the mean of the live load, μ_{LL}, and the coefficient of variation of the live load, COV_{LL}. The NCHRP defined the mean as the 75-year design life live load bias, λ_L, times the nominal AASHTO HL93 design load moment, L_n:

$$\mu_{LL} = \lambda_L L_n \tag{1}$$

The NCHRP researchers applied the survey trucks to a suite of bridges to determine the maximum moment produced by each truck. These were normalized by L_n to determine the bias for each individual truck. The mean and coefficient of variation of the bias set represents the statistical characteristics of the individual truck data for a maximum moment over a two-week period (9250 trucks in dataset is two weeks of ADTT). For instance, the maximum average two-week moment from a single truck is 90 percent (λ_{2week}=0.90) of the AASHTO HL93 nominal design load moment.

The NCHRP work projected the maximum two-week moment to other time frames using the tail end of the raw data and probability of occurrence of the maximum using cumulative density functions. The NCHRP method is described (and considered) in more detail in the present work. The 1-day, 2-week, 1-month, 2-months,-6 months, 1-year, 5-years, 50-years and 75-years maximum average moments ($\lambda_{1day}L_n$ – $\lambda_{75year}L_n$) for a single truck were determined for use in load combinations (multi-presence).

The NCHRP developed a rational set of load combinations to consider multi-presence of vehicles over a 75-year design life on a bridge from their perceived knowledge of traffic that was considered in the research. This entails combining various maximum vehicles either in the same lane or in adjacent lanes and determining the equivalent 75-year design life live load bias λ_L. For instance, considering two of the 75-year maximum truck in the same lane is too conservative. The NCHRP considered the following combinations to determine the 75-year design life maximum average demand for:

Same-Lane Multi-Presence (Following Truck), the:

a. Single maximum 75-year truck
b. Maximum 1-year truck followed by an average truck
c. Maximum 6-month truck followed by a maximum 1-day truck
d. Maximum 1-month truck followed by a maximum 1-month truck

Adjacent-Lane Multi-Presence (Two-Lane) the:

a. Single maximum 75-year truck
b. Maximum 5-year truck followed by an average truck
c. Maximum 6-month truck followed by a maximum 1-day truck
d. Maximum 2-month truck followed by a maximum 2-month truck

Additional permutations considered headway between trucks and whether dynamic impact is included as a function of headway (it is not for small headway as small headway would require lower speeds).

From the load combinations analyses, the maximum live load bias, λ_L, that represents the average maximum force effect through $\mu_{LL} = \lambda_L L_n$, was used in the reliability analyses for calibration of the AASHTO LRFD Bridge Specifications. The COV_L was estimated based on the coefficients of variation of the live load, impact, and lateral girder distribution factors.

The NCHRP work is described and used herein to establish the procedures to assess the Wyoming truck data and load combinations. The particular maximum projection method differs, and the load combinations differ to better represent traffic patterns in Wyoming, but the idea that the 75-year design life live load bias can be determined and used to represent the statistical characteristics of the live load is incorporated. The assessment of the I-80 loads is consistent with the calibration processes used to establish the LRFD design loads when the specifications were created.

NCHRP Reliability Study and Strength I Design Equation

An objective of the NCHRP work was to develop an LRFD Strength I design equation that leads to a target level of safety (reliability index $\beta = 3.5$). The equation for an optimized bridge has the form:

$$\varphi R_n = \gamma_{nc} D_{n_{nc}} + \gamma_w D_{n_w} + \gamma_L L_n (GDF)(1+I) \tag{2}$$

where ϕ is the resistance factor, γ_{nc} is the dead load factor, γ_w is the wearing surface load factor, γ_L is the live load factor. D_{nnc} is the nominal dead load, D_{nw} is the nominal wearing surface load, L_n is the nominal live load from the HL93 design loading, GDF is the lateral distribution factor, and I is the dynamic allowance impact factor.

Through reliability analyses, the NCHRP work concluded that the factors shown below results in reliability indices close to $\beta = 3.5$ over a wide range of design variability:

$$R_n = 1.25 D_{n_{nc}} + 1.50 D_{n_w} + 1.75 L_n (GDF)(1+0.33) \tag{3}$$

To determine the reliability index for a bridge design, the statistical properties of the random variables are set in a limit state equation:

$$Z = R - D_{nc} - D_w - LL = R - D_{nc} - D_w - L(GDF)(1+I) \tag{4}$$

where the R, D, and LL are random variables representing the strength (resistance), dead load, and live load, respectively. The live load LL is the product of the truck load, L, the girder distribution factor, GDF, and the impact, (1+I). Failure is defined as when $Z < 0$ (strength < combined loading). The reliability index is the number of standard deviations the mean of Z scales on the safe side of failure. For the reliability analysis, the mean, μ, and coefficient of variation, COV, for each variable must be determined. For steel bridges, the NCHRP work defined the following for the statistical characteristics:

R – Lognormal Distribution

μ_R	$= \lambda_R R_n$	Mean
λ_R	$= 1.12$	Bias
R_n	=	Nominal Resistance (Design Strength)
COV_R	$= 0.10$	Coefficient of Variation

D_{nc} – Normal Distribution

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

D_w – Normal Distribution

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

$LL = L(GDF)(1+I)$

μ_{LL}	$= \mu_L \mu_{GDF}(1+\mu_I)$	Mean
COV_{LL}	$= 0.18$	Coefficient of Variation

L – Normal Distribution

μ_L	$= \lambda_L L_n$	Mean
λ_L	$=$	Bias Determined by Live Load Model Above
L_n	$=$	HL93 Nominal Live Load
COV_L	$= 0.18$	Coefficient of Variation

GDF – Normal Distribution

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

I – Normal Distribution

μ_I	$= 0.10$	Mean
COV_I	$= 0.80$	Coefficient of Variation

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 a suite of bridge designs.

These statistical properties are shown here for use in the reliability analysis performed for the Wyoming truck data and load combinations in this report. However, this work uses optimized bridge designs (a better assessment of true reliability) instead of a suite of existing bridges and this work uses a different formulation for the coefficient of variation of the live load. Existing bridges include any effects of design methodology, designer conservatism, etc., that vary with agencies and designer, and the date/era in which the bridge was built.

Summary

The research objective is to examine the safety of Wyoming bridges on the I-80 corridor considering the actual truck traffic on the interstate based upon weigh in motion (WIM) data. This was accomplished by performing statistical analyses to determine reliability indices for a set of archetype bridges. This set of bridges include simple-span bridges with lengths between 30 ft and 200 ft (positive moments) and two-span bridges with equal spans lengths of 30 ft to 200 ft (negative moments). Adequate safety, as defined by AASHTO Bridge Specifications, is a reliability index of at least 3.50.

Wyoming DOT has several years of truck characteristics that were used to develop a live load model in a manner similar to the NCHRP work. However, projections for maximum single vehicle effects assume a normal distribution (different than the NCHRP work) and rational load combinations are developed that consider the traffic patterns in Wyoming. The results are the live load bias values and coefficients of variations for the different bridge archetypes that are used to determine the 75-year maximum load statistical properties for the reliability analyses.

Three example steel bridges from the NCHRP 20-7/186 report with varying ratios of dead, wearing surface, and live loads are used to perform the reliability analyses and assess safety. However, instead of using the nominal resistance of the actual example bridges (bridges are usually over-designed), the nominal resistance is determined assuming the bridges are optimally designed (performance ratio = 1). The performance ratio is a state where the design load effect is equal to the resistance for the strength limit state. This will yield a reliability index that represents the design equation. The NCHRP work statistical properties for the dead load, wearing surface, lateral distribution factor, and dynamic allowance impact are used with the statistical properties for live load developed in this work to determine reliability indices.

The results show that truck traffic and traffic patterns in Wyoming create a higher demand on bridge structures than that determined in the NCHRP work. This demand is demonstrated through reliability indices over the range of span lengths that are less than the target of 3.50. This important finding is especially true for shorter multi-span bridges (reliability indices significantly less than 3.50) where multiple closely spaced heavy axles that cross Wyoming are <u>not</u> well represented by the AASHTO HL93 design load.

Recommendations are provided to raise the reliability indices for design of bridges along the I-80 corridor in Wyoming. First, in order to adjust the indices for shorter multi-span span bridges, the AASHTO Commentary dual tandem load should be incorporated into the standard HL93 loading. The dual tandem better represents the high negative moments produced in short multi-span bridges. The second is that, to meet the target reliability index of 3.50, the live load factor, γ_L, should be increased. Increasing γ_L more-or-less uniformly increases reliability indices over the range of span lengths. If the normal distribution of truck traffic method is used for the truck projections, it is recommended that the live load factor be increased from 1.75 to 2.00. However, if the NCHRP 368 method of upper-tail projection is used, the live load bias values are decreased and, thus, the maximum load effect decreases, the reliability indices increase and the recommended increase of the live load factor is from 1.75 to 1.90.

CHAPTER 2: WYOMING TRUCK TRAFFIC LIVE LOAD MODEL

The reliability analyses require the statistical properties of the maximum moment load effects (live load model) for a 75-year design life. This includes the mean of the maximum moment and the coefficient variation. The mean is defined by

$$\mu_{LL} = \lambda_L L_n \quad (5)$$

where λ_L is the 75-year design life live load bias and L_n is the nominal HL93 design moment. This section develops the live load model, determining λ_L and COV_L, for Wyoming's truck traffic.

Wyoming DOT has collected size, weight and axle spacing of trucks on I-80 over several years. This research had access to the truck configuration data from the Pine Bluffs and Evanston weigh stations for use in this work. Table 1 shows the Pine Bluffs weigh station gross vehicle weight (GVW) data for the 2014 year and for the totals over nine years.

Table 1. Pine Bluffs Weigh in Motion Gross Weight Truck Data

GVW Range (kips)	2014	9 Years Total
0-0	0	0
0-10	0	0
10-20	12555	152599
20-30	33070	331037
30-40	71810	816322
40-50	94306	1254498
50-60	134951	1855343
60-70	198198	2560832
70-80	251274	3720862
80-90	20901	639095
90-100	3220	51644
100-110	757	10916
110-120	423	4411
120-130	205	2166
130-140	109	1246
140-150	57	829
150-160	39	598
160-and up	81	1062
Totals	821956	11403460

To develop the truck data set to use for the live load model, a representative one-year set of maximum trucks was desired. The one-year time frame allows for maximum force effects projections less than one year and out to 75-years effectively. However, the year chosen should also represent the longer term multi-year characteristics. Figure 2 shows the probability density functions for the GVW distributions for the year 2014 and the total of nine years of data at the Pine Bluffs weigh station. The statistical properties of the two are similar and the 2014 truck data were selected for the development of the live load model.

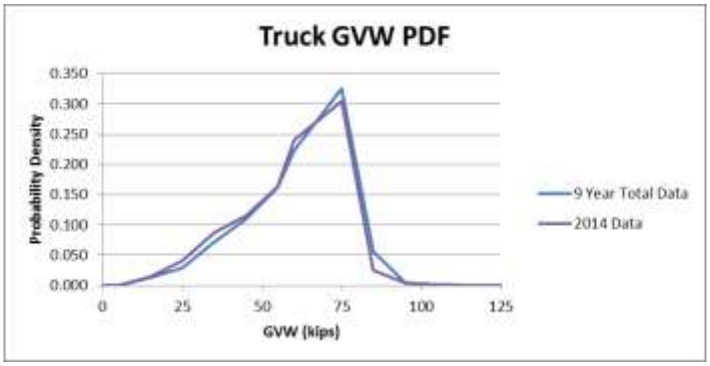

Figure 2. GVW of Pine Bluffs Weigh in Motion Station for 2014 and all years

The 2014 truck data includes over 800,000 vehicles. From these vehicles, the maximum 1000 GVW vehicles in the different classifications of vehicles (number of axles, length, etc.) were selection to produce the truck database. From the WYDOT Truck Sizes, Weights and Permits Gold Book Table 2 (WYDOT 2013), the largest legal GVW on I-80 is 117 kips for trucks with 8 or more axles. The truck database excluded any vehicles that exceeded 125 kips. It was assumed that any vehicle exceeding 125 kips was a special permit vehicle and would not routinely travel with normal traffic. Table 2 shows the 1000 trucks in the truck database and presents the characteristics in relation to number of axles. The legal load is the maximum shown in the WYDOT Gold Book for each number of axles cases. Actual legal loads may be less, depending on axle spacing.

Table 2. Truck Database Properties

Number of Axles	Number in Database	Average Length (ft)	Average GVW (kips)	Legal Load (kips)	Number Exceeding Legal	Percent Exceeding Legal
2	1	16.1	37.04	40	0	0.0%
3	35	20.8	62.4	60	23	65.7%
4	8	37.6	77.1	80	3	37.5%
5	325	59.7	97.6	100	173	53.2%
6	168	65.6	108.1	111	63	37.5%
7	296	70.9	114.7	115.5	125	42.2%
8	135	81.1	115.9	117	55	40.7%
9 or more	32	99.8	116.4	117	14	43.8%

The truck database was used to produce maximum positive and negative moments on a set of bridge archetypes to determine the bias factor (λ=M_{Truck}/L_n) for each vehicle on each bridge. An impact factor of 0.33 is applied to both the trucks in the database and the HL93 loading according to AASHTO LRFD. The HL93 loading does not apply impact on the lane load portion. Thus, the bias λ is correct for the live load model when $\mu_{LL} = \lambda_L L_n$.

The bridge archetypes shown in Table 3 include simple-span and two-span bridges of various lengths. The simple-span bridges are used to develop the positive moment live load model and the two-span bridges are used to develop the negative moment model.

Table 3. Bridge Archetypes for Live Load Model

Positive Moment	Negative Moment
Simple Span	Two-Span
30 ft	30 ft - 30 ft
50 ft	50 ft - 50 ft
100 ft	100 ft - 100 ft
150 ft	150 ft - 150 ft
200 ft	200 ft - 200 ft

Positive Moment Live Load Model

Single Truck on Simple Span

The truck database is applied to the bridge archetypes to determine the statistical properties of the bias $\lambda = M_{Truck}/L_n$ (mean and coefficient of variation) for the 1000 trucks for each archetype. This procedure is demonstrated here using the 150-ft simple-span bridge. As an example, Truck Record No. 1 in the database is a seven-axle, 60-ft long truck with a GVW = 96.12 kips. The maximum moment produced by this truck on the 150-ft span is 47543 in-k and the HL93 loading produces a maximum moment of 60103 in-k. For this truck, λ = 47543/60103 = 0.791. The Truck Record No 1 truck produces a moment that is 79.1 percent of the AASHTO HL93 design moment.

The 1000 trucks in the database creates a distribution of λ for the 150-ft simple span where the mean μ = 0.755 and the coefficient of variation COV = 0.121. Figure 3 shows the probability density function of the raw λ data and a normal distribution using the mean and COV. Figure 4 and Figure 5 show the cumulative density function and Figure 5 is plotted on probability "paper".

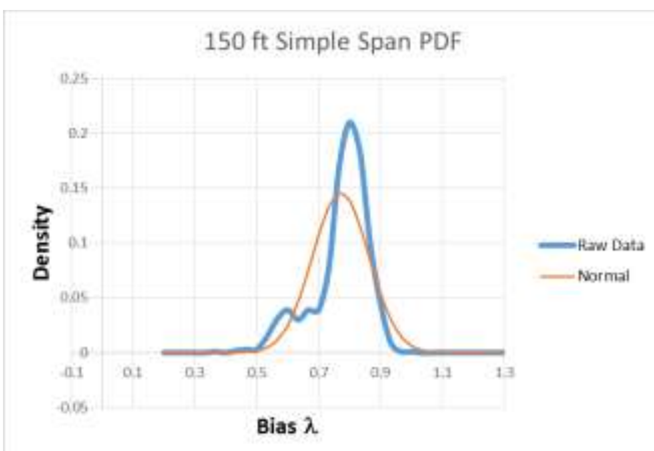

Figure 3. Probability Density Function for Bias λ

Figure 4. Cumulative Density Function Bias λ

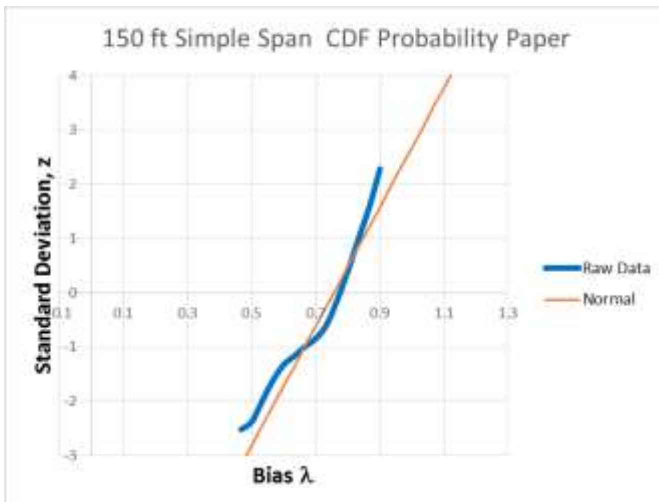

Figure 5. Cumulative Density Function for Bias λ on Probability Paper

Figure 3-4 show that the raw data is not exactly normally distributed. In Figure 5, the CDF on probability paper, a normal distribution is a straight line with slope of the inverse of the standard deviation ($1/\sigma$) and x intercept of the average value (μ). The raw data is fairly close to straight, but does trend to the left at the upper tail. This would result in slightly lower bias values when the bias is projected for the 75-year design life as discussed previously in the discussion on the NCHRP work. Assuming a normal distribution would result in slightly higher bias values when the bias is projected. This report uses a normal distribution for the projections which is conservative. The upper tail NCHRP method is addressed later.

Maximum Single Vehicle Moments (Maximum Bias λ) for Time Frames

Finding the statistical properties (μ and COV) for the maximum live load bias in a one-year time frame involves projecting the truck database properties (the data is for 2014). Knowing the maximum bias in 2014, the maximum moment in that year is determined to be $M_{1yr\text{-}max} = \lambda_{1yr} L_n$.

Monte Carlo simulation is used to find λ_{1yr} assuming a normal distribution (μ = 0.755 and COV = 0.121 for the 150-ft simple span). In one year, there will be 1000 truck crossings representing the heavy and non-permit trucks. Therefore, for one Monte Carlo trial, 1000 simulations are performed to find the maximum out of that 1000 simulations. This is one data point for the maximum one-year bias. The trials are repeated 100 times to produce 100 data points for the one-year the maximum bias. These 100 one-year data points are a distribution for the maximum bias in a one-year time frame, denoted by $\mu_{\lambda 1yr}$ and $COV_{\lambda 1yr}$ are the mean and coefficient of variation, respectively. For the 150-ft simple span, $\mu_{\lambda 1yr}$ = 1.052 and $COV_{\lambda 1yr}$ = 0.03. This finding means that the expected maximum moment on a 150-ft simple span in a one year time frame is 5.2 percent larger than the HL93 loading.

The process is repeated for various time frames of interest. For instance, for a one-month maximum bias, requires 1000/12 = 83 Monte Carlo simulations per trial and the statistical properties of the maximum one-month bias would be based on 100 trials. For the maximum bias over a 75-year design life, a 100 trials of 75000 (1000*75) Monte Carlo simulations were computed. Table 4 presents the 150-ft simple-span statistical properties for the live load bias for various time frames that are of interest.

Table 4. Single Vehicle Live Load Bias for 150-ft Simple Span

Time Frame	Bias Mean	Bias COV
Average	0.755	0.121
1 Day Max	0.820	0.081
2 Week Max	0.947	0.045
1 Month Max	0.979	0.044
2 Month Max	1.001	0.038
6 Month Max	1.032	0.033
1 Year Max	1.052	0.030
5 Year Max	1.092	0.025
50 Year Max	1.141	0.023
75 Year Max	1.149	0.021

As the time frame increases, the maximum expected moment on the bridge increases. For example, a higher maximum moment would be expected over the next 75 years than the maximum expected moment over the next one day. Also, the COV decreases as the time frame increases. Figure 6 illustrates these traits as the various time frame statistical properties are plotted as probability density functions.

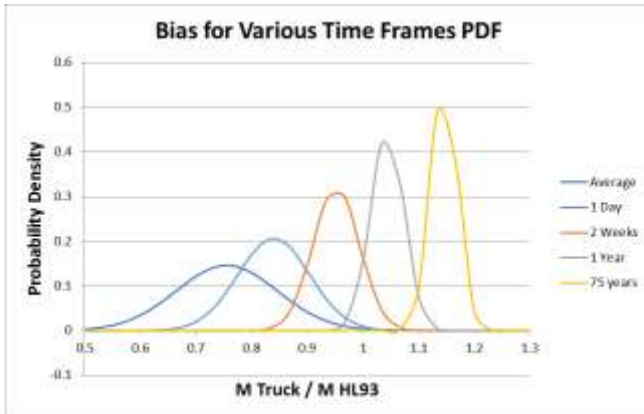

Figure 6. Bias PDF for Various Time Frames

The single-vehicle statistical characteristics of the maximum bias (moments) for various time frames is used in multi-presence load combinations to determine the 75-year design life live load bias, λ_L. This value is important as it is used in the reliability analyses to assess safety. If safety is not met, i.e., $\beta = 3.5$ or higher for the strength limit state, then the live load factor should increase, the load model changed, or both.

75-year Design Life Maximum Moment

To determine the statistical properties for maximum moment over a 75-year design life, load combinations that include multi-presence of vehicles on the bridge must be considered. This research is being performed because there is a concern that the truck traffic in Wyoming may produce demands on bridges that exceed those that were considered in the development of the AASHTO LRFD Bridge Specifications (AASHTO 2014). Wyoming's I-80 carries a large volume of cross-continental trucks and large energy industry trucks compared to many other states. Another concern is that I-80 is often closed during severe weather and truck traffic tends to gather on I-80 in large groups waiting for the road to open. When the road opens, this large group of trucks travels as a tight convoy over the bridges. Again, see Figure 1.

A rational and reasonable approach was used to develop load combinations that represent Wyoming truck traffic. The load combinations are based on:

1. Max Single Truck.
    a. Max of Two Trucks Side-by-Side.
2. Max of Two Trucks in the Same Lane (positioned for critical load effect).
    a. Normal traffic and space further apart (min headway = 50-ft per AASHTO 2014).
    b. When they are close together during road closures (small headway and impact is ignored).
3. Max "Herd" of four Trucks (In-Lane and Side-by-Side).
    a. Normal traffic and space further apart (min headway = 50-ft per AASHTO 2014).
    b. When they are close together during road closures (small headway headway and impact is ignored).

The 75-year design life live load bias, λ_L, is the maximum of the six load cases. In accordance with past NCHRP work, the road closure with small headway cases assumes there is no dynamic allowance impact. This appears to be reasonable and observed behavior after I-80 reopens due to weather events.

Selection of which time frame maximum bias values to use in each of the load cases is based on assuming:

1. Five percent of the trucks travel in pairs (100 percent correlation between the two trucks),
2. Of those five percent, 1 percent travel in a four-truck herd (100 percent correlation between the four trucks), and
3. I-80 is closed 3.33 percent of the time (12 days per year).

The following defines each of the load cases. The process will be applied to the 150-ft simple-span bridge. A "herd" is a term used to represent a group of four trucks.

Max Single Truck

For a 75-year design life, there are 75 * 1000 Trucks/yr = 75000 trucks from the truck database passing over the bridge. Therefore, the maximum average bias is the 75-year maximum average from Table 4. For the 150-ft simple span:

$$\lambda_L = \lambda_{75yr} = 1.149$$

Max of Two Trucks Side-by-Side

Assuming that five percent of fully correlated trucks travel in pairs, there are 75000(0.05) = 3750 multi-presence events of two trucks traveling side-by-side over the 75-year design life. Given that 1000 trucks/yr exist in the database, this represents 3750/1000 = 3.75 years of trucks. Thus, the maximum 3.75 year bias statistical properties should be used for the load case (probability of 3750 events = 1/3750 which equals the probability of the maximum 3.75 yr truck = 1/(3.75*1000)). This process is fully explained in NCHRP 368 (19xx) using statistical methods. Instead of using the maximum 3.75 yr bias, the maximum 5 yr bias is used in this work. A sensitivity analysis discussed later shows that the difference due to changing assumptions would be small. This is an important characteristic of this reliability analysis.

The live load bias, λ_L, is determined by superimposing the additional moment from the second truck to the full moment of the first truck. This load effect is approximated using historical and simple lateral distribution factors. The single-lane lateral distribution factor is assumed to be girder spacing S/14 and the two-lane distribution factor is assumed to be S/11. the additional moment from the second truck is:

$$\frac{S/_{11}}{S/_{14}} - 1 = 0.2727$$

For this work, it is estimated that the second vehicle adds 27.27 percent additional moment to a single vehicle. Therefore, the 75-year design life live load bias for the load case of two side-by-side maximum five-year trucks is,

$$\lambda_L = 1.2727\lambda_{5yr}$$

For the 150-ft simple span, the single truck five-year bias from Table 4 is 1.092. Therefore:

$$\lambda_L = 1.2727(1.092) = 1.389$$

Max of Two Trucks in the Same Lane – Normal Traffic

Assuming that 5 percent of fully correlated trucks travel in pairs, there are 75000(0.05) = 3750 multi-presence events of two trucks traveling in the same lane over the 75-year design life. From the previous load case which has the same conditions, the maximum five-year bias is used again.

The live load bias, λ_L, is determined by running two maximum five-year trucks over the bridge with a headway of 50-ft. However, there is not a defined five-year truck, only a five-year bias for the single trucks in the truck database. Therefore, the following analysis technique was used to determine the live load bias. For each truck in the database, two of the same trucks with a 50-ft headway were run across the bridge to determine the maximum moment. Dividing the two-truck moment by the single-truck moment determines the increase from the second truck. Averaging all of these increases for all of the trucks in the database gives an average increase (AvgInc) that can be used estimate the 75-year design life live load bias:

$$\lambda_L = (\text{AvgInc})\lambda_{5yr}$$

For the 150-ft simple span, the average increase from the second truck is 1.021 (with a small COV = 0.093) and the single truck five-year bias from Table 4 is 1.092. Therefore:

$$\lambda_L = 1.021(1.092) = 1.115$$

Max of Two Trucks in the Same Lane – Road Closure

When the road closes, trucks bunch up and travel over bridges at slow speeds and small headways when the road opens. The headway is assumed to be 10-ft. In accordance with NCHRP 368, it is assumed that there is no dynamic allowance impact for slow speeds and small headways. Again, according to NCHRP 368, the average impact is 0.10. In the reliability analyses presented later, the average impact of 0.10 is applied uniformly to all examples. Therefore, to account for no impact for the road closure cases, 75-year design life live load bias is divided by 1.10. This computation provides a comparison of the 75-year design life live load bias values from all the load cases is consistent.

Assuming that the road is closed 3.33 percent of the time and 5 percent of the trucks travel in pairs, for a 75-year design life there are 75000(0.0333)(0.05) = 125 multi-presence events of two trucks traveling in the same lane with a headway of 10-ft. Using the previous procedure, the maximum 125/1000 = 0.125 yr = 1.5-month single truck bias should be used. This work uses the maximum two-month bias.

The same procedure of applying an average increase that was used for same-lane trucks in normal traffic is used for the road closure case, except it is divided by 1.10 to remove the dynamic impact:

$$\lambda_L = \frac{(\text{AvgInc})\lambda_{2\text{mo}}}{1.10}$$

However, because the headway is only 10-ft, compared to 50-ft for the normal traffic case, the average increase will be significantly higher than the normal traffic average increase. For the 150-ft simple span, the two-month maximum bias is 1.001 and the road closure average increase for a 10-ft headway is 1.382

$$\lambda_L = \frac{(1.382)1.001}{1.10} = 1.257$$

Max Herd of Four Trucks (In-Lane and Side-by-Side) – Normal Traffic

Using 1 percent of 5 percent of 75000 trucks in 75 years results in 38 same-lane and side-by-side four truck multi-presence events. This loading represents a maximum 1.98-week truck. This work uses a herd of four maximum two-week trucks with two side-by-side followed by two side-by-side trucks trailing at a headway of 50-ft.

The 75-year design life live load bias is determined by combining the previous side-by-side analysis with the average increase procedure used for vehicles in the same lane:

$$\lambda_L = 1.2727(\text{AvgInc})\lambda_{2wk}$$

For the 150-ft simple span, the maximum two-week single vehicle bias is 0.947, the average increase is the same as for normal traffic at 1.021:

$$\lambda_L = 1.2727(1.021)0.947 = 1.230$$

Max Herd of four Trucks (In-Lane and Side-by-Side) – Road Closure

Assuming that the road is closed 3.33 percent of the time, and that 1 percent of 5 percent of trucks travel in herds, there are 1.25 same-lane with a 10-ft headway and side-by-side four truck multi-presence events when the road is closed. Using one event in 75 years, the average truck is used for the herd. Impact is removed as was described for the other road closure case:

$$\lambda_L = \frac{1.2727(\text{AvgInc})\lambda_{\text{Avg}}}{1.10}$$

Because the headway is only 10-ft, the average increase is 1.382. From Table 1, the average single vehicle bias is 0.755. For the 150-ft simple span:

$$\lambda_L = \frac{1.2727(1.382)0.755}{1.10} = 1.207$$

75-year Design Life Live Load Bias λ_L Load Cases

The 75-year design life live load bias for use in the reliability studies is the largest from the following load cases:

1) Single Truck,
   a) Max seven five-year.
2) TruckSide-by-Side Multi-Presence,
   a) Max five-year Truck side-by-side to Max five-year Truck.
3) Same Lane Multi-Presence,
   a) Max five-year Truck followed by Max five-year Truck, Headway = min 50-ft.
   b) Max 2 month Truck followed by Max 2 month Truck, Headway = min 10-ft, no Impact.
4) Herd of Four Trucks Multi-Presence,
   a) Two side-by-side Max 2 week trucks followed by Two side-by-side Max 2 week Trucks, Headway = min 50-ft.
   b) Two side-by-side Average trucks followed by Two side-by-side Average Trucks, Headway = min 10-ft, no Impact.

The controlling 75-year design life live load bias is λ_L = 1.389 for the 150-ft simple-span bridge. The side-by-side Max five-year trucks is the controlling load case. This means that the expected maximum moment over a 75-year design life on a 150-ft simple span is 38.9 percent larger than the nominal AASHTO HL93 loading ($M_{Max} = \lambda_L L_n$).

The load cases and procedures were applied to the other simple spans in the bridge archetypes. Table 5 presents the controlling 75-year design life live load bias values for the five simple-span bridges and the load case that controlled for each.

Table 5. Simple Span 75-year Design Life Live Load Bias

Span (ft)	λ_L	**Load Case**
30	1.497	Two Side-by-Side Max 5 yr Trucks
50	1.365	Two Side-by-Side Max 5 yr Trucks
100	1.334	Two Side-by-Side Max 5 yr Trucks
150	1.389	Two Side-by-Side Max 5 yr Trucks
200	1.382	Two In-Lane Max 2 mo trucks

Sensitivity of Load Case Model

It is thought that the load case definitions are reasonable and rational. They consider the traffic patterns of Wyoming truck traffic with maximum single vehicles, combinations of side-by-side or same lane vehicles, and herds of vehicles. The controlling load case for all but the 200-ft simple span was two side-by-side maximum five-year trucks. The 200-ft span was controlled by a herd of four maximum two-month trucks.

To examine the sensitivity of the choice for the maximum truck to use, Table 6 compares the controlling load case to two other alternatives: the same load case with a one step larger truck and the same load case with a one step smaller truck. A one step larger truck means that, for instance, two side-by-side fifty-year trucks considered instead of two side-by-side five-year trucks. Likewise, it would be two side-by-side one-year trucks considered instead of two side-by-side five-year trucks for a one step smaller truck.

Table 6. Sensitivity of Controlling Load Case

Span (ft)	Controlling Load Case	Controlling	Larger Vehicle		Smaller Vehicle	
		λ_L	Vehicle	λ_L	Vehicle	λ_L
30	Two Side-by-Side Max 5 yr Trucks	1.497	Max 50 yr	1.573 (+5.0%)	Max 1 yr	1.446 (-3.4%)
50	Two Side-by-Side Max 5 yr Trucks	1.365	Max 50 yr	1.426 (+4.4%)	Max 1 yr	1.316 (-3.6%)
100	Two Side-by-Side Max 5 yr Trucks	1.334	Max 50 yr	1.398 (+4.8%)	Max 1 yr	1.286 (-3.6%)
150	Two Side-by-Side Max 5 yr Trucks	1.389	Max 50 yr	1.452 (+4.5%)	Max 1 yr	1.339 (-3.6%)
200	Two In-Lane Max 2 mo trucks	1.382	Max 6 mo	1.423 (+3.0%)	Max 1 mo	1.357 (-1.8%)

Table 6 also shows the percentage increase and decrease for the larger and smaller truck analyses, respectively. The maximum increase if considering a one step larger truck in the load case is 5.0 percent and the largest decrease if considering a one step smaller truck is 3.6 percent. The change in assumptions is rather small which lends confidence in the rational selection of the maximum trucks used in the load cases.

Negative Moment Live Load Model

Single Truck on Two-Span

The negative moment single vehicles statistical properties of the mean and COV for $\lambda=M_{Truck}/L_n$ follows the process that was presented for the positive moment for simple spans. A 150-ft two-span will be used to demonstrate. The 1000 trucks in the database are analyzed to create a distribution of the single-vehicle bias. As an example, Truck Record No. 1 in the database is a 7 axle, 60-ft long truck with a GVW = 96.12 kips. The maximum negative moment produced by this truck on the 150-ft two-span is 22755 in-k and the HL93 loading produces a maximum moment of 48772 in-k. For this truck, λ = 22755/48772 = 0.467. The Truck Record No 1 truck produces a moment that is 46.7 percent of the AASHTO HL93 design moment.

The 1000 trucks in the database creates a distribution of λ for the 150-ft two-span where the mean μ = 0.443 and the coefficient of variation COV = 0.135. Figure 7 shows the probability density function of the raw λ data and a normal distribution using the mean and COV. Figure 8 and Figure 9 show the cumulative density function where Figure 9 is plotted on probability paper.

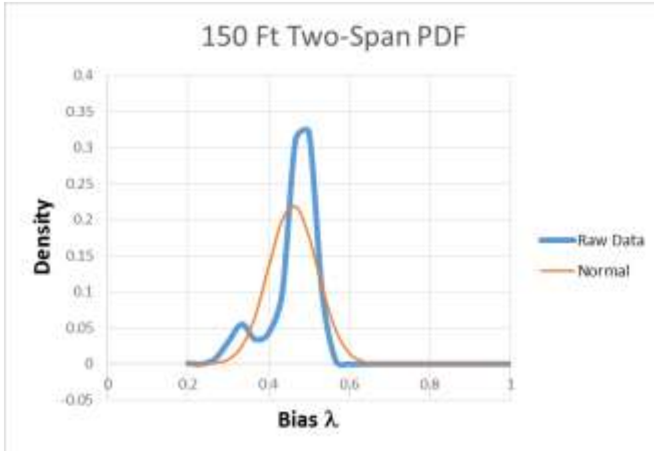

Figure 7. Probability Density Function for Bias λ

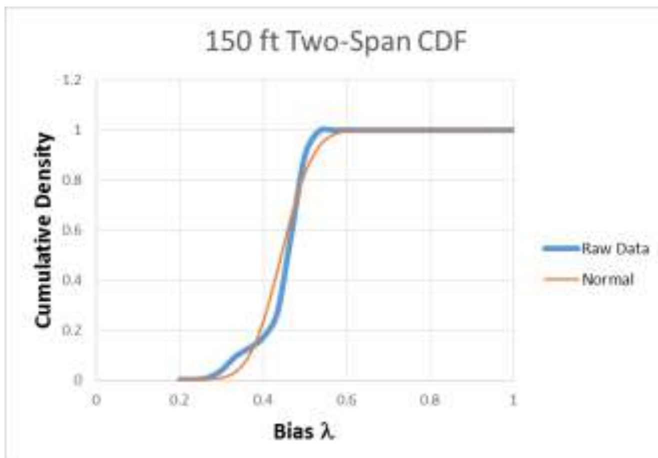

Figure 8. Cumulative Density Function for Bias λ

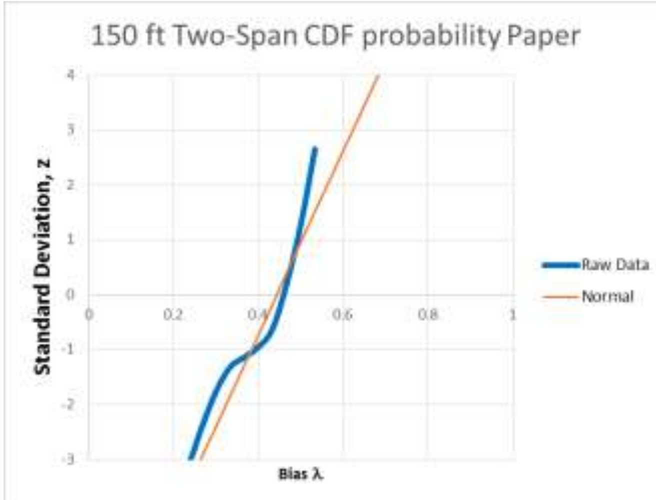

Figure 9. Cumulative Density Function for Bias λ on Probability Paper

As was the case for positive moment, Figure 7, Figure 8, and Figure 9 show that the raw data are approximately normally distributed. In Figure 9, the CDF on probability paper, a normal distribution is a straight line. The raw data are fairly close to straight, but does trend to the left at the upper tail. This characteristic results in slightly lower bias values when the bias is projected for the 75-year design life as discussed earlier in the discussion on the NCHRP work. Assuming a normal distribution would result in slightly higher bias values. This report uses a normal distribution for the projections which is conservative. The upper tail NCHRP method is addressed later.

Maximum Single Vehicle Moments (Maximum Bias λ) for Time Frames

The Monte Carlo process used for the positive moments was again employed to find the statistical properties for the negative moments at the specified time frames as shown in Table 7.

Table 7.Single Vehicle Live Load Bias for 150-ft Two-Span Bridge

Time Frame	Bias Mean	Bias COV
Average	0.443	0.135
1 Day Max	0.484	0.094
2 Week Max	0.571	0.047
1 Month Max	0.588	0.044
2 Month Max	0.603	0.040
6 Month Max	0.626	0.036
1 Year Max	0.639	0.034
5 Year Max	0.664	0.029
50 Year Max	0.695	0.023
75 Year Max	0.701	0.023

75-year Design Life Maximum Moment

The six load cases that were developed for the positive moment analysis are again used for the negative moment:

1. Single Truck,
    a. Max seven five-year Truck.
2. Side-by-Side Multi-Presence.
    a. Max five-year Truck side-by-side to Max five-year Truck,
3. Same Lane Multi-Presence,
    a. Max five-year Truck followed by Max five-year Truck, Headway = min 50-ft.
    b. Max two-month Truck followed by Max two-month Truck, Headway = min 10-ft, no Impact.
4. Herd of four Trucks Multi-Presence,
    a. Two side-by-side Max two-week trucks followed by Two side-by-side Max two-week Trucks, Headway = min 50-ft.
    b. Two side-by-side Average trucks followed by Two side-by-side Average Trucks, Headway = min 10-ft, no Impact.

<u>Max Single seven five-year Truck</u>

The maximum average 75-year design life bias for a single vehicle is:

$$\lambda_L = \lambda_{75yr}$$

For the 150-ft two-span bridge:

$$\lambda_L = 0.701$$

<u>Max Side-by-Side Max 5yr Trucks</u>

The analysis assumes (through lateral distribution factors) that the second truck adds 27.27 percent additional moment to a single vehicle. The 75-year design life live load bias is:

$$\lambda_L = 1.2727\lambda_{5yr}$$

For the 150-ft two-span bridge:

$$\lambda_L = 1.2727(0.664) = 0.845$$

<u>Max of Two Max five-year Trucks in the Same Lane – Normal Traffic</u>

This load case is based on normal traffic flow which means that the headway is a minimum of 50-ft. The 75-year design life live load bias is:

$$\lambda_L = (AvgInc)\lambda_{5yr}$$

where AvgInc is the analytical average increase for a second truck in the same lane for the 1000 truck database. For positive moment, a headway = 50-ft always maximizes the second vehicle increase. However, for negative moment, the analysis considered the maximum increase for each truck for a variable headway where the minimum headway is 50-ft. For the 150-ft two-span bridge, the average increase was 1.996 (with a small COV = 0.002). The large average increase makes sense because it represents another identical vehicle in the opposite span. Therefore, for the 150-ft two-span bridge, the 75-year design live load bias is,

$$\lambda_L = (1.996)0.664 = 1.326$$

<u>Max of Two Max 2 mo Trucks in the Same Lane – Road Closure</u>

This load case considers the condition when the road re-opens and the tightly bunched trucks cross the bridge. It is assumed that speeds are low, there is no impact, and headways may be as low as 10-ft. The live load bias is divided by 1.1 to remove the impact. The analysis considers variable headway with a minimum headway of 10-ft to determine the average increase from the second vehicle. The 75-year design life live load bias is:

$$\lambda_L = \frac{(\text{AvgInc})\lambda_{2\text{mo}}}{1.10}$$

For the 150-ft two-span bridge, the average increase is 1.713 and

$$\lambda_L = \frac{(1.713)603}{1.10} = 0.939$$

<u>Herd of four Max two-week Trucks (In-Lane and Side-by-Side) – Normal Traffic</u>

This considers four trucks travelling side-by-side followed by two side-by-side trucks with a minimum headway of 50-ft. The 75-year design life bias is determined by combining the previous side-by-side and in-lane procedures:

$$\lambda_L = 1.2727(\text{AvgInc})\lambda_{2\text{wk}}$$

The average increase is for the normal traffic in-lane analysis. For the 150-ft two-span bridge, the average increase is 1.996, and

$$\lambda_L = 1.2727(1.996)0.571 = 1.450$$

<u>Herd of four Average Trucks (In-Lane and Side-by-Side) – Road Closure</u>

The load case is again the combination of the road closure in-lane load case (removing impact and minimum headway = 10-ft) and the side-by-side additional moment:

$$\lambda_L = \frac{1.2727(\text{AvgInc})\lambda_{\text{Avg}}}{1.10}$$

For the 150-ft two-span bridge:

$$\lambda_L = \frac{1.2727(1.713)0.443}{1.10} = 0.879$$

75-year Design Life Live Load Bias λ_L Load Cases

The negative controlling 75-year design life live load bias λ_L = 1.450 for the 150-ft two-span bridge. The herd of four Max two week trucks is the controlling load case. This means that the expected maximum moment over a 75-year design life on a 150-ft two-span bridge is 45 percent larger than the nominal AASHTO HL93 loading ($M_{Max} = \lambda_L L_n$).

The load cases and procedures were applied to the other simple spans in the bridge archetypes. Table 8 presents the controlling 75-year design life live load bias values for the five two-span bridges and the load case that controlled for each.

Table 8. Single Vehicle Live Load Bias for 150-ft Two-Span Bridge

Span (ft)	λ_L	Load Case
30	1.558	Two Side-by-Side Max 5 yr Trucks
50	1.920	Two Side-by-Side Max 5 yr Trucks
100	1.386	Herd of Max 2 wk Trucks
150	1.450	Herd of Max 2 wk Trucks
200	1.356	Herd of Max 2 wk Trucks

Summary

This section developed the live load model for the reliability analyses of bridges in Wyoming subject to Wyoming truck traffic. Five simple span (spans 30-ft – 200-ft) bridge archetypes were used to develop statistical properties for the maximum positive moment that occurs over specific time frames (i.e., the maximum moment in a one-year time frame). Likewise, five two-equal-span (spans 30-ft – 200-ft) bridge archetypes were used for the maximum negative moment.

Rational load cases were developed to model the traffic pattern characteristics thought to exist on Interstate 80 across Wyoming. The load cases predict the maximum expected moment over a 75-year design life. These include consideration for the maximum single truck, multi-presence of two trucks in the same lane, multi-presence of two trucks in adjacent lanes, and herds of four trucks traveling as a group. The load cases also consider a situation where a road closure bunches trucks into a tight convoy of vehicles.

The load cases are:

Single Truck
 Max seven five-year Truck
Side-by-Side Multi-Presence
 Max five-year Truck side-by-side to Max five-year Truck
Same Lane Multi-Presence
 Max five-year Truck followed by Max five-year Truck, Headway = min 50-ft
 Max two-month Truck followed by Max two-month Truck, Headway = min 10-ft, no Impact:
Herd of four Trucks Multi-Presence
 Two side-by-side Max two-week trucks followed by Two side-by-side Max two-week Trucks, Headway = min 50-ft.
 Two side-by-side Average trucks followed by Two side-by-side Average Trucks, Headway = min 10-ft, no Impact

The maximum moment expected over a 75-year design life is the maximum live load bias multiplied by the nominal AASHTO HL93 design loading:

$$M_{Max75yr} = \lambda_L L_n$$

Table 9 (Table 5 and Table 8 combined) are the 75-year design life live load bias factors. These values will be used in the reliability analyses as the live load model to determine the mean of the truck live loading. The coefficient of variation for the truck live loading is derived later in the reliability analyses because it is a function of truck loading, lateral distribution factors and dynamic allowance impact.

Table 9. 75-year Design Life Live Load Bias -- Positive and Negative Moments

Span (ft)	Simple Span λ_L	Two-Span λ_L
	Positive Moment	Negative Moment
30	1.497	1.558
50	1.365	1.920
100	1.334	1.386
150	1.389	1.450
200	1.382	1.356

CHAPTER 3: RELIABILITY ANALYSIS OF WYOMING TRUCK TRAFFIC

The AASHTO Strength I Limit State design equation is the safety limit state in LRFD where:

$$R_n > 1.25D_{n_{nc}} + 1.50D_{n_w} + 1.75L_n(GDF)(1 + 0.33) \quad (6)$$

Satisfying the equation implies that the bridge will have an adequate level of safety, represented by a reliability index of 3.50 or better. The NCHRP projects that developed the design equation developed a live load model, similar to that developed herein, with a sample of truck from an Ontario truck survey used in NCHRP. Truck traffic and truck traffic patterns in Wyoming may be more severe than the sample trucks used in the NCHRP work leading to reliability indices below the target 3.50. If the reliability index is below 3.50, the bridge does not meet the safety expectations implied in the AASHTO LRFD Bridge Design Specifications.

The following reliability analyses use the Wyoming truck live load model developed herein with the established dead, wearing surface, lateral distribution, and impact statistical models from the NCHRP work (and shown in Section 1). These are labeled below with (NCHRP).

To determine the reliability index for a bridge design, the statistical properties of the random variables are set in a limit state equation:

$$Z = R - D_{nc} - D_w - LL = R - D_{nc} - D_w - L(GDF)(1 + I) \quad (7)$$

where the R, D, and LL are random variables representing the strength (resistance), dead load, and live load, respectively. The live load LL is the product of the truck load, L, the girder distribution factor, GDF, and the impact, (1+I). Failure is defined as when Z < 0 (strength < combined loading). For the reliability analysis, the mean, μ, and coefficient of variation, COV, for each variable must be determined.

R – Lognormal Distribution

μ_R	$= \lambda_R R_n$	
λ_R	= 1.12	(NCHRP)
R_n	= nominal resistance	
COV_R	= 0.10	(NCHRP)

For the nominal resistance, an optimized design is used to determine the true reliability index:

$$R_n = 1.25D_{n_{nc}} + 1.50D_{n_w} + 1.75L_n(GDF)(1 + 0.33)$$

D_{nc} – Normal Distribution

μ_{Dnc}	$= \lambda_{Dnc} D_{nnc}$	
λ_{Dnc}	= 1.05	(NCHRP)
D_{nnc}	=	Nominal Dead Load
COV_{Dnc}	= 0.10	(NCHRP)

D_w – Normal Distribution

μ_{Dw}	$= \lambda_{Dw} D_{nw}$	
λ_{Dw}	= 1.00	(NCHRP)
D_{nw}	=	Nominal Wearing Surface Load
COV_{Dw}	= 0.25	(NCHRP)

L – Normal Distribution

μ_L	$= \lambda_L L_n$	
λ_L	=	Determined by Live Load Model
L_n	=	HL93 Nominal Live Load
COV_L	=	Determined by Live Load Model

GDF – Normal Distribution

μ_{GDF}	= GDF	(NCHRP)
GDF	= 1.00	Girder Distribution Factor
COV_{GDF}	= 0.12	(NCHRP)

The nominal AASHTO HL93 loading used for L_n includes the GDF factor (already distributed to the girder) for the reliability analyses. Therefore, the GDF, and thus the mean of the GDF, is set to be 1.00. However, the coefficient of variation for the GDF is used to determine the COV for the live load.

I – Normal Distribution

μ_I	= 0.10	(NCHRP)
COV_I	= 0.80	(NCHRP)

LL = L(GDF)(1+I)

μ_{LL}	$= \mu_L \mu_{GDF} (1+\mu_I)$	
COV_{LL}	= 0.14	(NCHRP)

The coefficient of variation for the live load is a function of the lateral distribution factor, impact and the truck load. The live load is:

LL = L(GDF)(1+I) = L(DGF) + L(GDF)I

The coefficient of variation for the live load is:

$$COV_{LL} = \frac{\sigma_{LL}}{\mu_{LL}} = \frac{\sqrt{\sigma^2_{L(GDF)} + \sigma^2_{L(GDF)I}}}{\mu_L \mu_{GDF}(1+\mu_I)} \tag{8}$$

Where:

$$\sigma_{L(GDF)} = \mu_L \mu_{GDF} COV_{L(GDF)} = \mu_L \mu_{GDF} \sqrt{COV_L^2 + COV_{GDF}^2} \tag{9}$$

$$\sigma_{L(GDF)I} = \mu_L \mu_{GDF} \mu_I COV_{L(GDF)I} = \mu_L \mu_{GDF} \mu_I \sqrt{COV_L^2 + COV_{GDF}^2 + COV_I^2} \tag{10}$$

Substituting in $\mu_L = \lambda_L L_n$, $\mu_{GDF} = 1$, $COV_{GDF} = 0.12$ and $COV_I = 0.80$ yields:

$$COV_{LL} = \frac{\sqrt{(1+\mu_I^2)COV_L^2 + (1+\mu_I^2)(0.12)^2 + \mu_I^2(0.80)^2}}{(1+\mu_I)} = \tag{11}$$

The NCHRP reliability work used a live load COV_{LL} = 0.18, which represents a truck COV_L = 0.14. This is considerable large compared to the COV_L values determined in the live load model of the present work. To better represent the statistical characteristics for the reliability analyses performed here, a COV_L = 0.06 is used. Therefore, when the mean impact μ_I = 0.10, the live load COV_{LL} = 0.14. When impact is not applied to the road closure cases, the COV_{LL} would be 0.13, not a significant difference so the reliability analyses used is COV_{LL} = 0.14.

For the reliability analyses, μ_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 so that the live load mean in the reliability analyses did <u>not</u> include impact.

Monte Carlo Simulations for Reliability Indices

To determine the reliability index for the limit state equation, statistical methods are used to predict the probability that the limit state equation is less than 0 (probability that the strength is less than the combined load). 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 a large set of bridges.

For one Monte Carlo trial, the limit state equation Z is determined by simulating the R, D_{nc}, D_w and LL according to their distributions. The definition of failure is if Z is less than zero. For this work, 100,000 trials are used to find how many failures occur, #Fail. The probability of failure is p_f = #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. Z is the reliability index β.

Microsoft Excel's 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_w and LL. However, because R is lognormally distributed, the mean of ln(R) and the standard deviation of ln(R) are required.

From statistics

$$\mu_{lnR} = \ln(\mu_R) - 0.5\sigma_{lnR}^2 \tag{12}$$

$$\sigma_{lnR} = \sqrt{\ln{(1 + COV_R^2}} \tag{13}$$

The Monte Carlo simulation will be demonstrated with the third example from Table 7 in NCHRP 20-7/186 report, *Updating the Calibration Report for AASHTO LRFD Code*. The steel bridge has positive girder nominal design moment:

D_{nnc}	= 8496 ft-k	Nominal Dead Load
D_{nw}	= 1493 ft-k	Nominal Wearing Surface
L_n	= 7120 ft-k	Nominal HL93 (Includes GDF and Impact = 1.33)
Rn	= 26585 ft-k	Nominal Strength

The AASHTO Strength I design criteria shows that this bridge meets design expectations

$$R_n \geq 1.25D_{n_{nc}} + 1.50D_{n_w} + 1.75L_n(GDF)(1 + 0.33)$$

$$26585 \geq 1.25(8496) + 1.50(1493) + 1.75(7120)(1.0)(1 + 0.33) = 25320\ ft - k$$

The NCHRP Report used a live load bias $\lambda_L = 1.18$, a $COV_{LL} = 0.18$ and the actual nominal resistance $R_n = 26585$ ft-k (not optimized). A significant problem with the NCHRP example is that the HL93 loading L_n uses the 0.33 design impact as the mean of the impact for reliability. The correct mean for impact is 0.10. Thus, the live load is increased significantly leading to lower reliability indices than it should. The NCHRP work found 26 failures in 100,000 simulations resulting in a probability of failure $p_f = 0.00026$ (0.026 percent). Using the inverse cumulative function, $\beta = -\Phi^{-1}(p_f) = 3.47$. Figure 10 recreates the NCHRP example using their inputs, but with the Excel file developed for this work. The results are the same with a $\beta = 3.47$. This and other validations were made to confirm the present process.

		Live Load	1.75		Live Load Bias		1.180			
		phi	1							
				Nominal	Bias	**Mean**	**COV**	**Std Dev**		
									μ_{lnR}	μ_{lnR}
LogNormal	R	Resistance		26585	1.12	29775	0.1	2978	10.296	0.100
Normal	D	Dead		8496	1.05	8921	0.1	892		
Normal	Dw	Wearing Surface		1493	1	1493	0.25	373		
Normal	LL=(L+I)GDF	Live		7120	1.180	8402	0.18	1512		
								Number	Percent	
								Fail	Fail	
			29767.00373	8920.092	1492.846	8400.133		26	0.026%	
								BETA =	**3.47**	
	100000	Trials	R	D_{nc}	D_w	(L+I)GDF	Limit State	Fail?		
		1	29345.82	7712.99	2086.13	8874.48	10672.22			
		2	33371.25	8644.57	1578.28	8330.44	14817.96			
		3	31634.68	8638.41	1006.09	7858.72	14131.47			
		4	26460.51	8219.31	1424.11	7445.75	9371.34			
		5	27902.60	9555.90	1277.13	6772.18	10297.39			
		6	32971.54	6801.72	1188.23	7761.86	17219.73			
		7	32584.93	8987.70	1089.98	8256.99	14250.25			
		8	31270.23	8219.79	1970.58	9068.58	12011.27			

Figure 10. NCHRP 20-7/186 Table 7 Example 3 Monte Carlo Simulation for β

To fix the problems with the NCHRP example, two changes are incorporated. First, the optimized design R_n should be used when checking reliability at the design limit state. The optimized nominal strength equals the factored load side of the design equation and $R_n = 25320$ ft-k. The second is that the 0.33 design impact is removed by dividing L_n by 1.33 (and having the Excel apply the mean impact of 0.10). The only other change is to use this work's $COV_{LL} = 0.14$ instead of the NCHRP work value of 0.18.

Repeating the example with the adjusted variables and new COV_{LL}, the failure rate decreases to 0.006 percent resulting in a reliability index $\beta = 3.85$ as shown in Figure 11.

		Live Load	1.75		Live Load Bias		1.180			
		phi	1							
				Nominal	Bias	**Mean**	**COV**	**Std Dev**		
									μ_{lnR}	μ_{lnR}
LogNormal	R	Resistance		25320	1.12	28358	0.1	2836	10.248	0.100
Normal	D	Dead		8496	1.05	8921	0.1	892		
Normal	Dw	Wearing Surface		1493	1	1493	0.25	373		
Normal	LL=(L+I)GDF	Live		5889	1.180	6949	0.14	973		
								Number	Percent	
								Fail	Fail	
			28342.51077	8920.9914	1493.593	6949.189		6	0.006%	
								BETA =	**3.85**	
	100000	Trials	R	D_{nc}	D_w	(L+I)GDF	Limit State	Fail?		
		1	32073.22	8385.22	1269.34	6926.89	15491.77			
		2	31618.09	9263.22	1199.33	7505.04	13650.51			
		3	28500.89	8913.12	1766.72	5975.51	11845.54			
		4	30806.05	10313.07	1927.32	6987.49	11578.16			
		5	31839.89	10403.92	1439.04	4787.34	15209.59			
		6	27664.45	10023.08	1864.04	8154.15	7623.17			
		7	25194.25	8556.54	1951.16	7665.22	7021.33			
		8	28892.43	8318.11	1431.80	7240.36	11902.16			

Figure 11. NCHRP 20-7/186 Table 7 Example 3 with Fixed Variables

Figure 11 demonstrates the reliability analysis of the example bridge using the 75-year design life bias of 1.18 from the NCHRP work. However, the live load bias values of from the Wyoming live load model are significantly higher. This is an important finding of the present work.

Reliability Indices for Wyoming Traffic

Three steel girder example bridges from the NCHRP 20-7/186 report will be used to assess the reliability of bridges subject to the Wyoming live loads. Examples 1 and 3 represent positive moments and Example 2 represents negative moments. Figure 11 presents the nominal moments for the three examples. The table also shows the percent of total nominal load each load variable carries.

Table 10. NCHRP Example Bridges

Bridge	Action	D_{nnc} (ftk)	D_{nw} (ftk)	L_n with I=0.33 (ftk)	Total Nominal (ftk)	L_n I=0.33 removed (ftk)	Optimized R_n (ftk)
Example 1	Positive moment	**9071 (58.0%)**	**1247 (8.0%)**	5322 (34.0%)	15650	**4002**	**22540**
Example 2	Negative Moment	**27017 (62.4%)**	**3529 (8.4%)**	11521 (20.6%)	42067	**8662**	**59227**
Example 3	Positive Moment	**8496 (49.7%)**	**1493 (8.7%)**	7120 (41.6%)	17109	**5353**	**25320**

For an optimized design, only the ratio of total nominal load is important for each load variable. The size of the bridge (magnitudes of load variables) will not change the reliability indices as long as the ratio of loads remains the same. The three examples have varying levels of dead-to-live load ratios that will produce a range of reliability indices. The bold columns are the nominal dead, wearing surface and live (without the design I=0.33) that are used in the Monte Carlo simulations to determine reliability indices.

Table 11 presents the 75-year design live load bias factors for different span lengths.

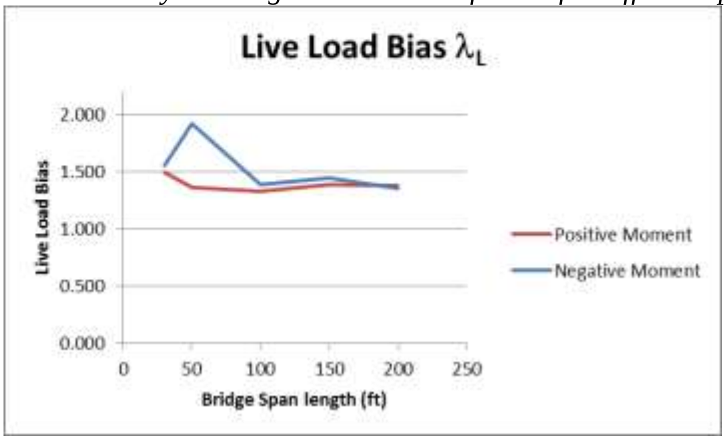

Figure 12 illustrates the live load bias as a function of span.

Table 11. 75-Year Design Life Live Load Bias

Span (ft)	Simple Span λ_L	Two-Span λ_L
	Positive Moment	Negative Moment
30	1.497	1.558
50	1.365	1.920
100	1.334	1.386
150	1.389	1.450
200	1.382	1.356

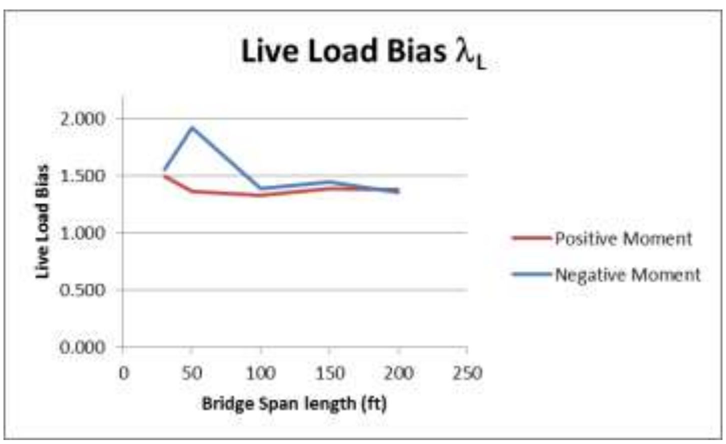

Figure 12. Live Load Bias

The target reliability index is β = 3.50. With significantly higher live load bias, the reliability indices will decrease. Reliability analyses were performed for the live load bias factors in Table 11.

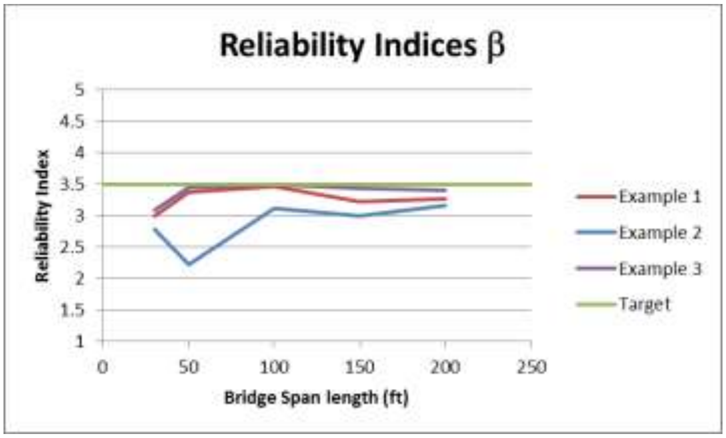

Figure 13. Reliability Indices for Wyoming Live Load Model

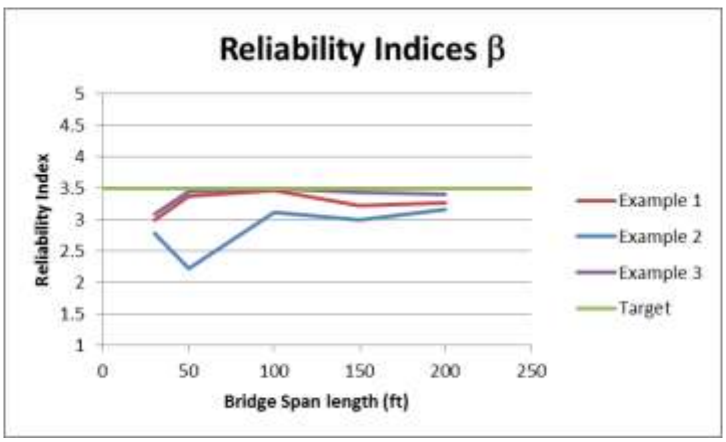

Figure 13 demonstrates that the reliability indices for the Wyoming live load model are significantly below the target reliability. Bridges on I-80 in Wyoming do not meet the safety expectations (target β) implied in the AASHTO LRFD Bridge Design Specifications.

Recommendations to Meet Target Safety Expectations

Bridges along I-80 do not meet the expected safety associated with a target reliability index of 3.50. There are two issues that need to be addressed: (1) the high negative moment bias for short-span bridges, and (2) the overall low reliability indices for all span lengths.

Short Span Negative Moment Recommendation

The heavy trucks in the 1000 truck database are at lengths that straddle the interior support of short multi-span bridges with groups of heavy axles causing large negative moments. The AASHTO HL93 negative moment does not capture that large moment well.

The AASHTO LRFD commentary C.3.6.1.3.1 addresses this issue and states that, where heavier versions of "low boy" type vehicles are probable, a load to consider for negative moments is a pair of tandems placed in adjacent spans combined with the lane load. WYDOT does not currently consider the low-boy tandem loading. Table 12 presents the HL93 moments and the commentary low-boy moments for the negative moment regions.

Table 12. HL93 and Commentary Low-boy Tandem Negative Moments

Span	HL93 (in-k)	Low-Boy (in-k)	Low-Boy Increase
30	3944	5266	33.5%
50	7512	10024	33.4%
100	27767	23386	0.0%
150	48772	43578	0.0%
200	73975	68317	0.0%

For shorter spans, the increased design moment leads to lower live load bias factors and, thus, higher reliability indices. For longer spans, the commentary low-boy tandem moments are not larger than the HL93 moments.

Figure *14* shows that, for the negative moment, the low-boy tandem loading lowers the live load bias factors within the range of bias values for all the other bridge archetypes.

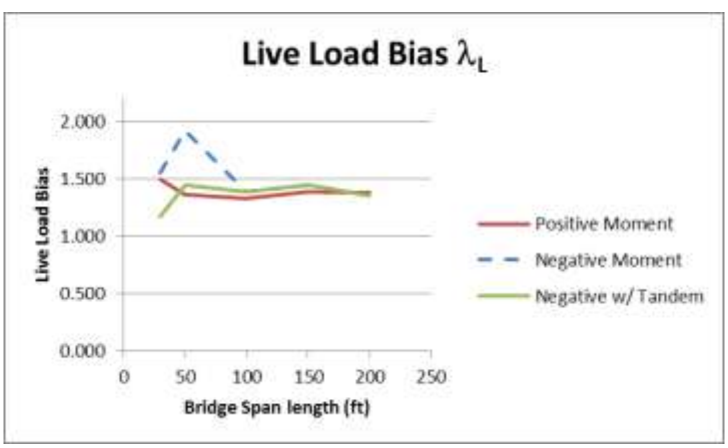

Figure 14. Live Load Bias with Commentary Low-boy Tandem Loading

The first recommendation is that WYDOT incorporate the commentary low-boy tandem loading in the design of bridges.

When examining reliability indices, Figure 15 shows that when Example 2 considers the low-boy tandem loading, the reliability indices for shorter spans are increased to the levels of the other bridge archetypes. However, the overall reliability indices, especially for the negative moments, are lower than the target of $\beta = 3.50$.

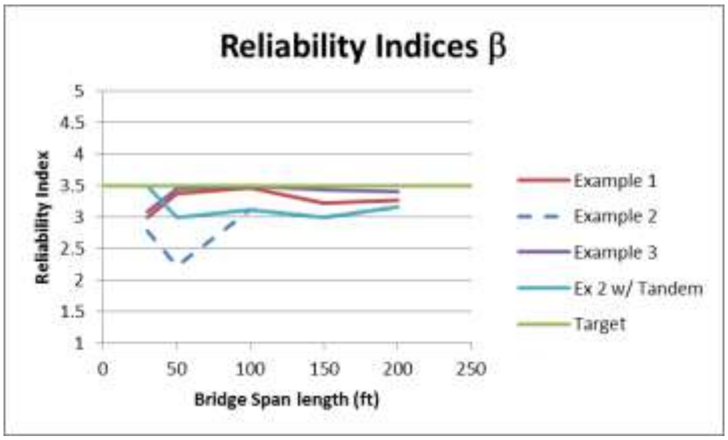

Figure 15. Reliability Indices considering Commentary Low-boy Tandem Loading

Design Equation Live Load Factor Recommendation

The design live load factor, $\gamma_L = 1.75$, is the design equation variable that can directly and increase uniformly reliability indices. An increase in γ_L will increase the nominal required capacity R_n, which will increase β.

AASHTO LRFD addresses this in the commentary C3.6.1.2.1. The commentary states that consideration should be given to site-specific modifications to the HL93 loading if:

- The legal load is significantly greater than typical.
- The roadway carries unusually high percentages of truck traffic.
- Flow control causes trucks to collect.

Interstate I-80, with its energy industry truck traffic, high percentage of trucks and common road closures, meets all of these criteria.

Figure 16 shows the required live load factor, γ_L, to raise the reliability index to the target of 3.50. The negative moments (low-boy tandem considered) in Example 2 requires the largest increase to about γ_L = 2.00 (a 14.3 percent increase over the current 1.75).

Figure 16. Required Live Load factor for Target β = 3.50

If the live load factor is increased to γ_L = 2.00, Figure 17 illustrates the resulting reliability indices. Most of the indices are above the target of 3.50 with only a couple dipping slightly below.

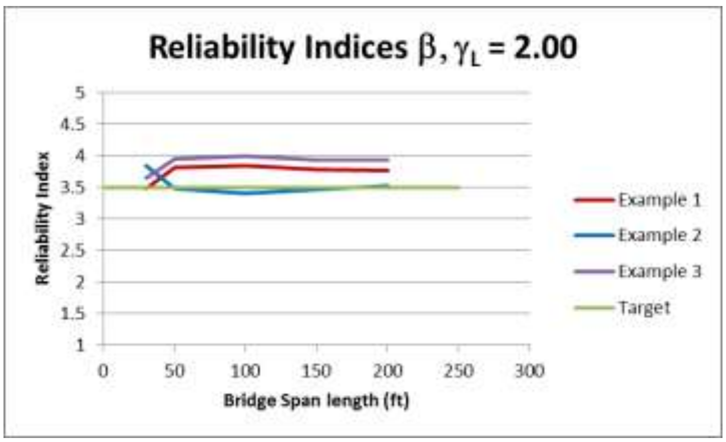

Figure 17. Reliability Indices for Live Load Factor γ_L = 2.00

The second recommendation is, due to the truck traffic and traffic patterns on I-80, to raise the live load factor to γ_L = 2.00.

Consider NCHRP Maximum Average Projection Method

The live load model in this work assumes a normal distribution for single truck moments based on the 1000 truck dataset mean and coefficient of variation. The NCHRP work that developed the AASHTO LRFD bridge specifications modeled only the upper tail of the raw data distribution (Figures 4 and 8).

The cumulative density function method using the upper tail region of the raw data is based on the probability of having the maximum truck during the given time frame, i.e., 1 divided by the total number of trucks. This probability corresponds to the z value of a standard cumulative density function ($z = \Phi^{-1}(1 - 1/\text{Number of trucks})$). The use of probability paper demonstrates the process.

Table12 shows the process for the 30-ft simple span bridge. Using 1000 (heavy) trucks per year is an ADTT of 2.74. To find the maximum average 6-month truck, there are 500 trucks in the 6 months. The probability that the maximum truck is in that 6 months is 1/500 = 0.002. The number of standard deviations, z, that considers that0.002 exceeds the max moment is $z = -\Phi^{-1}(1 - 1/500)) = 2.878$. The probability is 0.002 that, out of the 500 trucks, there will be a truck causing moments greater than the mean plus 2.878 standard deviations. Figure 17 is the cumulative density function of the raw data and the assumed normal distribution used to develop the live load model. The straight line that represents the upper tail of the raw data is determined manually. The bias $\lambda = 1.096$ for the average maximum 60-month vehicle at the horizontal value of the intersection of the raw data and $z = 2.878$. The process can project the average maximum truck for any time frame as shown in Table 12 and Figure 18. However, the method cannot determine the coefficients of variation for the average maximums. The assumed normal distribution is also shown for comparison.

Table 13. Single Vehicle Live Load Bias for 30 ft Simple Span NCHRP Method

Timeframe	Number of Trucks	Probability of Heaviest Truck	$z = \phi^{-1}$(1-Prob)	Bias Mean
Average			0.000	0.802
1 Day	2.74	0.3650	0.345	0.837
2 Weeks	38	0.0261	1.942	1.001
1 Month	83	0.0120	2.257	1.033
2 Months	167	0.0060	2.512	1.059
6 Months	500	0.0020	2.878	1.096
1 Year	1000	0.0010	3.090	1.118
5 Years	5001	0.0002	3.540	1.164
50 Years	50005	2.00E-05	4.108	1.222
75 Years	75008	1.33E-05	4.200	1.232

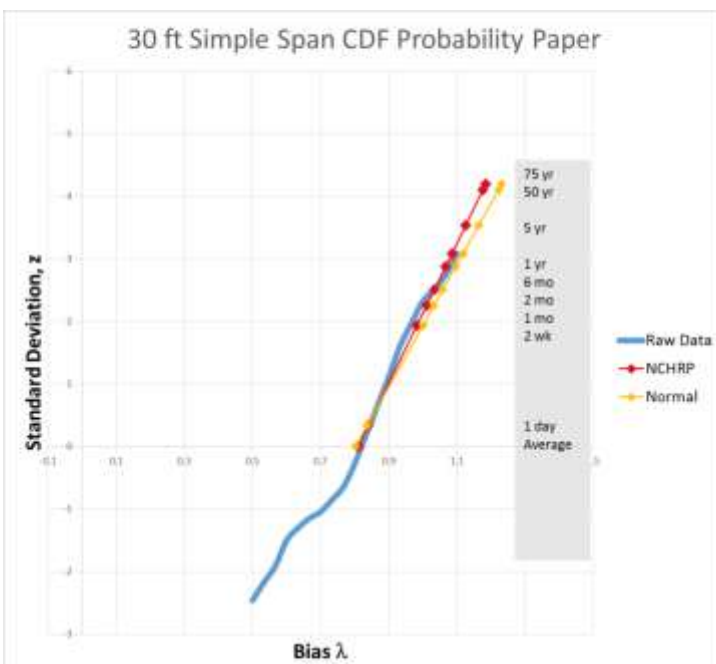

Figure 18. Raw Data and Normal Distribution CDF for 30-ft Simple Span on Probability Paper

For this span, the NCHRP method results in lower bias factors. This, in turn, would result in higher reliability indices. Using the NCHRP method may result in a smaller (or no) increase in the live load factor.

Table 14 directly compares the normal distribution method to the NCHRP method for the 30-ft simple span.

Table 14. Single Vehicle Bias for Normal and NCHRP Methods for 30-ft Simple Span

Time Frame	Normal Method	NCHRP Method
Average	0.802	0.810
1 Day Max	0.887	0.841
2 Week Max	1.030	0.983
1 Month Max	1.060	1.011
2 Month Max	1.076	1.034
6 Month Max	1.114	1.066
1 Year Max	1.136	1.085
5 Year Max	1.176	1.125
50 Year Max	1.236	1.176
75 Year Max	1.249	1.184

For the positive moment spans, the difference is not significant. But, as shown in Table 15 and Figure 19, for the 50-ft two-span negative moments, the difference can be significant.

Table 15. Single Vehicle Live Load Bias for 30-ft Simple Span NCHRP Method

Timeframe	Number of Trucks	Probability of Heaviest Truck	$z = \phi^{-1}(1\text{-Prob})$	Bias Mean
Average			0.000	**0.725**
1 Day	2.74	0.3650	0.345	**0.763**
2 Weeks	38	0.0261	1.942	**0.941**
1 Month	83	0.0120	2.257	**0.976**
2 Months	167	0.0060	2.512	**1.005**
6 Months	500	0.0020	2.878	**1.046**
1 Year	1000	0.0010	3.090	**1.069**
5 Years	5001	0.0002	3.540	**1.119**
50 Years	50005	2.00E-05	4.108	**1.183**
75 Years	75008	1.33E-05	4.200	**1.193**

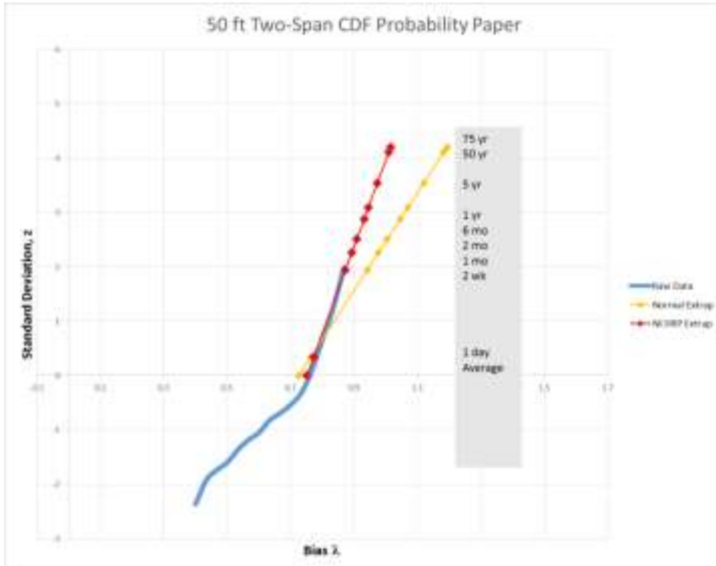

Figure 19. Raw Data and Normal Distribution CDF for 50-ft Two-Span on Probability Paper

Table 16 is the direct comparison between the normal distribution method and the NCHRP method for the 50-ft two-span bridge.

Table 16. Single Vehicle Bias for Normal and NCHRP Methods for 50 ft Two-Span

Time Frame	Normal Method	NCHRP Method
Average	0.725	0.750
1 Day Max	0.823	0.772
2 Week Max	0.964	0.872
1 Month Max	0.996	0.892
2 Month Max	1.022	0.908
6 Month Max	1.059	0.931
1 Year Max	1.081	0.945
5 Year Max	1.137	0.973
50 Year Max	1.194	1.009
75 Year Max	1.204	1.015

Using the NCHRP method instead of the assumed normal distribution could lower or even remove the recommended increase in the live load factor. However, this does not change the recommendation to incorporate the commentary low-boy tandem load for bridge design as is done here.

The NCHRP method single vehicle bias factors were applied to the load cases for the bridge archetypes. Although not all the intermediate steps are shown here, Table 17 presents the 75-year design life live load bias factors, λ_L.

Table 17. 75-Year Design Life Live Load Bias λ_L

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

Performing the reliability analyses for the NCHRP method live load bias factors, Figure 20 shows the required live load factor necessary for each bridge to reach a reliability index β = 3.50.

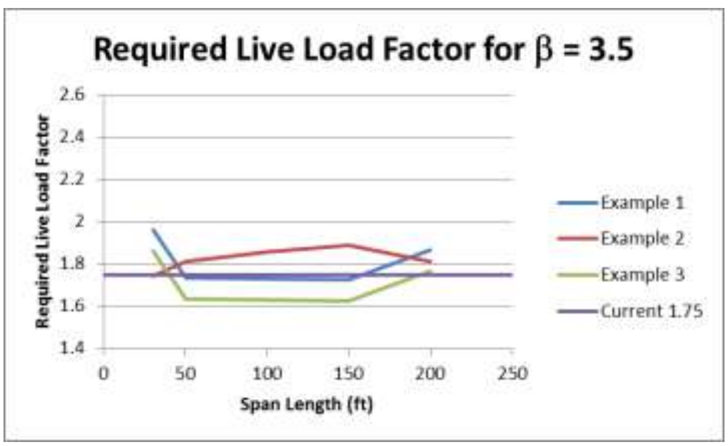

Figure 20. Required Live Load factor for Target β = 3.50 for NCHRP Method

Figure 21 shows that increasing the live load factor to 2.00 is not necessary if using the NCHRP method; however, an increase to 1.90 appears to be adequate. Figure 20 are the reliability indices for a live load factor γ_L = 1.90 (8.6 percent increase over 1.75).

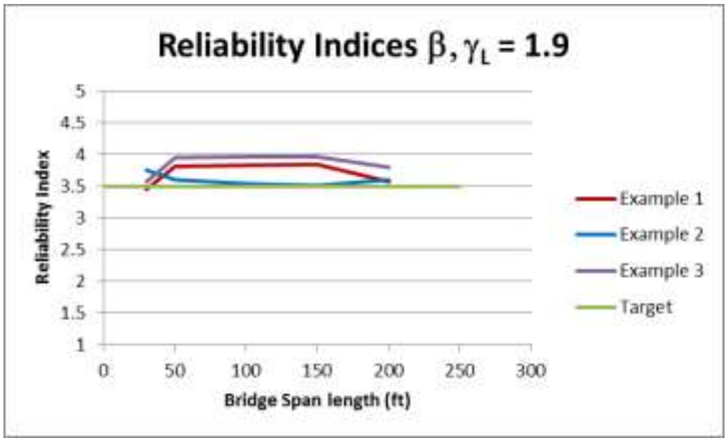

Figure 21. Reliability Indices for Live Load Factor γ_L = 1.90 for NCHRP Method

Summary

Wyoming's truck traffic and traffic patterns create larger demands on bridges than that considered in the AASHTO LRFD Bridge Design Specifications. The reliability analyses show that Wyoming's bridge reliability indices (safety) are less than the target implied in the AASHTO LRFD design procedures. Recommendations to raise the reliability indices were defined and are summarized in the conclusions.

CHAPTER 4: CONCLUSIONS AND RECOMMENDATIONS

The purpose of this work was to assess the safety (in terms of AASHTO design expectations) of bridges along the Interstate 80 corridor for Wyoming's truck traffic. The safety limit state in AASHTO is the Strength I limit state. Truck traffic in Wyoming produce demands on bridges that exceed those that are considered in the AASHTO LRFD Bridge Specifications. Wyoming's I-80 carries a large volume of cross-continental trucks and large energy industry trucks compared to many other states. Another concern is that I-80 is often closed during severe weather and truck traffic tends to gather on I-80 in large groups waiting for the road to open. When the road opens, this large group of trucks travels as a tight convoy.

The safety assessment was performed through reliability analyses. WYDOT records of truck characteristics (weights, axle spacings, lengths) were used to develop a truck database that included the top 1000 critical trucks that crossed the Pine Bluffs weigh station in the year 2014. A set of ten bridge archetypes were used to develop a live load model for the reliability studies. The set of bridges included simple-span bridges with lengths between 30 ft and 200 ft (positive moments) and two-span bridges with equal spans lengths of 30 ft to 200 ft (negative moments). The truck database was analytically applied to the bridge archetypes to determine the statistical properties of individual vehicles on the ten bridge archetypes. The maximum moment was assumed to be normally distributed which appears to be appropriate, yet slightly conservative. The load effect for shear was beyond the study scope.

Six load cases were developed for the live load model that included maximum single trucks, in-lane and adjacent lane multi-presence of two vehicles and herds of 4 trucks traveling as a group. The load cases also considered bunching of vehicles during road closures along I-80. The load cases are:

1. Single Truck
   a. Max 75 yr Truck
2. Side-by-Side Multi-Presence
   a. Max 5 yr Truck side-by-side to Max 5 yr Truck
3. Same Lane Multi-Presence
   a. Max 5 yr Truck followed by Max 5 yr Truck, Headway = min 50 ft
   b. Max 2 mo Truck followed by Max 2 mo Truck, Headway = min 10 ft, no Impact
4. Herd of 4 Trucks Multi-Presence
   a. Two side-by-side Max 2 wk trucks followed by Two side-by-side Max 2 wk Trucks, Headway = min 50 ft.
   b. Two side-by-side Average trucks followed by Two side-by-side Average Trucks, Headway = min 10 ft, no Impact

Through statistical analysis, a 75-year design life live load model was developed for the mean and coefficient of variation for the maximum load effect. The live load model was applied to reliability studies on example bridges (positive and negative moments) with varying ratios between live load and dead load. Reliability indices were produced through Monte Carlo simulation for safety assessment of Wyoming bridges along I-80 for truck traffic characteristics moving across those bridges.

Findings

Truck traffic along I-80 creates more demand than that assumed in the AASHTO LRFD bridge design procedures. The greater demand results in reliability indices that do not meet target safety levels. For shorter two-span bridges, Wyoming's truck traffic creates significantly more demand on bridges and results in unacceptably low reliability indices.

The target reliability index is $\beta = 3.50$. For the medium and longer span bridges, the analyses show the minimum β for positive moments is approximately 3.25 and for the negative moments it is just below 3.0. For shorter spans in positive moment, the minimum reliability index is approximately 3.0. However, for shorter two-span bridges in negative moment, the reliability index approaches 2.00. This is an important finding.

Recommendations

Bridges along I-80 do not meet the expected safety associated with a target reliability index of 3.50. Two issues should be addressed: (1) the unacceptably low reliability indices for short multi-span bridges (2) the overall low reliability indices for all span lengths.

There is an optional low-boy tandem load where there is a tandem in adjacent spans in the AASHTO LRFD commentary that significantly increases the negative design live load moments. Using the low-boy tandem, the reliability indices for the shorter two-span bridges were raised to 3.00 and above, placing this bridge type into the range of the reliability indices for the other length bridges.

Recommendation – WYDOT incorporate the commentary low-boy tandem load case as part of the HL93 loading for designing interstate bridges

If the commentary low-boy tandem loading is used, all of the reliability indices are fairly consistent. However, they are below the target. Raising the design live load factor, γ_L, directly and fairly uniformly increases reliability indices. An increase in γ_L to 2.00 (from 1.75) increases almost all of the reliability indices above 3.50 with just a couple dipping slightly below.

Recommendation – WYDOT increases the live load factor, γ_L, to 2.00

An alternative to a live load factor increase to 2.00 is to consider a different method for the statistical properties of the live load model. The AASHTO LRFD Bridge Specifications were developed under NCHRP projects that used truck database raw data upper tail statistical procedures to estimate maximum truck load effects. When the procedures were applied to the Wyoming 1000 truck database, the required increase of the live load factor is 1.90, smaller than the 2.00 noted above.

Alternative Recommendation - WYDOT increases the live load factor, γ_L, to 1.90

Future Considerations

The results of this work shows that the truck traffic in Wyoming does produce demands on bridges that exceed those that were considered in the development of the AASHTO LRFD Bridge Specifications. Recommendations were offered to maintain the expected Strength I limit state safety for bridges along the I-80 corridor. However, given the results, there is a concern for the AASHTO damage limit states. The AASHTO Service II limit state controls the structural damage and permanent set. The live load model has been developed and could be applied to the Service II limit state to assess the potential for premature damage, cumulative damage and rideability issues.

The database and procedures developed are applicable with modification for the Service and Fatigue limit state. These are important and hence a phase II study should be considered.

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