LATERAL LOAD DISTRIBUTION ON BRIDGES

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ABSTRACT

The AASHTO LRFD design code for maximum live loads is believed to be overly conservative. By achieving greater accuracy within the design code money can be saved on bridge materials and used more efficiently on other aspects of bridges, such as maintenance. In an attempt to determine the level of conservativeness, the focus of this paper is primarily on lifetime maximum live loads due to a multiple presence of trucks on bridges. Only girder bridges were covered in this paper. The configuration and loading of trucks within multiple presence events and the rate of occurrence of such events were observed on a representative Interstate highway. Accepted statistical methods were used to determine the design point value (Q*) for lifetime maximum live loads. The single presence of a truck and the amount of load it exerts was also studied and a design point value was determined for single presence. A comparison of single to multiple presence design point values was performed. The larger design point value was then used in a comparison to the most probable code predicted lifetime maximum live load and showed a numerical level at which the design code is overly conservative.

INTRODUCTION

Design engineers must design for the worst case scenario and be confident that their bridge or structure will withstand a maximum loading condition. When considering maximum live loads on bridges one may be drawn to think of the heavy live loads exerted on bridges during construction. One may also think of the occasional permit required transportation of an 80+ ton girder, which is the weight of girders commonly transported along our nation's highways. While not all trucks are heavily loaded permit vehicles, a combination of trucks simultaneously crossing a bridge can exert heavy loads on that bridge. Such simultaneous crossings of trucks are termed "multiple presence events". The loadings and configurations within a multiple presence are not fully understood in design and statistical research must be done to better classify multiple presence loads and occurrences.

Currently for girder bridges, the AASHTO design code incorporates multiple presence occurrences within equations for girder distribution factors. A girder distribution factor represents the amount of a wheel line loading carried by a bridge girder. For example, say the wheel line of a truck drives directly over a bridge girder then theoretically the bridge girder should carry the full load of the wheel line and the distribution factor for that girder would be one. In actuality, the flexure of the bridge deck would redistribute some of the wheel line loading to other girders so the distribution factor would be less than one. For longer span bridges, AASHTO uses the multiple presence factors [1] provided in section 3, (Table 3.6.1.1.2-1) of the AASHTO LRFD code. In designing longer span bridges, design engineers design for a condition where all lanes of traffic are loaded side by side with trucks. Since this project dealt with girder bridges, only the conservativeness within girder distribution factors was investigated.

The objective of this research was to better classify multiple presence occurrences by studying and collecting data from ambient traffic, then using the collected data to address the conservativeness within the code and locate areas in the code where reduction of design loads is possible.

BACKGROUND

In order to investigate lifetime maximum live loads a thorough understanding of the design code and the safety incorporated within the code is required. The most basic design equation states that the resistance of the sections used in the design should be greater than or equal to the effects of the loadings experienced by the design [2]. If this basic equation shown below is not satisfied, the limit state is exceeded.

Resistance of sections (R) ≥ *Effects of Loadings* (Q) (1)

Uncertainties in loads have long been accounted for by using judgment and learning from past experiences. Design philosophy based on the theory that if a structure fails, the elements within the structure have to be made bigger has served well for centuries and is reflected in the ancient structures that still stand today. For example, the Pont du Gard built in Nimes, France during the Roman Empire is a remarkable historic structure that still stands due to the redundancy it has incorporated in its design. The Pont du Gard is a large aqueduct/bridge more than capable of handling the loadings of its time. Roman engineers followed the general guidelines of equation (1) so well that when the river that the Pont du Gard spans had its worst flood to date and washed away a modern bridge, the Pont du Gard remained standing with only minor damage. Whether or not Roman engineers understood equation (1) to the same level that we understand it today is unclear, but it is evident in the Pont du Gard that redundancy and over design creates a large factor of safety within a structure.

Today, we view equation (1) in the modified form shown below in equation (2). The modified form is termed the equation of Load and Resistance Factor Design (LRFD) and is currently used by AASHTO in bridge design [2].

$$\Phi R_n \geq \Sigma \gamma Q_n \qquad (2)$$

In this equation Φ is a statistically based resistance factor usually less than one and γ is a load factor, statistically based and usually greater than one. The terms R_n and Q_n represent nominal values for both resistance of materials and effects of loadings respectively. These nominal values were determined through the use of research based statistics and can be seen in the probability density function shown in Figure 1 below.

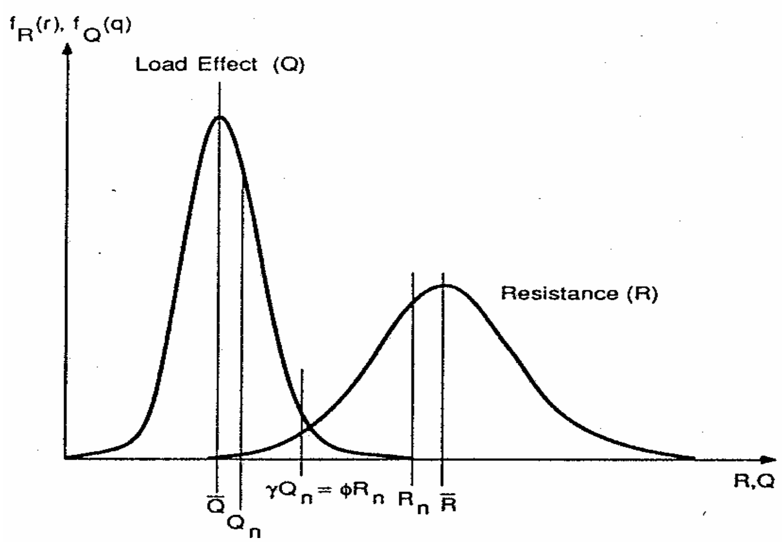

Figure 1. LRFD probability density function.

As seen in Figure 1, the probability density function for resistance is mostly to the right of that for load. The overlap of the two curves represents a region that leads to the probability of failure. To achieve an acceptably small probability of failure LRFD uses load and resistance factors (γ and Φ) as multipliers to account for the unpredictable. The LRFD equation achieves added safety with conservative nominal values for both load and resistance.

There is a constant demand for more accuracy within the LRFD equation. In an attempt for more accuracy in design loads, prior studies on maximum live loads have been performed by Ghosn and Moses [3]. Ghosn and Moses established permit checking equations in two cases of permit overloads. The first case pertains to the effects of a permit vehicle traveling alongside a vehicle of normal traffic. The second case pertains to the effects of a permit vehicle traveling alongside another permit vehicle. In 1991, based on Ohio DOT permit overloads, Fu and Moses developed an overload checking method for highway bridges [3]. Probably the most renowned person in the field of maximum bridge live loads is Professor Andrzej S. Nowak from the University of Michigan. Professor Nowak is credited for using statistical parameters in the calibration of the ASSHTO LRFD code [4]. Professor Nowak developed load and resistance models from weigh-in-motion (WIM) data he gathered and used those models in a statistical analysis to determine a design point value (Q*) representative of the lifetime maximum truck loading that a bridge would experience. The Gaussian distribution that Professor Nowak used in calibration of the AASHTO LRFD code is shown below in figure (2).

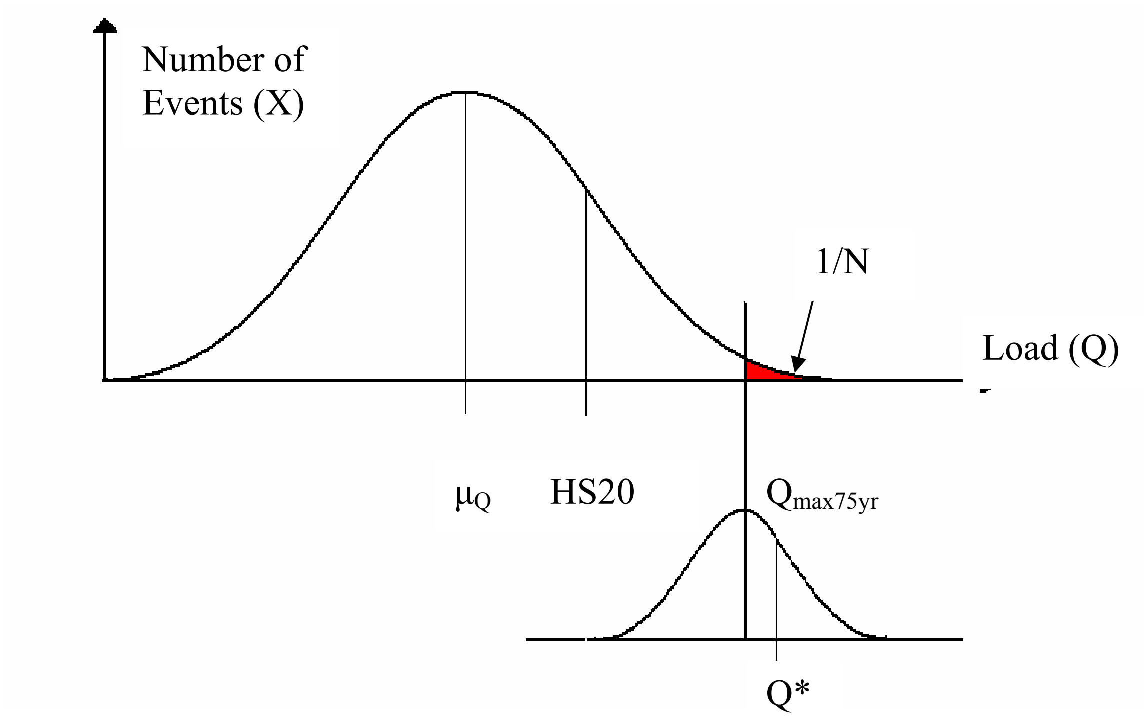

Figure 2. The Gaussian distribution used by Nowak in determining the design point value for the maximum live load experienced in the 75 year design life of a bridge.

The first Gaussian curve illustrates the distribution of truck loadings sampled by Professor Nowak with a mean value μ_Q and the design value for an HS20-44 above the mean. The shaded region is symbolic of the exceedance probability (1/N). The variable (N) is representative of the number of trucks experienced by the bridge in its 75 year design life. The corresponding load, ($Q_{max75yr}$) is the load that is exceeded on an average once in 75 years. (χ/N) is a function of (N) that represents the number of standard deviation above the mean where ($Q_{max75yr}$) lies. Once the 75 year maximum loading is determined it is used in another distribution as a mean value to account for uncertainties; and a conservative design point value (Q*) is then determined. The design point value depends on the coefficient of variation (V_Q), the sensitivity (α_Q), and the target reliability index (β). Professor Nowak's method for obtaining the design point value is the currently accepted method and is the method that was used in this project to determine observed values of Q* for both single and multiple presences.

METHODALOGY AND APPROACH

Live traffic cameras were chosen on as the best approach at monitoring how frequent a multiple presence was on a busy stretch of highway. Traffic cameras provide safety for the traffic observer and an overhead view in which truck configurations can easily be determined. The University of Delaware's Intelligent Transportation Systems (ITS) laboratory provided the desirable environment for traffic monitoring. The ITS lab is equipped with a large viewing screen that portrays any traffic camera selected. The footage from the traffic cameras are on the University of Delaware website courtesy of the Delaware Department of Transportation (DelDOT). The criteria for the selecting the appropriate traffic camera for viewing consisted of:

1. Selecting a highway with a high value of ADTT
2. Selecting a highway with four lanes
3. Selecting a highway with the most undisturbed flow of traffic possible (meaning no on-ramps or off-ramps on that portion of highway)
4. Selecting a highway that had a visible reference point in which traffic would be recorded as soon as it reached that reference point.
5. Selecting a traffic camera that provides a steady view of the highway and provides a clear view of the traffic present on the highway.

The traffic camera that best met these five requirements was DelDOT traffic camera 007M that monitors a four lane stretch of highway on Interstate 95. The stretch of highway monitored relates to the bridge 1-704, used in determining the loadings of trucks. Both the stretch of highway and bridge 1-704 carry interstate 95 traffic and have four major lanes of traffic.

With the appropriate traffic camera selected a procedure was needed for monitoring multiple presence occurrences. The number of trucks involved and configuration of the trucks were considered the two factors of importance in a multiple presence. Noting the configurations within the occurrences allowed for categorization of the data. Categorization was classified according to the sample data sheet shown below in Table 1.

Test Number	Multiple Trucks (Number of Trucks in Lane)				Separation Distance (Same Lane Traffic) (in terms of truck lengths)					Stagger Distance (Multiple Lane Traffic) (in terms of truck lengths)				
	4	3	2	1	0.25	0.5	0.75	1	other	0.25	0.5	0.75	1	other
1				2					1.5					
2		1	1	1						X			X	
3			4			X X X								
4			1	1						X				

Table 1. Sample data sheet showing the data recorded in a multiple presence occurrence.

It was decided that the best method for visually gauging truck distances within their configurations would be to record the distances in terms of truck lengths. The standard truck length was considered to be that of the design truck, an HS20-44, with spacing between the first and second axle of 14 feet, and between the second and third axle ranging from 14 feet to 30 feet. The maximum distance recorded in a multiple presence was a distance of two truck lengths. It was decided that the effects of any truck outside that two truck length distance would not aid in producing a maximum loading condition. Each test within Table 1 is representative of a multiple presence occurrence. During each occurrence the number of trucks in each lane was recorded. The column termed separation distance was used when the multiple presence occurrence involved two or more trucks in the same lane. The column termed stagger distance was used in recording occurrences in which trucks were in adjacent lanes. For example, test 2 in Table 1 would be read in the following fashion. There were three trucks involved in this multiple presence. The trucks were in lanes three, two, and one. Since all the trucks were in adjacent lanes the stagger distance column was used stating that front bumper of one truck was a quarter of a truck length apart from the next and the third was a full truck length behind the others. An illustration of the stagger distances is given below in Figure 3.

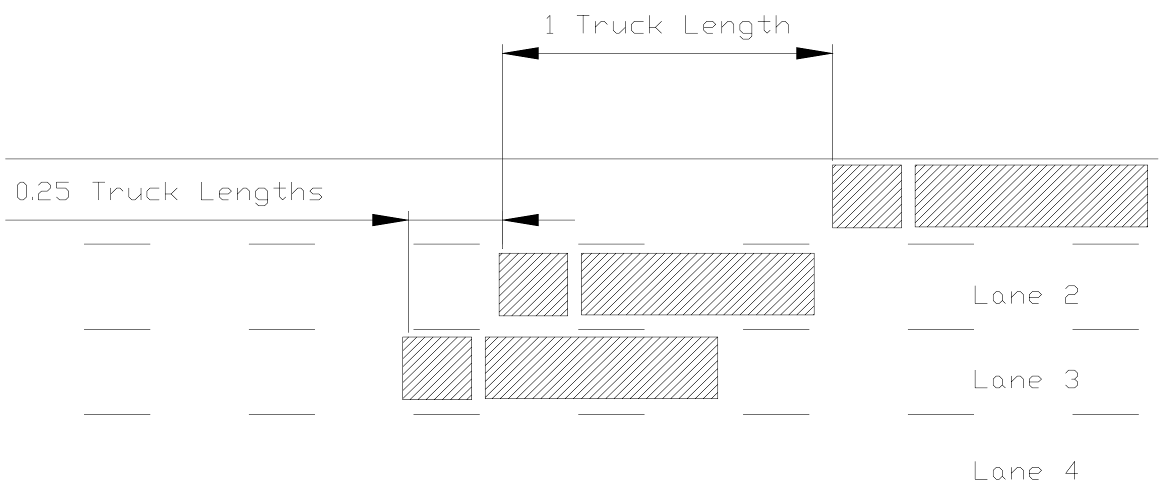

Figure 3. Truck configuration during the multiple presence observed in test 2 of Table 1

Test 1 in Table 1 involved a multiple presence of same lane traffic. Test 1 would be read in the following fashion. Two trucks were involved in this multiple presence. The trucks were both in lane one and the rear of the first truck was one and a half truck lengths from the front bumper of the second. An illustration of the separation distances is given in the Figure 4 below.

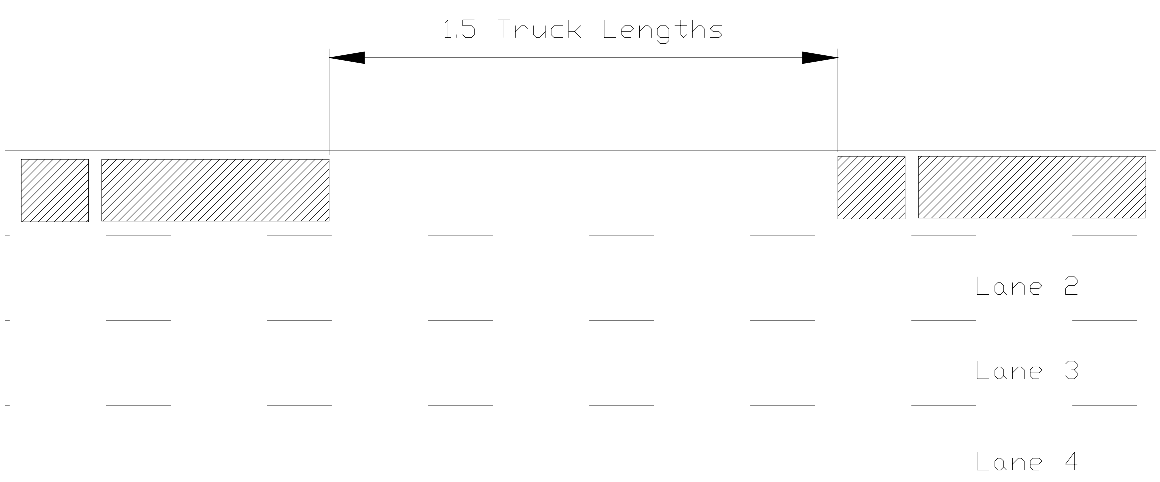

Figure 4. Truck configuration during the multiple presence of test 1 of Table 1

Single presence data was also recorded in the ITS lab and the only information recorded on a single presence was the lane of traffic in which it occurred. Table 2 below illustrates a sample data sheet of the information recorded during a single presence.

Test #	Single Truck (Number of Trucks in Lane)			
	4	3	2	1
1		1		
2				1
3				1
4			1	

Table 2. Sample data sheet showing the data recorded in a single presence occurrence.

Due to the rapid flow of traffic on interstate 95 it was determined that it would be too difficult to record both multiple presence occurrences and single presence occurrences within the same time frame. It is for this reason that one time frame was dedicated to monitoring and gauging distances only during multiple presence occurrences and another time frame was dedicated to single presence. Thirty minute time frames were used and five hours of data was collected from the ITS laboratory.

In obtaining field data on the loading of the trucks during multiple presences an appropriate bridge had to be selected for instrumentation. Bridge 1-704, which spans the Christina creek, was selected as the candidate bridge due to its high ADTT and the easy accessibility of its approach span. 1-704 is a slab-on-girder bridge skewed at an angle of 13° and consists of three simple spans. The two approach spans are 24.6 ft. (7.5m) in length and are non composite but were observed to act compositely. The main span is composite and 62.3 ft. (19m) in length. Bridge 1-704 carries Interstate 95 southbound traffic and consists of four lanes and an exit ramp. Original construction of the bridge consisted of four W24X84 steel girders spaced at 8 ft. 4 in. on center and two W36X135 fascia girders at the same spacing. Two separate occasions of widening the roadway added two W36X135 girders spaced at 5 ft. 3 in. to the northern end of the bridge, and four W36X135 girders at the spacing illustrated in Figure 5 to the southern end of the bridge. The bridge deck is composed of concrete and has a thickness of 8.5 inches. [5]

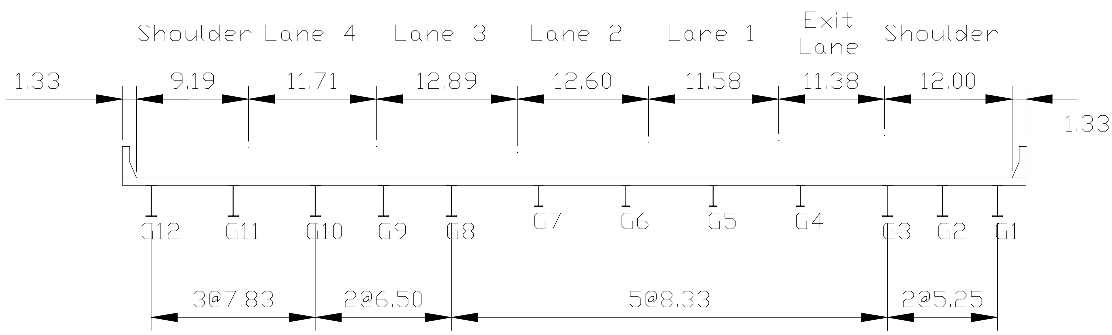

Figure 5. Cross-sectional view of Interstate 95 southbound bridge 1-704 (dimensions are given in ft.)

The twelve girders of the eastern approach span on bridge were instrumented with strain gages. The strain gages were placed at the mid span and on the outside bottom flange of each of the twelve girders shown in figure 5. Once the instrumentation was placed a person waited by the roadway for the opportune moment in which a multiple presence would occur and relay to another person via radio transmitter to initiate the test. The multiple presence configuration was visually observed and recorded on a data sheet in a similar procedure as that used in the ITS laboratory data. The configuration was recorded to simplify unnecessary guessing in the evaluation of strain time history plots. It is important to note that not all of the multiple presences that occurred within the testing time frame were recorded. The loadings during single presences were also recorded. Overall 64 data sets were recorded, of which 46 were multiple presences and 18 were single presences.

The strain data collected from bridge 1-704 was converted into Microsoft Excel format and the maximum value for micro-strain experienced by each girder during a test were placed on a separate spreadsheet in the form shown below in Table 3.

	MULTIPLE PRESENCE MAXIMUM MICROSTRAINS											
Test #	**G1**	**G2**	**G3**	**G4**	**G5**	**G6**	**G7**	**G8**	**G9**	**G10**	**G11**	**G12**
TEST1	0.59	6.72	24.30	29.93	49.35	32.90	6.77	2.47	0.58	0.59	1.12	2.96
TEST2	1.18	1.12	3.96	16.43	30.92	48.13	52.43	16.06	2.90	1.78	1.12	6.50
TEST3	0.59	1.12	2.26	5.28	8.33	44.48	58.63	17.29	6.95	1.18	0.56	2.96
TEST4	0.59	2.80	6.22	44.60	68.98	35.34	21.99	4.94	1.74	1.18	1.67	3.55
TEST5	1.18	1.68	3.96	5.28	10.70	42.65	56.38	16.68	1.74	0.59	0.56	2.37

Table 3. Maximum micro-strain values observed on each girder of bridge 1-704 during load testing of multiple presences.

The maximum micro-strain values from each test then had to be converted into moment values using a relationship between equations for flexure and stress-strain. The equation for flexure, Equation (3), relates the moment to principal stress as shown below.

$$\sigma = \frac{M \times y}{I} \qquad (3)$$

Where:

σ = principal stress
M = bending moment
y = distance from the centroid to the most extreme fiber
I = moment of inertia

The relation between stress and strain is defined in equation (4) as

$$\sigma = \mathrm{E} \times \varepsilon \qquad (4)$$

Where:

E = the modulus of elasticity of steel (29000 ksi)
ε = principal strain

Combining the equations (3) and (4) and solving for the moment provides equation (5) shown below

$$M = \frac{\mathrm{E} \times \varepsilon \times I}{y} \qquad (5)$$

The modulus of elasticity, the moment of inertia, and the distance from the centroid to the most extreme fiber are known quantities and can be combined to form a constant. Due to the different properties among the two girders, the W24X84 and the W36X135 sections required two separate equations when evaluating their maximum moments. Also since the approach span acts compositely new values for I and y had to be calculated. The composite cross sections for the two girders [7] are shown below in figure 6.

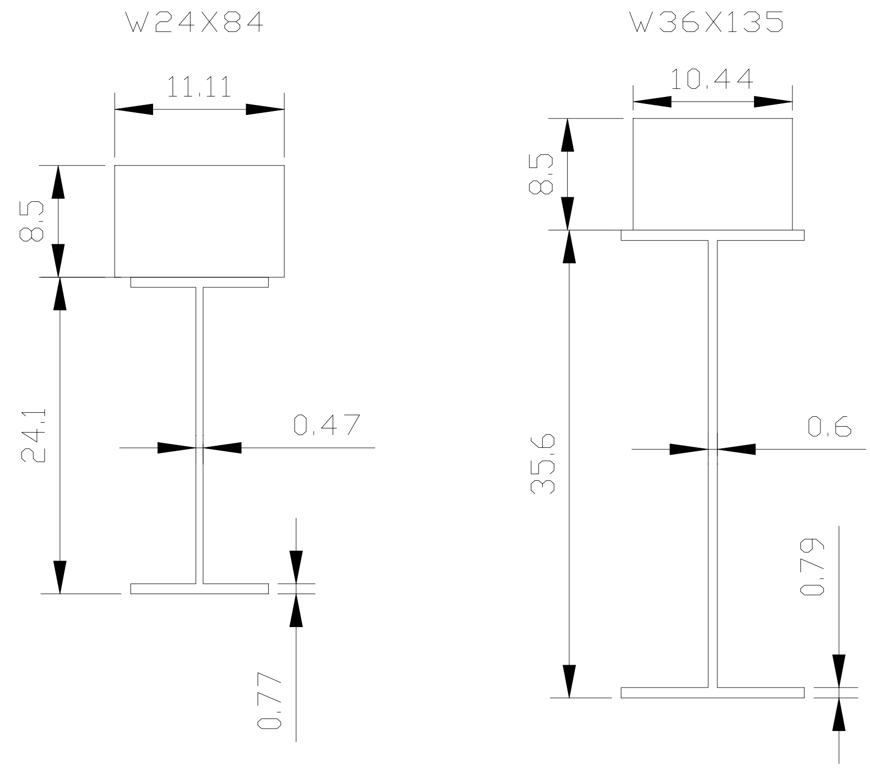

Figure 6. The composite cross sections of the two types of girders used in bridge 1-704 (dimensions are given in inches).

In order to imitate the composite behavior between the concrete deck and the steel girder the concrete deck had to be reduced to the size of a piece of steel with equivalent strength. In order to do so the width of the concrete had to be divided by a value of nine, representative of the modular ratio between concrete and steel. For the W24X84 girders which are all spaced at 8.33 ft. from each other the effective transformed reduced down to 11.11 inches. For the W36X135 girders, the spacing that provided the larger moment was 7.83 ft. which reduced down to an effective transformed width of 10.44 inches. The centroid (γ) for both composite sections was found using equation (6) shown below.

$$\gamma = \frac{\Sigma \gamma A}{\Sigma A} \qquad (6)$$

The A in equation (6) represents the area. From equation (6) the centroid for the W24X84 was determined to be 7.61 inches from the top of the concrete making the value for y equal to 24.99 inches. For the W36X135, γ was determined to be 11.03 inches from the top of the concrete and y was equal to 33.07 inches. The moment of inertia for the two sections was determined from equation (7) shown below.

$$I = \Sigma I \qquad (7)$$

The moments of inertia for the W24X84 and the W36X135 were calculated to be 8,076.6 in^4 and 21,507.3 in^4 respectively. Now with the two variables of equation (5) that describe composite behavior identified, equation (5) could be simplified into the following two equations (8) and (9).

For W24X 84 girders → $$M = \frac{29000 \times 8076.6 \times \varepsilon}{24.99} = 781{,}050\,\varepsilon \text{ [kip-ft.]} \qquad (8)$$

For W36X135 girders → $$M = \frac{29000 \times 21507.3 \times \varepsilon}{33.07} = 1{,}571{,}696\,\varepsilon \text{ [kip-ft.]} \qquad (9)$$

Equations (8) and (9) were used in table 4 to convert the maximum micro-strain values of table 3 into moment values. Since girder 5 of bridge 1-704 was rehabilitated with carbon fiber reinforced polymer (CFRP) plates, which provide an additional 11.6% of flexural strength, a correction had to be made [5]. The correction was implemented whenever girder 5 had the controlling strain value in a test, and involved reducing the observed moment by a factor of (61/69) [$\mu\varepsilon / \mu\varepsilon$]. The reduction factor includes strain values that were observed in actual testing of girder 5 on bridge1-704 before and after rehabilitation. Table 4 below shows the values for the observed moments of the controlling girders of Table 3.

	OBSERVED VALUES			
	Maximum Micro-Strain Among Girders	Maximum Strain	Maximum Value of Moment [Kip-ft]	
Test #			W24x84	W36x135
TEST1	49.35	0.00004935	34.08	
TEST2	52.43	0.00005243	40.95	
TEST3	58.63	0.00005863	45.79	
TEST4	68.98	0.00006898	47.63	
TEST5	56.38	0.00005638	44.04	

Table 4. This table shows the controlling value of strain being converted into a moment using equation (8) and a correction for girder 5 when needed.

The results for the 18 single presence tests were evaluated using the exact same procedures and tables as the ones just discussed for multiple presence occurrences.

RESULTS

The data collected from the ITS laboratory was collected on five separate days and at random periods of the day ranging from 10:00am to 3:00pm due to accessibility of the lab. Overall two hours and thirty minutes of multiple presence occurrences were recorded and two hours and thirty minutes of single presences were also recorded. Within the two hour thirty minute time frames 1005 single presences and 143 multiple presences were observed. These observed numbers indicate a 14.3% occurrence of multiple presences within the average daily truck traffic (ADTT). A value of 33.3% is currently assumed in the AASHTO code and was developed in Professor Nowak's research. However, Professor Nowak's value was never supported by any field data on multiple presence probabilities [6]. When comparing the two values conservativeness is evident.

Once values for moments were obtained from Table 4, the data was ready for a statistical analysis similar to the one used in Professor Nowak's calibration of the AASHTO design code. First, Excel was used in determining a mean value and a standard deviation for the observed moments of both single and multiple presences. For the single presence data the maximum observed mean moment (μ_M) was 32.72 kip-ft. and the standard deviation (σ) was 13.46 kip-ft. For multiple presence data the maximum observed mean moment was 37.42 kip-ft. and the standard deviation was 13.46 kip-ft. As expected the mean moment for multiple presence data was larger than single presence data but not by a significant amount. The reason for such a small difference is probably due to the small amount of data recorded and the fact that the bridge was only 24.6 ft. in length. A short span bridge would only be affected significantly by a multiple presence configuration in which the trucks were directly adjacent of each other when on the bridge, such an occurrence only happened once during the testing time frame. It was also noticed that the standard deviation is larger for the single presence then multiple presence data. This scenario is believed to be a result of the fewer number of single presence tests taken. Since there were more tests recorded among multiple presences there were more values which could have lead to less scatter among the data. The values for mean moment and standard deviation were used in a Gaussian distribution to find values of the maximum 75 year truck moment ($M_{max75yr}$) for both single and multiple presences. $M_{max75yr}$ was found using Equation (10) shown below.

$$M_{max75yr} = \mu_M + (\chi)_{1/N} (\sigma) \qquad (10)$$

The only undetermined variable in equation (10) is the normal deviate $(\chi)_{1/N}$ which is related to the exceedance probability (1/N). (N), being the number of trucks experienced by a bridge during its design life is determined using equation (11).

$$N = (ADTT) \text{ x } (365 \frac{days}{year}) \text{ x } (75\, year) \qquad (11)$$

In order to calculate N for single and multiple presences on bridge 1-704 the ADTT value for bridge 1-704 was needed. A value of 3,695 for ADTT was obtained from DelDOT [8]. The value of N for single presence ADTT was determined to be 1.01×10^8 trucks. The ADTT for multiple presences was determined by taking 14.3% of 3,695 yielding N for multiple presences to be 1.44×10^7 trucks. Once N was calculated for both single and multiple presences, the –log (N) was then calculated to provide a value that could be used in the probability table of appendix J for the extrapolation of the normal deviate (χ). For single presence the –log (N) provided a value of 8.00497 which was extrapolated between normal deviates of 5 and 6 to provide a value of 5.594 for (χ). For multiple presence the –log (N) provided a value of 7.15834 which was also extrapolated between 5 and 6 to provide an (χ) value of 5.250. With values for all the variables in equation (10) determined, the maximum 75 year moments for single and multiple presences were then determined and are shown below.

Single Presence → $M_{max75yr} = 32.72 + (5.594)(13.46) = 108.01$ kip-ft

Multiple Presence → $M_{max75yr} = 37.42 + (5.250)(10.31) = 91.55$ kip-ft

It is interesting to note that the single presence 75 year maximum moment is larger than the multiple presences. The difference is due to the larger standard deviation for single presences as well as a much larger value of (N). Since the single presence value is the larger of the two it was the value that was used in the remaining analysis and comparison to the code values.

Using the $M_{max75yr}$ value for single presence as a mean value in a second distribution the observed design point value (Q*) was then determined using equation (12) shown below.

$$Q^* \approx M_{max75yr} (1 + (V_Q)(\alpha_Q)(\beta)) \qquad (12)$$

The value for the coefficient of variation (V_Q) was taken to be 20%, the value for sensitivity (α_Q) was taken to be 60% and the value for the reliability index (β) was taken to be 3. Ultimately equation (12) was reduced to equation (13) shown below.

$$Q^* \approx M_{max75yr} (1.36) \qquad (13)$$

The observed design point value for the lifetime maximum live load experienced by bridge 1-704 was determined to be 146.89 kip-ft.

Now that an observed design point value was calculated for bridge 1-704, a code predicted design point value had to be determined for comparison. Using the computer program BRASS, a maximum moment due to an HS20-44 loading was determined to be 98.4 kip-ft. The BRASS value was also verified through hand calculations and proven to be correct. An impact factor of 33% was added to this value to provide a moment of 130.87 kip-ft. Next, distribution factors were calculated for bridge 1-704 using AASTHO equations [1] provided in Section 4, Table 4.6.2.2.2b-1 and the largest governing distribution factor was determined to be 0.761. The governing distribution factor is for a W24x84 girder when two or more lanes of traffic are loaded. By multiplying the distribution factor by the moment, the new value for the maximum moment was determined to be 99.59 kip-ft. In a final step of multiplying by a load factor of 1.75 the code predicted design point value for the lifetime maximum live load was determined to be 174.29 kip-ft.

In comparing the observed to the code predicted design point value there is a 27.40 kip-ft. difference. This is a significant difference between the two which proves conservativeness within the design code due to the data collected in this project.

CONCLUSIONS

The objective of estimating the level of conservativeness in the LRFD code was met. In the example considered herein, the code specified design moment was found to exceed that predicted using site specific data by about 19%. However this numerical value is not representative of all bridges and may not even be the most accurate value for bridge 1-704. Error was definitely present in this project and could be improved upon in future experiments. Error could have been present in visual observations of multiple presences. Error could have also been present within the instrumentation used on bridge 1-704. There are other areas in which error could have crept in but the main purpose of this research was to develop a procedure that could be improved upon, and that purpose was achieved with a certain level of success.

An important observation that could be made from this research is that single presence loadings should by no means be ignored when compared to multiple presence loadings. As was shown in this research single presence moments were greater the multiple presence moments. This research just proves that both cases should be evaluated when designing bridges.

Another important observation to be made from this research is that the current LRFD design procedures may be too conservative. As was shown in this research, reduction of the LFRD values is possible. Possible areas of reduction are distribution factors, load factors, the HS20-44 design truck, or ADTT values. All of the listed area could be studied in further research as to how much reduction could be allowed. As a curiosity, the design point value for an ADTT of 10,000 was calculated and the difference when compared to the actual ADTT of bridge 1-704 was roughly 2 kip-ft. This small difference just indicates that ADTT does not significantly affect lifetime maximum live loads. The most likely candidate for reduction of the area listed would probably be distribution factors.

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APPENDIX