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Calibration factors for AASHTO LRFD and Pakistan code of practice for highway bridges based on statistically analyzed wim loads

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Abstract: Based on the statistically analyzed Weigh In Motion (WIM) data, WIM Peak recorded data and National Highway Authority (NHA) Pakistan legal load limits a detailed comparison is performed with AASHTO LRFD (HL-93) Loading and Pakistan Code of Practice for Highway Bridges (PCPHB 1967) live load model in order to derive Calibration Factors. A two lane simply supported bridge of span lengths varying from short (20m) to medium (50m) is modeled in Leap CONSPAN; a software developed by Bentley Software, Inc. All the above mentioned loading patterns are applied to the modeled bridge structures to study the live load effects. Based on the results obtained from analysis, Calibration Factor "r" from short to medium spans has been derived both for AASHTO LRFD and PCPHB for the bridge designing in Pakistan. Calibration Factor "r" based on 99.7% confidence level using lognormal statistical distributions for AASHTO LRFD is 1.67 and 1.09 for PCPHB. It means that the AASHTO LRFD and PCPHB live loads should be enhanced by 67% and 9% respectively for the analysis and design of bridges in Pakistan. Calibration Factor "r" when WIM peak loads considered are 2.45 and 1.56 for AASHTO LRFD and PCPHB respectively.

Keywords: *AASHTO LRFD, Pakistan Code of Practice for Highway Bridges, WIM Loads, Statistically Analyzed WIM Loads, Live load effects comparison, Calibration factors*

1 Introduction:

Highway bridges are required to be designed to support all vehicular loads safely that are expected to move over it in its design life. Most considerable of all the loads for any bridge structure are the live loads for determining strength parameters of the structure. Advancement in truck technology with passage of time and uncontrolled overloading are the main factors, that have occupied minds of researchers and has resulted in frequent updating of the international codes of practice for bridge designing. Different live load models have been used in different parts of the world to represent their domestic traffic trends. Unlike the advanced countries most of the developing countries like Pakistan don't have its updated code of practice for bridges. PCPHB [1] is based upon AASHTO Standard of 1961. Design live loads are old and the code has not been updated with time. As with the passage of time truck, axle loads and axle configuration has changed significantly but Pakistan is still using old British 1935 based standards.

In Pakistan Code of Practice for Highway Bridges (PCPHB, 1967) design live loads were taken same as that introduced by British in India in 1935 (BS 153, 1937). As the type of vehicles and the truck loads changed significantly so the author consider different type of vehicle configuration (As specified by NHA) and statistically analyzed WIM loads.

Live load effects on the bridges are influenced by a number of factors other than Gross Vehicular Weight (GVW), such as span length of bridge, axles spacing, number of axles, number of lanes, number of vehicles and vehicular occupancy as well. In Pakistan the unhealthy market competitions; illegal modification/fabrication/manufacturing of trucks and uncontrolled traffic situations on the roads has mainly put the bridge into a state of overloading. The illegal manufacturing such as a truck inadequate number of axles as per design specifications and also limited spacing puts a distress in the bridge structure and therefore the loss in the structural strength and durability.

Though the live loads as given by PCPHB (1967) are at slightly higher side of AASHTO but still do not represent the actual traffic trend in Pakistan. It is necessary for a bridge structure to safely carry the imposed loads without any reduction in strength and design life. The traffic trends in Pakistan have been changed significantly with time thereby abandoning the use of PCPHB loading for bridge design. The change in axle configuration, traffic congestion on roads/bridges and extensive overloading has resulted stressing the bridges beyond their strength limits and has made a major maintenance issue in Pakistan. Each year, billions of rupees are being spent for the rehabilitation, reconstruction and retrofitting of the bridges and its components due to these reasons.

Live load model was developed by S. Nowak in 1993 for highway bridges. Statistical data was used for models based on dead loads, truck loads and dynamic loads. Extreme 75 years loads were determined by statistical extrapolation. The important parameters included multiple presence factors and girder distribution factors. The extreme load was calculated based on simulation. The live load model developed served as the basis for development of new design provisions for Canada (OHBDC) and the United States (AASHTO LRFD) codes [2].

A concise explanation was given by Sexsmith in 1994 to the development of the live load model for the Ontario Highway Bridge Design Code, Canada [3].

Research work was carried out by Miao and Tianjum in 2001 to develop methodology for deriving statistical live load models for short span highway bridges. The methodology developed was applied to use the Hong Kong weigh-in-motion (WIM) data for establishing live load models for bridge designs for that region. The results of the data illustrated that the distributions of the traffic vehicles in Hong Kong was comprised of three basic distributions, namely the Inverse Gaussian, Lognormal and the Gamma distributions. The analysis also revealed that the maximum value of gross vehicular weight and axle weight during the bridge design life of 120 years, calculated according to the proposed statistical method were very close to the Hong Kong legal weight limits [4].

Study was carried out by K. Altay et all in 2003 to find that an increase in weight of the legal truck would shorten repair or replacement time for many bridges. For a few main routes of the state these results were used to assess the effects of a 10 to 20% increment in truck weight on the bridges. It was concluded also that transverse cracks in bridge decks are primarily caused by shrinkage soon after the construction and not affected by increase in axle weight. However, decks within thickness up to 9 inches or with girder spacing more than 10 feet may be susceptible to longitudinal/flexural cracking which could decrease the design life [5].

Considering the accurate live model in the design process is so distinctive that the Commonwealth of Virginia changed from using the old AASHTO-ASD specifications to AASHTO-LRFD specifications. The new specification features used, revised the live loads, gave more conservative impact factors, introduced a new load distribution method for the analysis and probabilistic based limit state design approach by T. Baber et all in 2007 [6].

Another research study was conducted by Lutomirska in 2009 to derive a live load model for long span length structures. The live load model derived was valid for span with lengths between 600 ft and 5000 ft, and it was also intended that the derived load model tends to reflect the current traffic patterns, number of trucks passing and their weights. It was concluded that most of the bridges are appropriate to be design with current HL-93 live load. It was also observed that some of the bridges, facing with high ADTT and increased percentage of overloaded loaded vehicles require special attention to the increased design live load [7].

Another similar research by J.O Brien et all in 2011 revealed that many highway bridges in the world used to carry traffic in two same direction lanes, and modeling the traffic loading on such bridges was the subject of the research. To model multiple presence loading events specially, those featuring one truck in each lane different assumptions were used there [8].

Research study was carried out by Paik et all in 2012 to examine the assessment method for the safety of concrete bridges in the Korean expressway based on the probabilistic concepts. A new updating procedure for bridge analytical model using measured response was presented. Further, by using the updated analytical model the rating factor of the test bridge was increased [9].

OBrien and Enright in 2013 worked on WIM data to determine aggressiveness of traffic for bridge loading. The database results were presented based on the analysis of extensive WIM data which was collected at five different European highway sites recently. The analyzed data was used for the simulation of Monte Carlo bridge loading considering the two lane traffic, both in the same and in bi-directional. Simulation model results were used to calculate the bridge load effects such as bending moments and shear forces; furthermore these load effects values were compared with the design specified values for bridges with different lengths by the Euro code for traffic loading on bridges [10].

Yi Zhou in 2014 worked on "Accurate and up to date evaluation of extreme load effects for bridge assessment". To update the late 1980's calculated maximum load effects used for the calibration of the live load model LM1 of the Euro code 1991-2, they used the most recent WIM data along with advanced extrapolation techniques like Gaussian fitting, Gumbel distribution, Rice formula, Generalized Extreme Value distribution. Based on the method robustness vs the traffic data, the extrapolation period and the load effects considered results were discussed, so as to assess the likelihood of the final results of the data [11].

2 WIM Data Collection:

Traffic data was obtained from the weighing station (Mullan Mansoor, Attock, Punjab) located at the longest route N-5 of Pakistan running from the port city of Karachi to the border crossing at Torkham the Afghanistan border, which was subjected to the commercial vehicles for the month of January, February and March 2010. Total 101,585 numbers of vehicles passed through Mullan Mansoor weighing station for the first three months of year 2010 were recorded. 2 Axle vehicles recorded were having the highest frequency whereas 5 Axle vehicles with the lowest. Recorded numbers of different type of vehicles based on number of axles are plotted in Figure(1).

3 WIM Data Analysis:

The collected WIM data was analyzed using lognormal distribution. If x is a log normally distributed random variable, then y = $ln(x)$ is a normally distributed random variable or If a random variable, x, has a normal distribution $N(\square, \square)$, then the distribution of y = exp[x] is log-normal, denoted $logN(\square, \square)$. The probability density function of such a random variable has the form

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma x} \exp\left(-\frac{(In(x)-\mu)^2}{2\sigma^2}\right), x > 0 \quad \text{....... Eq1}$$

Where μ is the location parameter or log mean, and σ is the scale parameter or log standard deviation. The mean value and variance are given in Equation (2) and Equation (3) respectively.

$$\bar{\mu} = \exp\left(\frac{1}{n}\sum_{i=1}^{n}\log(x_i)\right) = \left(\prod_{i=1}^{n} x_i\right)^{1/2} \quad \text{.......... Eq 2}$$

$$\bar{\sigma} = \exp\left[\left[\frac{1}{n-1}\right]\sum_{i=1}^{n}\left[\log(\frac{x_i}{x})\right]^2\right]^{\frac{1}{2}} \quad \text{...... Eq 3}$$

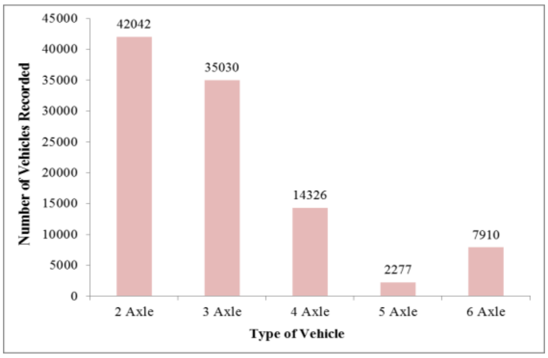


Figure 1: *Number of vehicles recorded at WIM station*

Based on mean x/ standard deviation^[1],[2] ,[3] i.e. 68.3%, 95.5% and 99.7% confidence levels, Statistically analyzed WIM loads were developed for modeling, analysis and comparison purposes. Lognormal distribution is applied based on confidence levels to cater the variation in the recorded traffic loads data so as to make rational decision for analysis of bridge structures. Peak recorded axle loads at weighing station, NHA legal loads [12] and PCPHB live loads are tabulated in Table(1), Table(2) and Table(3) respectively [13-16].

Table 1: *WIM Peak Recorded Axle Loads (KN)*

Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	GVW (KN)
171.32	270.47	-	-	-	-	441.79
133.67	221.53	245.36	-	-	-	600.56
196.52	248.69	204.27	198.49	-	-	847.98
123.86	242.81	163.38	213.20	229.28	-	972.52
97.97	215.35	213.30	187.70	209.17	228.20	1151.69

Table 2*: NHA Legal Axle Loads (KN)*

Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	GVW (KN)
53.94	117.68	-	-	-	-	171.62
53.94	107.87	107.87	-	-	-	269.68
53.94	117.68	107.87	107.87	-	-	387.36
53.94	117.68	101.33	101.33	101.33	-	475.62
53.94	107.87	107.87	101.33	101.33	101.33	573.69

Table 3*: PCPHB Axle Loads (KN)*

Loading Class	Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	Axle 7	Axle 8	GVW
Class A	26.67	26.67	111.21	111.21	66.68	66.68	66.68	66.68	542.50
Class B	16.00	16.00	66.72	66.72	40.01	40.01	40.01	40.01	325.50
Class AA	68.65	137.29	137.29	137.29	137.29	68.65	-	-	686.46

Parametric live load models are developed based on statistically analyzed 68.3%, 95.5% and 99.7% confidence levels WIM loads which can follow the defined shapes while considering mean and standard deviation of the data sample. The developed statistical loads are mentioned in Table(4), Table(5) and Table(6) respectively.

Table 4: *68.3% Confidence Level Axle Loads (KN)*

Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	GVW (KN)
63.74	122.58	-	-	-	-	186.33
78.45	132.39	117.68	-	-	-	328.52
63.74	112.78	83.36	93.16	-	-	353.04
63.74	147.10	102.97	117.68	127.49	-	558.98
63.74	132.39	132.39	102.97	117.68	73.55	622.72

Table 5*: 95.5% Confidence Level Axle Loads (KN)*

Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	GVW (KN)
78.45	147.10	-	-	-	-	225.55
93.16	142.20	132.39	-	-	-	367.75
73.5	137.	102.	117.	-	-	431.4

5	29	97	68			9
78.45	161.81	117.68	137.29	147.10	-	642.33
73.55	147.10	147.10	117.68	132.39	98.07	715.88

***Table 6**: 99.7% Confidence Level Axle Loads (KN)*

Axle 1	**Axle 2**	**Axle 3**	**Axle 4**	**Axle 5**	**Axle 6**	**GVW (KN)**
93.16	166.71	-	-	-	-	259.87
107.87	156.91	147.10	-	-	-	411.88
88.26	166.71	122.58	137.29	-	-	514.85
93.16	181.42	132.39	152.00	166.71	-	725.69
88.26	161.81	161.81	132.39	147.10	122.58	813.95

4 Modeling:

Two lane single span simple I-beam bridge structures are modeled and analyzed using Leap CON- SPAN [17,18] for comparison of the live load effects. Axle spacing as recommended by National Highways Authority Pakistan are used for modeling of different axle vehicles as given in the Table(7).

***Table 7**: Axle Spacing's*

S. No	**Vehicle Type**	**Axle No**	**Axle Width (m)**	**Axle Width c/c (m)**	**1-2 (m)**	**2-3 (m)**	**3-4 (m)**	**4-5 (m)**	**5-6 (m)**
1	2 Axle	1 + 1	2.20	1.83	4.60	-	-	-	-
2	3 Axle	1 + Tendem	2.30	1.83	6.10	1.40	-	-	-
3	4 Axle	1 + 1 + Tendem	2.60	1.83	3.40	6.80	1.30	-	-
4	5 Axle	1 + 1 + Tridem	2.50	1.83	3.30	5.10	1.40	1.40	-
5	6 Axle	1+ Tendem + Tridem	2.50	1.83	3.50	1.20	5.80	1.40	1.30

For Modeling of the simply supported bridge the following parameters are taken into considerations.

- Bridge spans (20m to 50m)
- Bridge Cross-Section (Simply Supported)
- No of Girders (4 No in each case)
- No of Diaphragms (Variable in each case)
- Sizes of Girders (NHA Type C, E, G and J for 20m, 30m, 40m and 50m respectively)
- Slab Thickness (Constant in all cases)

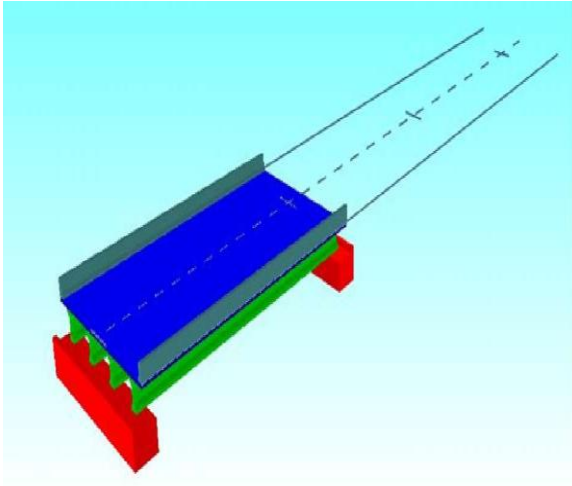


***Figure 2:** Two lane simply supported bridge*

Detailed Dimensions of NHA Girder Type C&E and Girder Type G&J are given in Table(8).

Typical cross sectional view of NHA girder types are drawn in Figure(1).

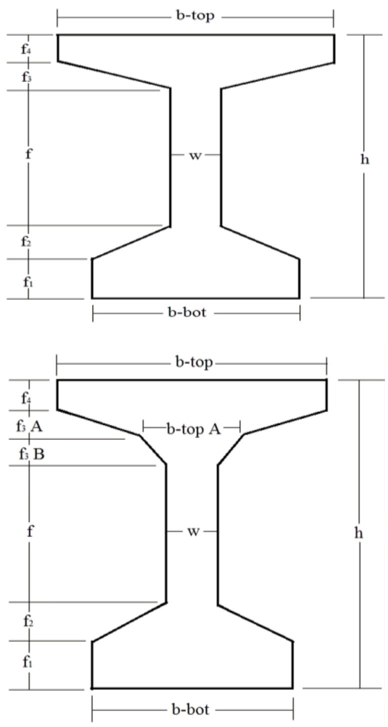


***Figure 1**: NHA Girder Type C, E (Left) and G, J (Right)*

NHA Girder Type C, E are used in 20m and 30m span while Girder Type G, J are used for 40m and 50m span modeling. Bridge cross section is shown in Figure (4).

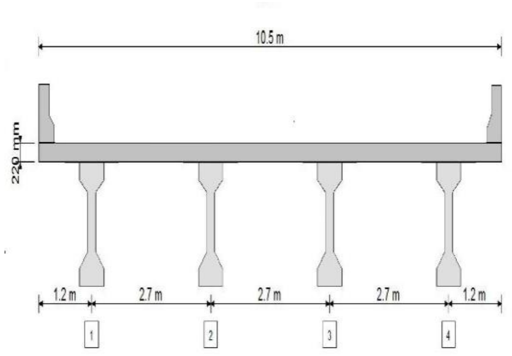
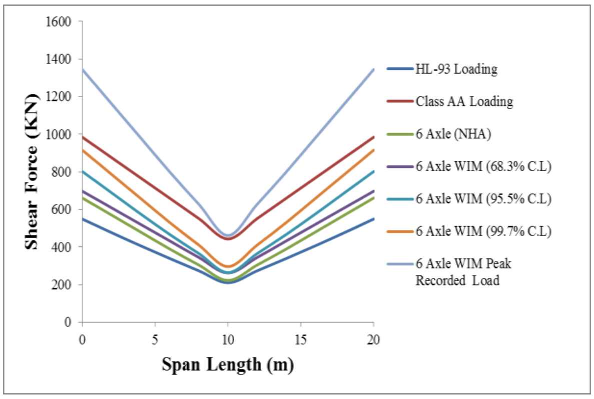


***Figure 4**: Bridge Cross Section*

While assigning the material properties "default" values are chosen. "1/2-270K-LL" straight pattern pre-stressing tendons are selected. For all type of Permit Vehicles expect for Class AA PCPHB Loading Multi Trips loading conditions and max with traffic frequency is selected with Permit Factor of 1.85. The Live Load Shear and Moment for Strength-II for the bridge spans considered are plotted as shown from Figure(5) to Figure(12).

5 Live Load Effects Comparison:

Following are the shear and moment diagrams for the considered live loads. Maximum load effects of all the vehicle type (for different number of axles) based on statistically analyzed WIM data, WIM peak load considered data are compared with AASHTO LRFD (HL-93) loading, PCPHB Loading and NHA Legal load limits. All the live loads are applied independently to the modeled bridge structures with varying span lengths. Maximum load effect of all the vehicular types along with HL-93 loading is plotted. In all the cases 6 axle vehicles produces maximum load effects. Figure(5) to Figure(8) shows shear force comparison of live loads with maximum effect of each load types for both exterior and interior girders.



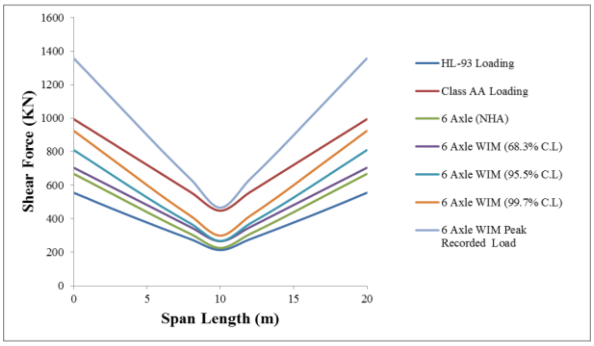
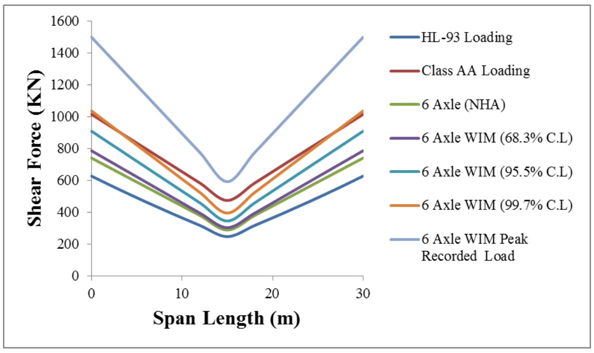


***Figure 2**: 20m span Exterior Girders (Left) and Exterior Girder (Right) Shear*



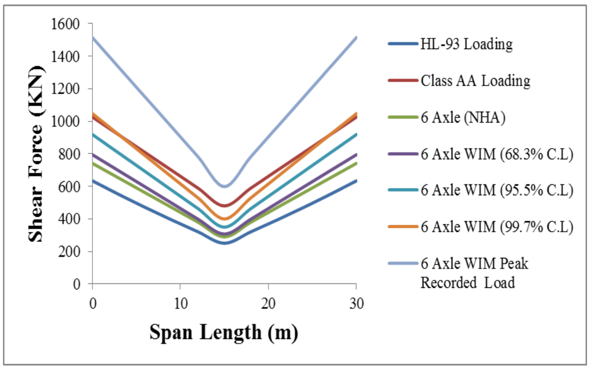
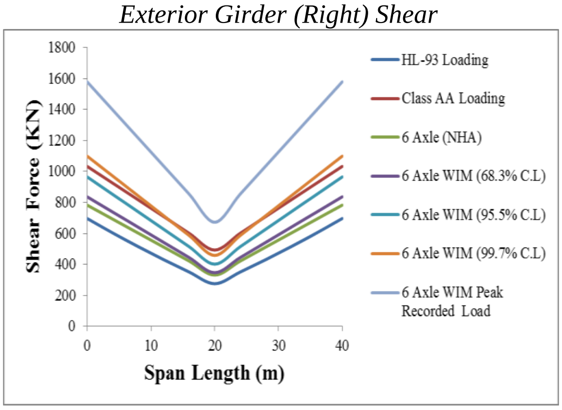


***Figure 3**: 30m span Exterior Girders (Left) and Exterior Girder (Right) Shear*



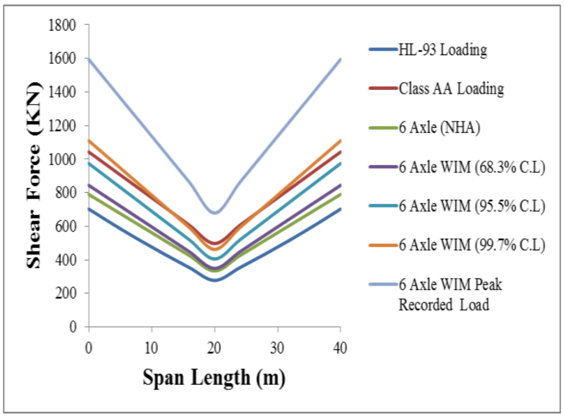
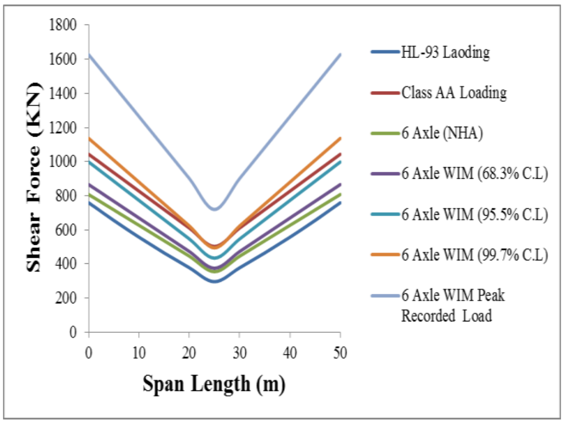


***Figure 4:** 40m span Exterior Girders (Left) and Exterior Girder (Right) Shear*



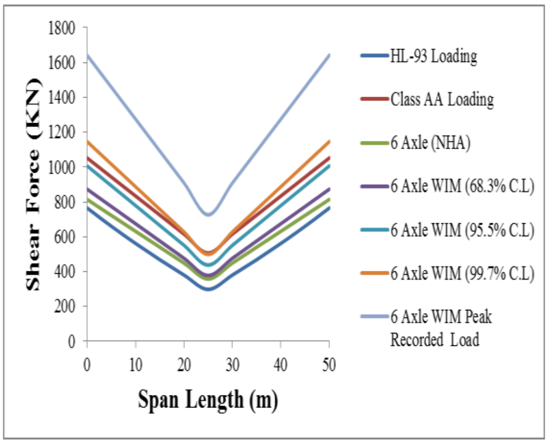
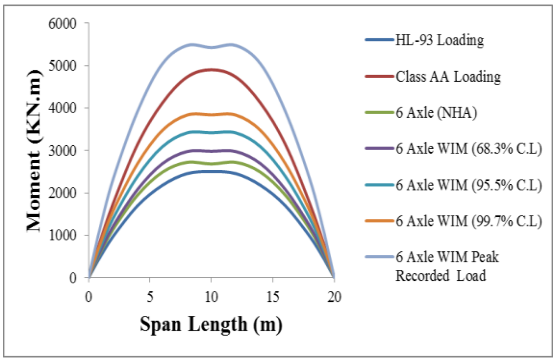


***Figure 5**: 50m span Exterior Girders (Left) and Exterior Girder (Right) Shear*

As the span length increases from 20m to 50m with an increment of 10m, live load effects changes too. Figure(9) to Figure(12) shows the bending moment variation and comparison of live loads with maximum effect of each load type considered along with HL-93 loading.



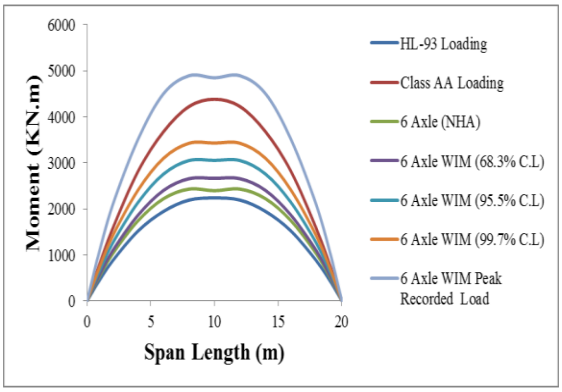
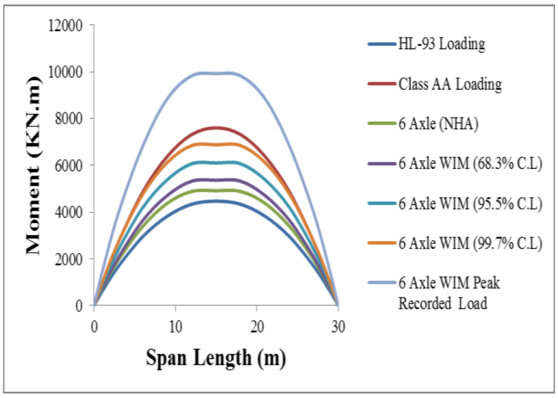


***Figure 6**: 20m span Exterior Girders (Left) and Exterior Girder (Right) Moment*



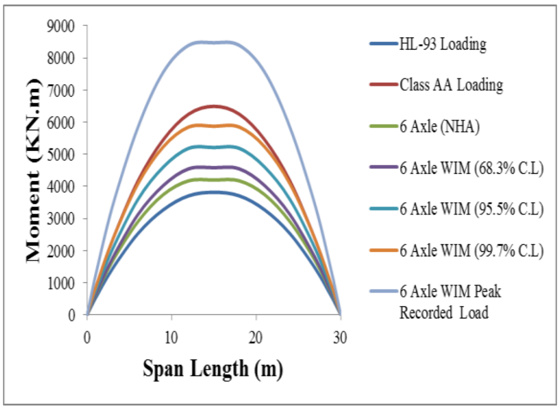


***Figure 7**: 30m span Exterior Girders (Left) and Exterior Girder (Right) Moment*

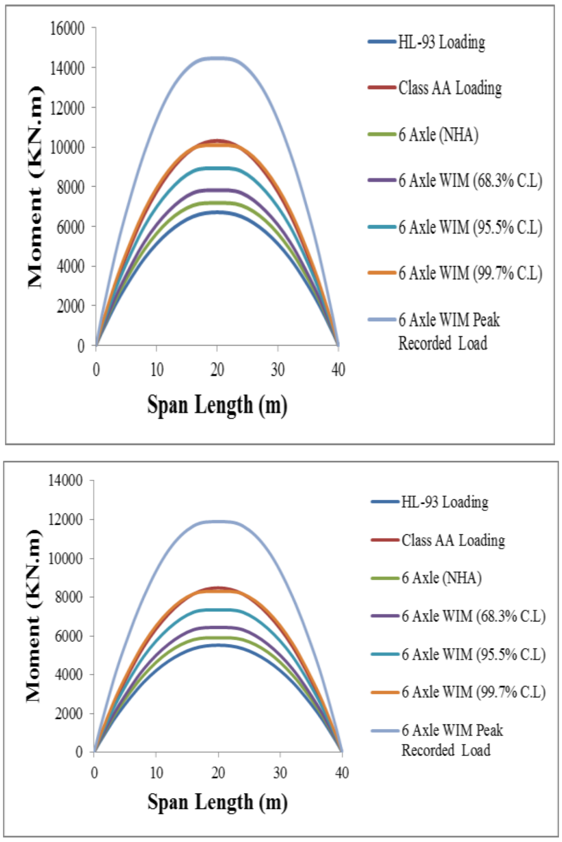


***Figure 8**: 40m span Exterior Girders (Left) and Exterior Girder (Right) Moment*

6 Calibration Factor "r":

It is almost a standard practice to propose calibration factors for different states or countries with some well-known code of practice which reflects the prevailing traffic loading in that region. Calibration Factor "r" is the ratio of maximum live load effects (Shear and Moment) of WIM of traffic to the maximum live load effects of renowned codes. As most of the advanced countries have their own updated bridge design code which are based on their prevailing traffic loadings so they don't need calibration factors for live load models. On the other hand developing countries like Pakistan which is still using outdated code of British India developed in 1967, which do not precisely reflect the current traffic conditions. There is a dire need to calibrate these load models with International codes such as AASHTO LRFD [19].

Following are the graphs showing the Calibration Factors for AASHTO LRFD, PCPHB based on two lanes Leap CONSPAN modeled simply supported bridges varying from 20m to 50m in span. Calibration Factors "r" is the ratio of the maximum live load effects to AASHTO and PCPHB live loading. Figure (13) and Figure (14) shows calibration for AASHTO and PCPHB for shear force and bending moment keeping in view variation in vehicular load data.

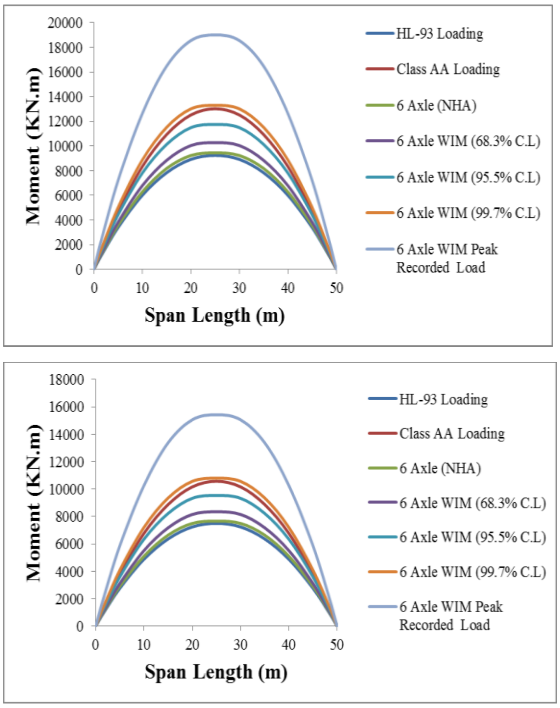


***Figure 9**: 50m span Exterior Girders (Left) and Exterior Girder (Right) Moment*

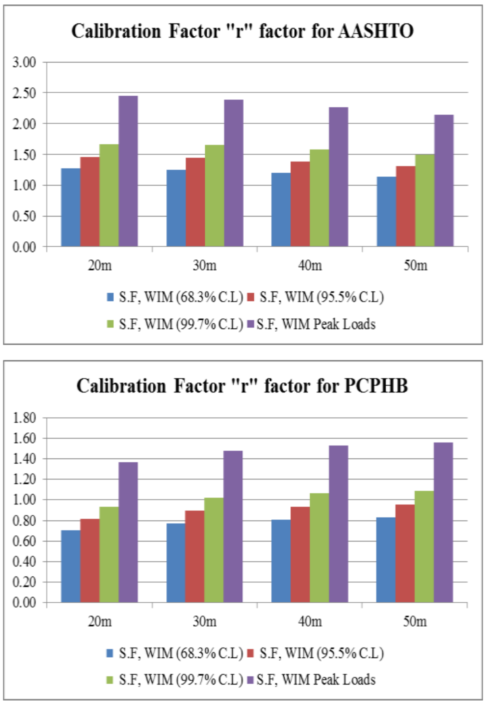
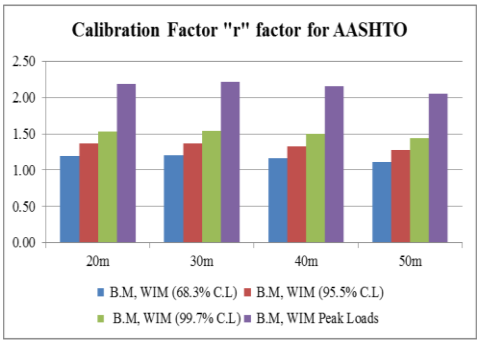


***Figure 10**: Shear Force Calibration for AASHTO (Left) and PCPHB (Right)*

What type of statistical WIM data range to be used to study the live load effects for analy¬sis and design of the bridge structure? The author derived Calibration Factors "r" regarding shear force and bending moments for various span lengths and different loading conditions while using different data range as shown in Figure(13) and Figure(14).



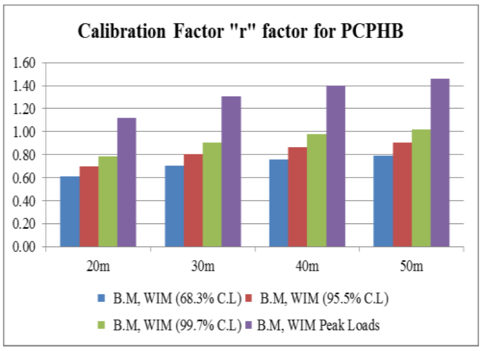


Figure 11: *Bending Moment Calibration for AASHTO (Left) and PCPHB (Right)*

7 Conclusions:

Based on this specific research some conclusions have been made as discussed below.

The actual Peak WIM vehicular data produces the maximum load effects in comparison to AASHTO LRFD (HL-93) or PCPHB live loads effects in all cases.

- For shorter span (20m) PCPHB Class AA load is the governing load if WIM Peak load effects are not taken into consideration.
- As the span increases beyond 30m the 99.7% confidence level statistically analyzed load effects converges to Class AA loading effects in general.
- Beyond 40m span length Mean x/ Standard Deviation3 (confidence level of 99.7%) is the governing load effects to produce maximum live load shear and bending moments.
- ASSHTO LRFD (HL-93) loading and NHA legal loads are not the governing load in any case from short to medium span simply supported bridges.
- Calibration Factor "r" based on 99.7% confidence interval using statistical distributions for AAHTO LRFD is 1.67. Means that the AASHTO live loads should be enhanced by 67% for the analysis and design of bridges in Pakistan.
- Calibration Factor "r" based on WIM peak considered loads for AAHTO LRFD is 2.45. Means that the AASHTO live loads should be enhanced by 145% for the analysis and design of bridges in Pakistan.
- Calibration Factor "r" based on 99.7% confidence interval using statistical distributions for PCPHB is 1.09. Means that the PCPHB live loads should be enhanced by 9% for the analysis and design of bridges in Pakistan.
- Calibration Factor "r" based on WIM peak considered loads for PCPHB is 1.56. Means that the PCPHB live loads should be enhanced by 56% for the analysis and design of bridges in Pakistan.

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