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Investigation of the Adequacy of Bridge Design Loads in Pakistan

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ABSTRACTS

Weight, configuration, and volume of traffic vary from country to country. But, in developing countries like Pakistan, bridges are designed based on codes of developed countries. Hence, these bridges may not have desired safety level. In this study, safety levels of three sample bridges has been investigated in terms of structural reliability index. Live load effects (shear and moments) in girders were determined using weigh-in-motion data (WIM) and were extrapolated to 75 years using non-parametric fit. Two live load models and two strengths, required by 1967 Pakistan Code of Practice for Highway Bridges (PHB Design-Case) and that required by the 2012 AASHTO LRFD Bridge Design Specifications (AASHTO Design-Case) were used in reliability analysis. It is found that actual trucks produce moment and shear in girders 11 to 45 percent higher than live load models of PHB and AASHTO design cases. Values of structural reliability indices vary from 1.25 to 2.50 and from 2.45 to 3.15 for PHB and AASHTO design cases, respectively, and are less than the target reliability index value of 3.50 used in the design codes as benchmark. It is revealed after the research that bridges in Pakistan may not have desired safety level, and current live load models may not be the true representation of service- level truck traffic.

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1. INTRODUCTION

Highway bridges are designed to safely carry heavy live loads (truck loading), which are expected to move over these bridges during the service life. Since future loads are not deterministic, present truck loading and its configuration is used to forecast loads and develop live load models that result in safe and rational design. In Pakistan, bridges are designed as per 1967 Pakistan Code of Practice for Highway Bridges 1967 (PHB Code) and 2012 AASHTO Load and Resistance Factor Design (LRFD) Bridge Specifications (ASHTO Specifications). Typically, bridge superstructure is designed using live load model of PHB Code or HL-93 live load model (AASHTO live load model) that was developed using truck data from the Ontario, Canada (Kayser and Nowak, 1989). Since, truck traffic varies significantly in axle weights, axle configuration, gross vehicle weights (GVW) and traffic volume from site to site, state to state and country to country, therefore, different states of US calibrated HL-93 live load model based on the state specific truck traffic (Chotickai and Bowman, 2006). Furthermore, extensive research and investigation for analysis of bridge live load has already been carried out in US, Canada and Europe by many researchers (Hwang and Nowak, 1991). Weigh in Motion (WIM) data was used as basis for such types of research. Statistical procedure for development of live load model (live load models consist of a combination of lane load and a truck load defined by axles pacing and axle weight in design codes) based on Ontario truck survey data was carried out by many researchers (Hwang and Nowak, 1991). Protocol for collecting data using WIM instruments/records in NCHRP project 12-76 has been reported. WIM based live load model for bridge reliability was also presented. Protocols for collecting and using traffic data in bridge design were published in National Cooperative Highway Research Program 683 under the supervision of transportation research board of US in 2011. But, in developing countries like Pakistan, live load models specified in design codes of developed countries are used without calibrating them as they are based on country specific truck traffic. Furthermore, in Pakistan, over the years, service-level truck traffic has changed significantly in axle weights and configuration, gross vehicle weight (GVW) and traffic volume due to developments in truck industry to meet the heavier loads carrying demands by various industries. Thus, use of live load models of PHB Code and AASHTO Specifications for designing of bridges in Pakistan needs detailed evaluation to ensure adequate safety level as these live load models may not be a true representation of service-level truck traffic of Pakistan. In Pakistan, no significant work has been carried out for live load modeling and bridge reliability. Comparative study of live loads for the design of highway bridges in Pakistan was carried out by Ali *et al.* (2012).

In this research, the live load data at different WIM stations were collected and their effects (moments / shear) were determined using Microsoft excel software. The actual live load effects were extrapolated using nonparametric fit to 75 years and statistical parameters i.e., mean / maximum values and co-efficient of variations were determined. Using these statistical parameters, values of structural reliability index were determined through first order reliability methods for all the selected bridges. Non parametric statistics is used to describe the behavior of the sample bridge incorporating extensive engineering judgment. After the analysis it is revealed that live load model of 1967 PHB code is not the true representation of existing truck traffic on roads. Bridges being designed using

1967 PHB code may not have desired safety level which may lead to rapid deterioration of bridges or in reducing their design life. Thus, there is a need to develop a new live load model for design of bridges in Pakistan or to enforce legal load limits.

2. LIVE LOAD MODELS USED FOR DESIGN OF BRIDGES IN PAKISTAN

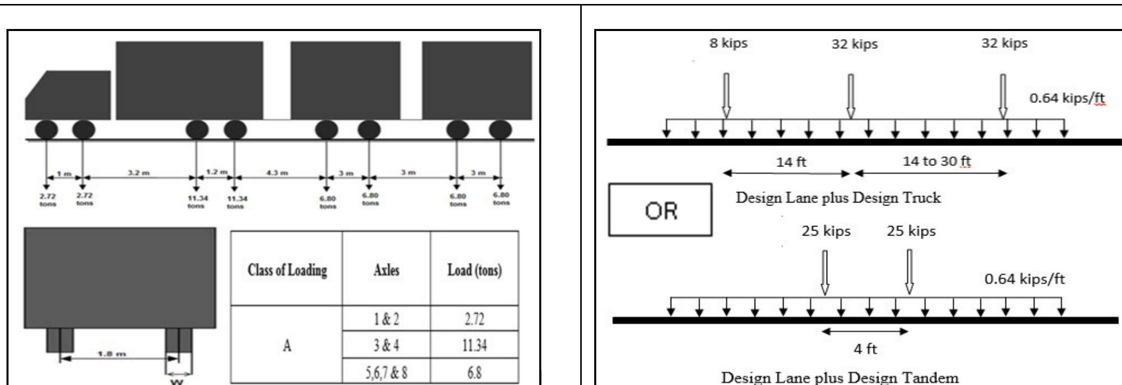
The highway loading on the bridge consists of a truck train loading and 70 ton military tank. In PHB code 1967, the design live loads are classified as Class A, Class B and Class AA loading. In this research PHB Class A and AASHTO HL-93 loading is considered as design trucks, the details of which are in the following.

2.1. Class A Loading (Standard Train Loading)

The Class A loading was proposed with the objective of covering the worst combination of axle load and axle spacing, likely to arise from the various types of vehicles that are normally expected to use the roads. This load train is reported to have been arrived at after an exhausted analysis of all Lorries made in all countries of the world. The loading consists of a train of wheel loads (8-axles) that is composed of a driving vehicle and two trailers of specified axle spacing and loads as shown in **Figure 1a**. This loading in bridge designing is generally adopted on all roads on which permanent bridges and culverts are constructed.

2.2 AASHTO LRFD Live Loading

AASHTO LRFD live loading commonly known as HL-93 loading was developed in 1993. AASHTO live load model, included in AASHTO Specifications was developed using truck data from the Ontario Ministry of Transportation, Canada. This is a hypothetical live load model proposed by AASHTO for the analysis of bridges with a design period of 75 years. HL-93 loading consists of three components: (1) Designed truck, (2) Designed tandem, and (3) Designed lane as shown in **Figure 1b**. Therefore the extreme load effects for the vehicular live load are the larger of the following: (1) The combined effect of design lane load and design truck with variable axle spacingor and (2) The combined effect of design lane load and designed tandem.



(a) Train Loading Class A (PHB CODE, 1967) – 8 Axle

(b) AASHTO Design Vehicle – HL-93

Figure 1. Live load models used for design of bridges in Pakistan.

3. WIM DATA AND ITS ANALYSIS

WIM is used for collecting the data pertaining to live load due to trucks on bridges. The information include the gross vehicle weight (GVW), axle spacing, axle weight, number of axles and average daily truck traffic (ADTT). WIM data was acquired in collaboration with National Highway Authority (NHA), Pakistan in the raw form. The same was filtered to get the data in required form and was used for analyzing the effects of live load on the sample bridges. Data was recorded at three locations; 1) Sangjani weigh station, 2) Mansoor weigh station and 3) Peshawar temporary weigh station. The filters can be used to screen the database for bad data or unlikely trucks during the data transfer process. Following guide lines given in National Cooperative Highway Research Program 683 were considered for filtering out the bad data: (a) Total number of axles ≥ 2, (b) Total number of axles ≤ 12, (c) Sum of axle spacing is greater than the length of truck, and (d) Sum of axle weight is greater than GVW of truck.

Maximum numbers of axle were restricted to 12 only with the reason that such trucks resulted in very high load effects. These high values are the representative of a special or permit vehicle. To achieve optimum reliability, special or permit trucks needs to be dealt separately. The detail of acquired WIM from three different stations is in the following.

3.1 Sangjani Weigh Station

Sangjani Weigh Station is located on National Highway 5 (N-5), Pakistan. Data acquired from Sangjani weigh station was recorded in later half of the year 2012. Six months of truck data was recorded. Total of about 273,399 trucks of different configuration were recorded during this period. Before processing, the data was filtered for errors in the recording by deleting the wrong or abnormal entries. A total of 42,656 (15.60%) trucks were removed after the application of filter on the raw data. The remaining numbers of trucks (230,743) were used for further analysis as shown in **Table 1**. Maximum entries comprised of trucks from 2 axles to 6 axles while few entries consists of above 6 axles trucks. Truck data up to 12 axles were included in the analysis.

Maximum GVW recorded at Sangjani from the filtered data is 163.4 tons and its corresponding configuration is 12 axles. Mean GVW for the data recorded at this site is 35.92 tons as shown in **Figure 2**. Mean GVW of this site is much lower as compared to the mean GVW of Ontario truck data which is 75 tons. Comparing between GVW of actual truck to GVW of design trucks (Class A and HL-93 trucks) is shown in **Figure 3**. Result indicates that 39.18 and 2.17% of GVW of actual trucks are higher than GVW of HL-93 and Class A design trucks, respectively.

Table 1. Number of vehicles and maximum GVW in each category – Sangjani.

	Truck Configuration (Number of Axles)											Total
	2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	
Number of Trucks	101022	114606	9282	1787	4014	13	5	8	3	2	1	**230743**
Max GVW (tons)	32.43	56.59	66.82	86.30	109.30	106.05	123.70	143.80	124.80	136.00	163.40	

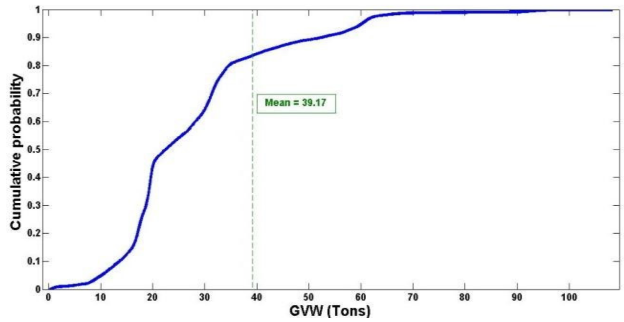


Figure 4. CDF plot of GVW at Mansoorweigh station.

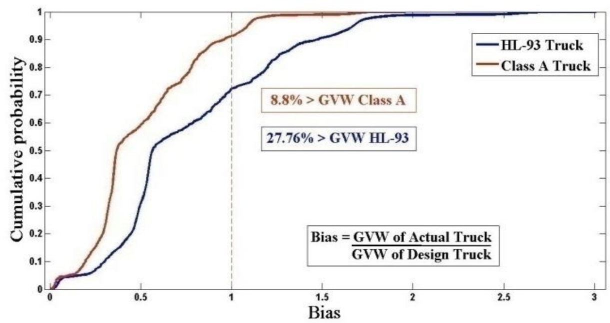


Figure 5. Bias of GVW of actual trucks in Mansoorweigh station.

3.2. MullanMansoor Weigh Station

Mullan Mansoor Weigh Station is located on N-5. Three months of truck data was recorded at this weigh station in 2009. Total of about 116,009 trucks of different configuration were recorded during this period. Unlike the data recorded at Sangjani, axle spacing was missing in the data files recorded at this site. For the missing axle spacing, standard axle spacing measured on ground at Peshawar by Ali *et al.* (2012) was applied for analysis. The truck configuration were up to 12 axles at Mullan Mansoor. A total of 11,456 (9.9 percent) trucks were removed after the application of filter on the raw data. Summary of number of vehicles as per axles is shown in **Table 2** and their maximum GVW is 108 .3 tons and its corresponding configuration is 6 axles.

3.3. Peshawar (Temporary Weigh Station)

A temporary weigh station was established at Hayatabad in Peshawar to monitor the truck traffic by researchers of UET Peshawar (Ali *et al.*, 2012) in collaboration with Peshawar Development Authority (PDA). Data acquired at this site was limited to very few trucks (411 trucks). The data includes the vehicles up to 12 axles only. Summary of number of vehicles as per axles and their max GVW is summarized in **Table 2** and maximum truck GVW recorded is about 88.12 tons and its corresponding configuration of truck is 6 axles.

Average GVW for the data recorded at this site is 37.35 tons as per the CDF of the GVW for Peshawar survey data and is shown in **Figure 6**. Comparison between GVW of actual truck to GVW of design trucks (Class A and HL-93 trucks) is shown in **Figure 7**. As per the comparison 42.58 and 37.71% of GVW of actual trucks are higher than GVW of HL-93 and Class A design trucks, respectively.

Table 2. Number of vehicles and maximum GVW in each category – Peshawar.

	Truck Configuration (Number of Axles)											Total
	2	3	4	5	6	7	8	9	10	11	12	
Number of Trucks	150	66	33	3	154	1	1	1	-	-	-	411
Max GVW (tons)	30.42	37	44.93	54.37	88.12	80.456	82.70	87.80	-	-	-	

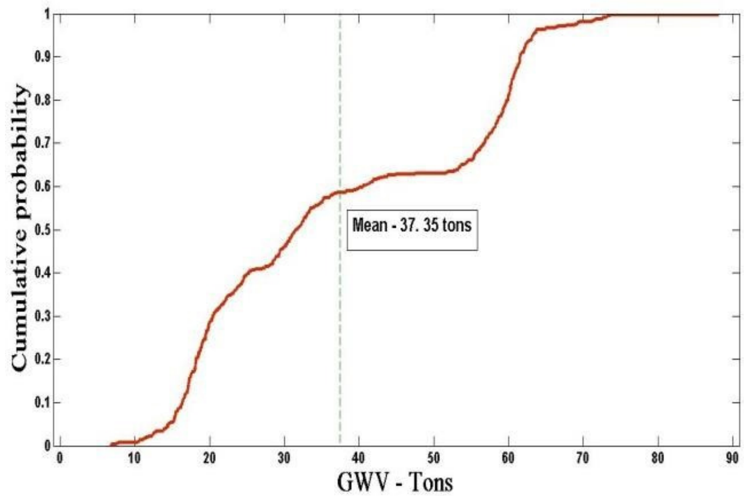


Figure 6. CDF plot of GVW at Peshawarweigh station.

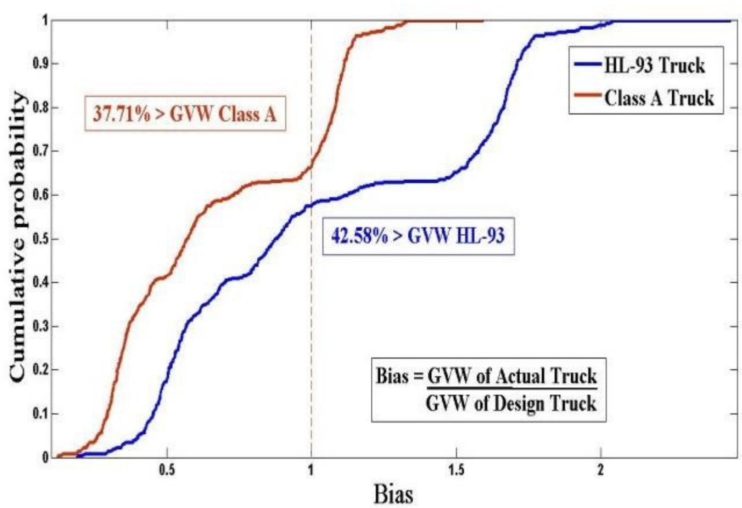


Figure 7. Bias of GVW at Peshawarweigh station.

4. Determination of Maximum Moment and Shear Using Influence Lines

For a simply supported bridge, calculation of load effects (moments and shear) involves determination position of loading on the beam. For calculating absolute maximum moments/shear for a large number of trucks, codes as per number of axles were developed in a computer program using MS Excel. Three sample bridges for each site were selected for Reliability analysis. All these Bridges are simply supported, pre-stressed concrete girder bridges and details are in the following:

(1) Muhammad Wala Bridge – Sangjani. Muhammad Wala Bridge was constructed in 2010. This bridge consists of pre-stressed and simply supported girder shaving a clear span of 47.2 m. Overall width of the bridge is 12.09 m and road way width is 12.05 m. It is a three lane bridge, having 180 mm deck thickness and 100 mm thick wearing surface and consists of four pre-stressed concrete girders.

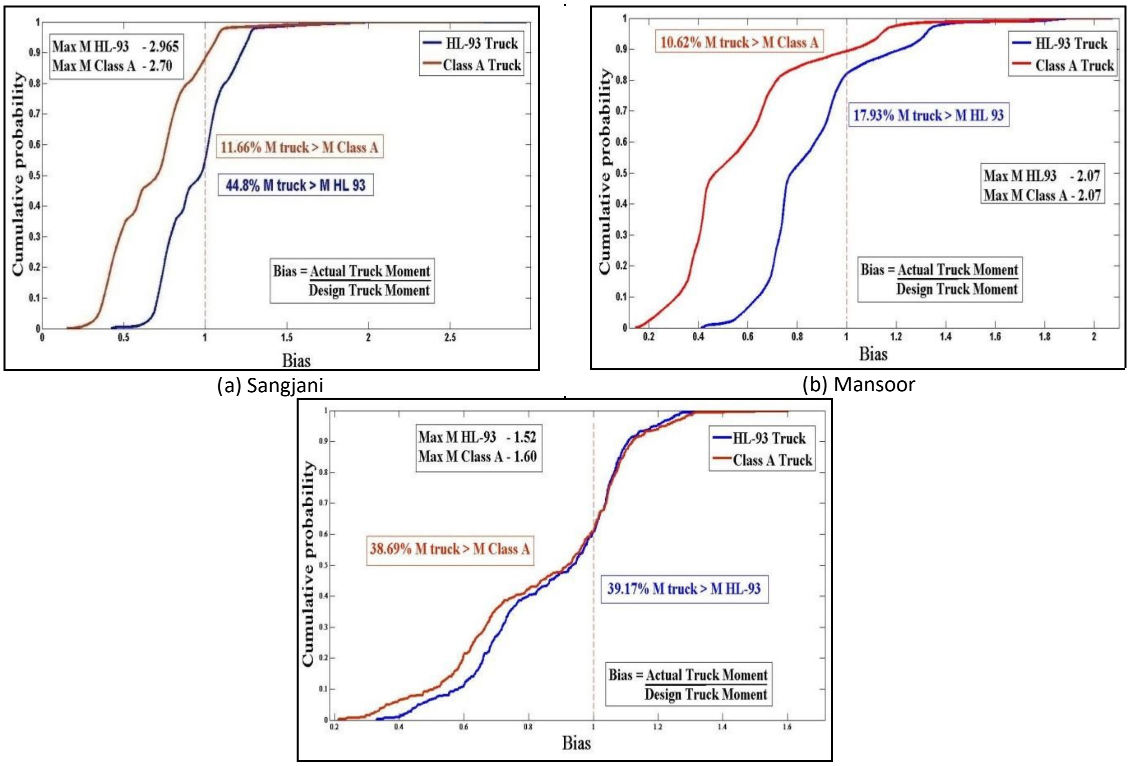
(2) Mansoor Bridge–MullanMansoor. Mansoor Bridge is identical to Muhammad Wala Bridge with a clear span of 47.19 m. This bridge was constructed in 2009. It consists of four pre-stressed girders having a span of 47.19 m and 3.03 m spacing between girders. Again it is a three lane bridge, having 180 mm deck thickness and 100 mm (average) thick wearing surface.

(3) Bagh-e-Naran Bridge – Peshawar. This is a 20 years old bridge having a clear span of 12.8 m. This bridge consists of pre-stressed and simply supported girders. Overall width of the bridge is 8.69 m and road way width is 7.39 m. It is a two lane bridge, having 190 mm deck thickness and 100 mm thick wearing surface. It consists of five pre-stressed concrete girders and spacing between each girder is 1.90 m.

4.1 Determination of Maximum Moment

Maximum moment was calculated using influence lines by running each actual recorded truck on the sample bridge at all three sites. Similarly, maximum moment was also calculated for HL-93 and Class A design truck. Normalized moments were calculated by dividing the actual truck moment with the moment of HL-93 and Class A design truck. The probability curve of Sangjani - bridge is shown in **Figure 8a** and results show that 44.80% and 11.66% of actual trucks produce moments higher than that produced by HL93 and Class A design trucks. Maximum value of moment is about 2.97 and 2.70 times higher than the moment produced by both the designed trucks, respectively.

17.93 and 10.62% of actual trucks produce moment higher than moments of HL-93 and Class A design trucks respectively, at Mullan Mansoor – bridge (**Figure 8b**). Similar increasing trend was observed at this bridge and maximum value of moment was in order of 2.07 times higher than the moment produced by both HL-93 and Class A truck. In case of Peshawar – bridge (**Figure 8c**), 39.17 and 38.69% of actual trucks produce moment higher than that produced by HL-93 and Class A design truck. Maximum value of moment was 152 and 160% higher than the moment produced by HL-93 and Class A truck, respectively. The analysis of moments clearly indicate a significant increasing actual traffic load as compared to design loads resulting in rapid deterioration of bridges.



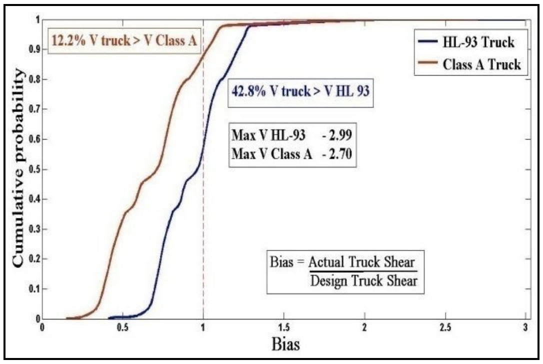
(a) Sangjani (b) Mansoor

(c) Peshawar

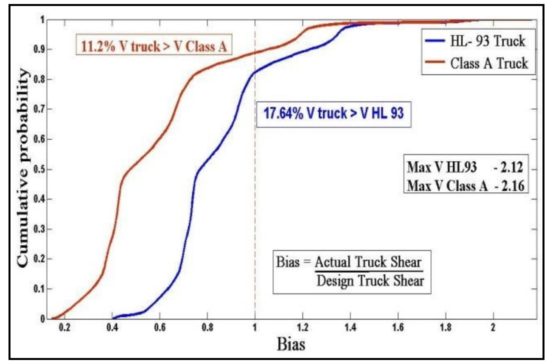
Figure 8. CDF plot of moments at different weighing stations.

4.2. Determination of Absolute Maximum Shear

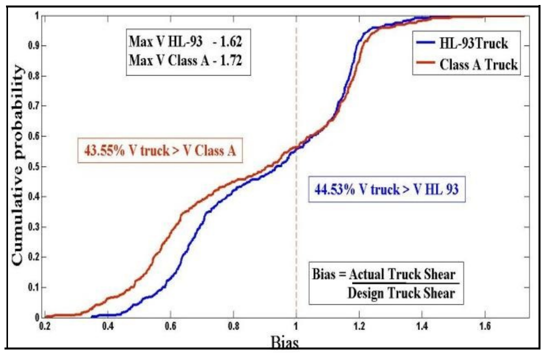
For determination of maximum shear, same procedure (as for moment calculation) was adopted for each truck. Normalized shear was calculated by dividing the truck shear with the design truck shear. Results at Sangjani - bridge (**Figure 9a**) indicate that 42.80 and 12.20% of actual trucks produce maximum shear higher than produced by HL-93 and Class A design trucks respectively. Maximum value of shear is 2.99 and 2.70 times higher than the shear produced by design trucks. Similar trend was observed at other sites. In case of Mansoor – bridge, 17.64 and 11.20% of actual trucks produce maximum shear higher than that produced by design trucks as shown in **Figure 9b**. Maximum value of shear was in the range of 212% higher than the shear produced by HL-93 and Class A truck, respectively. At Peshawar– bridge (**Figure 9c**), about 44% of actual trucks produce higher shear higher and maximum value of shear was about 170% higher than the moment produced by HL-93 and Class A truck, respectively.



(a)Sangjani



(a) Mansoor



(c) Peshawar

Figure 9. CDF plot of shear at different weighing stations.

5. EXTRAPOLATION OF LIVE LOAD EFFECTS TO 75 YEARS RETURN PERIOD

As per AASHTO LRFD code, moment and shear effects obtained from actual recorded truck data needs to be extrapolated to 75 years using statistical techniques for predicting the maximum value the bridge has to encounter over its design life period. Different techniques were used to extrapolate the value for data projection to 75 years. In this study, Nonparametric Fit method was used for predicting the mean maximum 75 years value. After calculating maximum load effects using influence lines, additional filter was applied: Normalized moment ($M_{truck}/M_{design\ truck}$) or normalized shear ($V_{truck}/V_{design\ truck}$) having values less than 0.15 were removed from the data as it has little or no effects on the bridge. Similarly ratios greater than 3 were also removed as these values are representative of special or permit vehicles which need to be dealt separately to avoid designing uneconomical bridges.

5.1. Maximum Mean Load Effects for 75 Years Return Period

ADTT is used to find the standard normal variable (*z*) on vertical axis of CDFs of moment and shear for different return period. ADTT for one day at Sangjani represents 1289 vehicle.

Corresponding probability is 1/1289 = 0.000775795 and its *z* value is 3.16. Similarly the data for two weeks represents 18,275 vehicles. Corresponding probability is 0.0000547 and *z* value is equal to 3.87. In the same way six months of truck traffic probability is equal to 4.33383 10^{-6} and standard normal variable is equal to 4.45.

Similar calculations for standard normal variable were done for other two bridges. For determination of probability and standard normal inverse for 75 year return period we assumed that no abrupt increase in the traffic volume occurs during the same period using available ADTT for six months as was done by earlier researchers. **Table 3** summarizes the different values of number of trucks N, probability $1/N$, and standard normal inverse z for 75 years return period for all the three bridges. Here, N is the number of vehicles, corresponding probability is $1/N$ and z is the standard normal inverse. Number of trucks for 75 years is calculated by multiplying 75 with number of trucks in one year as in the following : N_{75} = 75 x 461,486 = 34,611,450**.**

5.2. Determination of cov using kernel function

Extension of upper tail of CDF for moment and shear ratios was performed using nonparametric fit method. COV is calculated by dividing the standard deviation with mean of load effects of actual truck data:

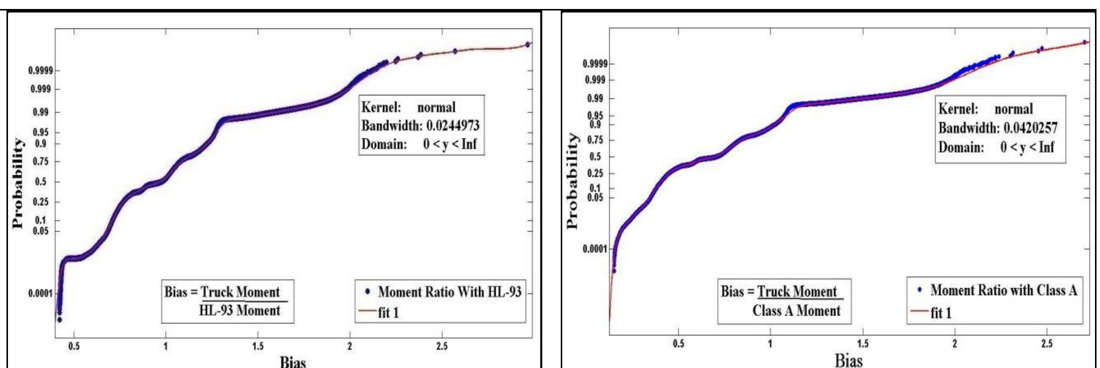
$$COV = \frac{\sigma_x}{\mu_x}\left(\frac{\text{standard deviation of actual load effect}}{\text{Mean value of actual load effect}}\right) \quad (1)$$

5.3. Nonparametric Fit for moments

Kernel function as normal and bandwidths as 0.0244973 and 0.0420257 resulted in the best fit to the CDF curve for Sangjani – bridge for normalized moment with HL-93 and Class A truck respectively as shown in **Figures 10a and b** respectively. **Figures 11a and b** show the best fit to the CDF curve using bandwidths 0.0222738 and 0.0450362 for normalized moments with HL-93 and class A design vehicle respectively for Mansoor - bridge. Similarly, best fit curve for normalized moments at Peshawar - bridge was obtained using Bandwidths 0.0789475 and 0.0925799 as shown in **Figures 12a and b**, respectively.

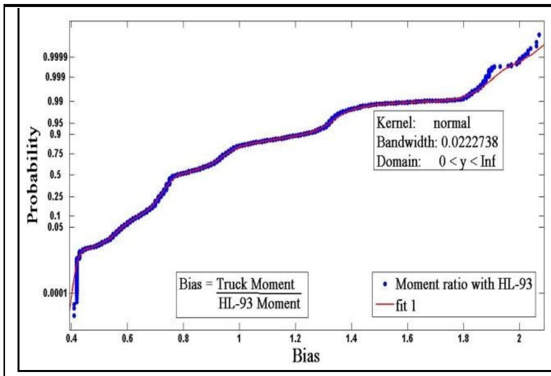
Table 3. Number of trucks with corresponding probability and time period.

Time Period 75 years	Number of Trucks N	Probability (1/N)	Standard Normal Inverse 'z'
Sangjani	34611450	2.88922E-08	5.43
Mansoor	28478400	3.51143E-08	5.39
Peshawar	101606400	9.8419E-09	5.61

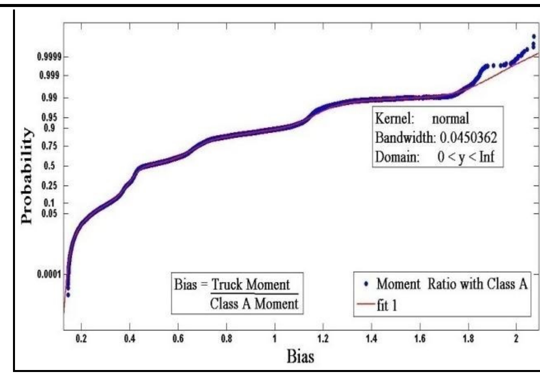


(a) Nonparametric fit– M_{HL-93} – Sangjani (b) Nonparametric fit – $M_{Class\ A}$- Sangjani

Figure 10. Data for Sangjani.

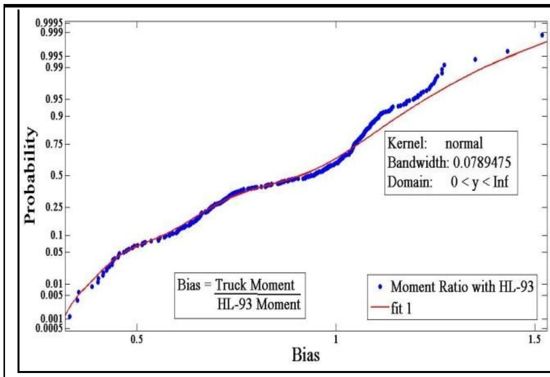


(a) Nonparametric fit to data – M_{HL93}-Mansoor

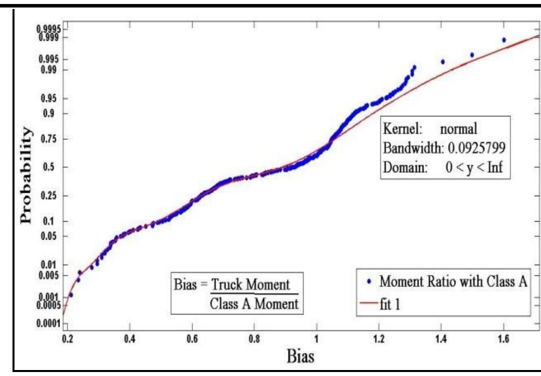


(b) Nonparametric fit to data – $M_{Class\ A}$ - Mansoor

Figure 11. Data for Mansoor.



(a) Non parametric fit – M_{HL-93} – Peshawar

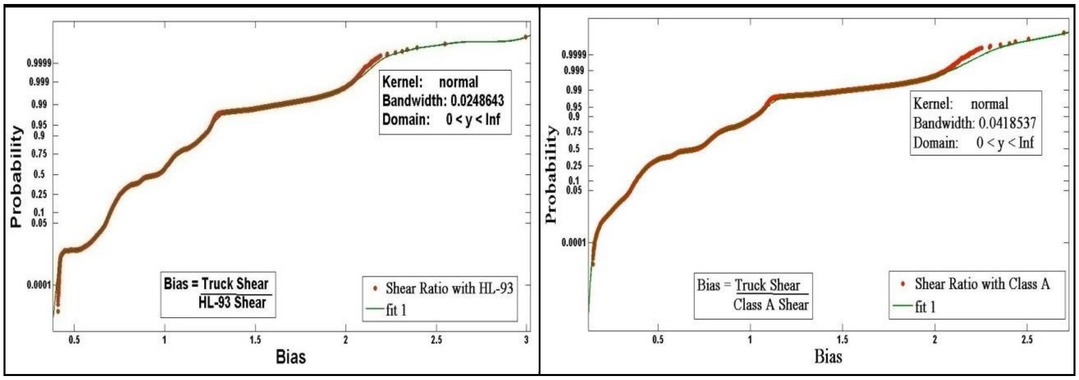


(b) Nonparametric fit – $M_{Class\ A}$- Peshawar

Figure 12. Data for Peshawar.

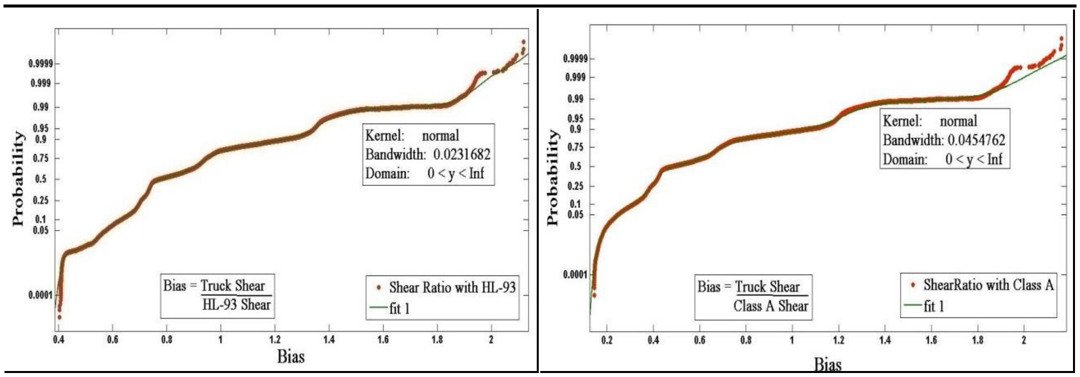
5.4. Nonparametric Fit for shear

Same procedure was applied for normalized shear to get the best fit using the nonparametric approach. Using Kernel function as normal and bandwidths as 0.0248643 and 0.0418537, it resulted in the best fit to the CDF curve for Sangjani - bridge for normalized shear with HL-93 and Class A truck respectively as shown in **Figures 13a and b**, respectively. **Figures 14a and b** show the best fit to the CDF curve using bandwidths 0.0231682 and 0.0454762 for normalized shear with HL-93 and class A design vehicle respectively for Mansoor - bridge. Similarly, best fit curve for normalized shear at Peshawar – bridge was obtained using bandwidths using Bandwidths 0.110299 and 0.133782, as shown in **Figures 15a and b**, respectively.



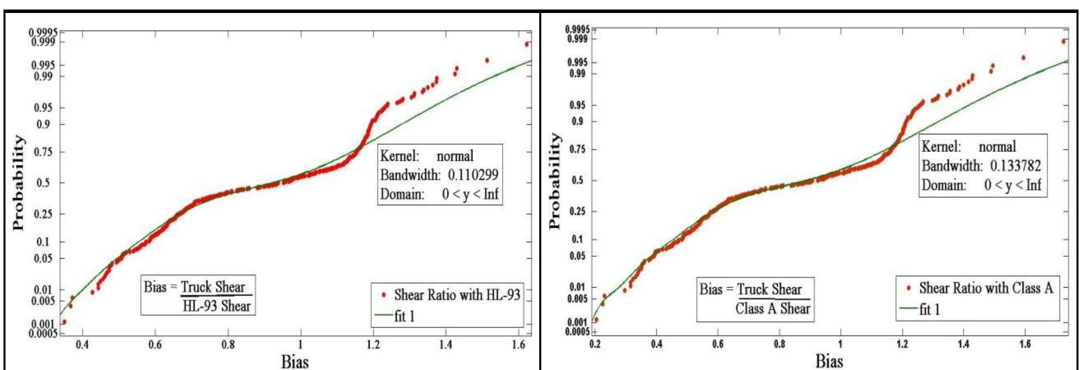
(a) Nonparametric fit – V_{HL-93}- Sangjani (b) Nonparametric fit – $V_{Class\ A}$-Sangjani

Figure 13. Data for Sangjani.



(a) Nonparametric fit to data–V_{HL-93}–Mansoor (b) Nonparametric fit to data–V_{ClassA}-Mans

Figure 14. Data for Mansoor.



(a) Nonparametric fit – V_{HL-93} – Peshawar (b) Nonparametric fit – $V_{Class\ A}$- Peshawar

Figure 15. Data for Peshawar.

6. EXTRAPOLATION TO 75 YEARS FOR MOMENTS

For Sangjani - bridge, mean maximum value of moment ratio corresponding to 75 years return period was obtained using the nonparametric fit having a *z* value of 5.43 (refer **Table 4**) is 3.15 and the COV is 0.22 for HL-93 truck. Mean value of maximum moments for Class A truck is equal to 3.002 and the COV is 0.39. In the case of extrapolated values of shear for HL-93 truck, the mean maximum shear is equal to 3.19 and the COV is 0.23, whereas for Class A truck, the mean maximum shear is 2.99 and COV is 0.39. Similarly, the mean maximum moment/shear for other two bridges is tabulated in **Table 4**.

7. RELIABILITY ANALYSIS OF LIVE LOADS EXTRAPOLATED TO 75 YEARS

Reliability analysis in code for buildings was proposed by Beck and Dória (2008). For bridges, code calibration for reliability analysis was proposed by Kayser and Nowak (1989) that used reliability models in bridge evaluation. Multiple presence was analyzed by many researchers (Tabsh and Nowak, 1991) that defined the bridge resistance as the max gross vehicle load that is causing the formation of a collapse mechanism. Hwang and Nowak (1991) added dynamic load induced by the vehicular load to the statistical model of live load. Other researcher presented protocol for collecting weigh-in-motion records. Probability of failure cannot be solved directly due to complexities involved in the calculations. Most suitable way of predicting probability of failure is based on the reliability index. Reliability can be defined as the "probability that unsatisfactory performance or failure will not occur. Probability that a system will perform its intended function for a specific period of time under a given set of conditions.

$$R = 1 - Pf \quad (2)$$

Reliability index is directly related to probability of failure as

$$\beta = -\Phi^{-1}(Pf) \quad (3)$$

where Φ^{-1} is the standard normal distribution function

$$Pf = (R - Q) < 0 \quad (4)$$

Both R and Q are modeled as Random variables and possess certain amount of uncertainties. Uncertainty in bridge reliability is related to truck weight, truck volume and truck type. The above equation shows that β is inversely related to Pf.

Table 4. Mean Maximum Moment and Shear for 75 years by Nonparametric fit

Stations	Moment/Shear	Recorded Data	75 Years	COV
Sangjani – Bridge	$M_{Truck}/M_{HL\text{-}93}$	2.96	3.15	0.22
	$M_{Truck}/M_{Class\ A}$	2.70	3.002	0.39
	$V_{Truck}/V_{HL\text{-}93}$	2.99	3.19	0.23
	$V_{Truck}/V_{Class\ A}$	2.70	2.99	0.39
Mansoor - Bridge	$M_{Truck}/M_{HL\text{-}93}$	2.07	2.21	0.27
	$M_{Tuck}/M_{Class\ A}$	2.07	2.39	0.48
	$V_{Truck}/V_{HL\text{-}93}$	2.12	2.27	0.28
	$V_{Truck}/V_{Class\ A}$	2.16	2.49	0.49
Peshawar - Bridge	$M_{Truck}/M_{HL\text{-}93}$	1.52	2.16	0.26
	$M_{Truck}/M_{Class\ A}$	1.60	2.42	0.31
	$V_{Truck}/V_{HL\text{-}93}$	1.62	2.65	0.29
	$V_{Truck}/V_{Class\ A}$	1.72	3.13	0.36

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7.1. Load Combinations

According to AASHTO LRFD code and PHB Code, types of loads on bridge structure can be divided into two main categories: permanent load and transient load. Simultaneous occurrence of dead load, live load and impact load forms as the basic combinations for design of highway bridges. In this study, effects of dead load, live load and impact load are considered. Each load components can be expressed in terms of random variable. Variation of these loads is defined by their statistical parameters and CDF.

(1) Dead load. Dead load includes the self-weight of the structural components (DC) and self-weight of wearing surface (DW). both structural and nonstructural components can be categorized as factory made elements (precast concrete embers)and cast in place concrete members. All these components have different degree of variation that is why each component is considered separately. All components of dead load are treated as normal random variable and are represented by the CDF taken as normal. **Table 5** describes statistical parameters of dead load which are based on previous research carried out in literature.

(2) Live load. Live load covers a range of forces produced by moving vehicles on the bridges.live load effects are influenced by a numbers of parameters which includes GVW, axle weight, number of axles, axle configuration, position of vehicle, bridge span, and the girder spacing and number. WIM data acquired at three sites (Sangjani, Peshawar, and Mansoor) was used to analyze the effects on sample bridges. Mean maximum 75 year's value calculated using nonparametric method was used for further analysis. Summary of results are in **Table 5**.

(3) Dynamic load. Dynamic load is the function of road surface roughness, vehicle suspension system (vehicle dynamics) and the frequency of vibration of the bridge (bridge dynamics). For maximum 75 years value for single truck the dynamic load does not exceeds 0.15 of live load, and 0.10 of live load for two side by side trucks. Contribution of these three parameters varies from site to site and is almost impossible to predict accurately. It was decided to specify dla (dynamic load allowance) as a constant percentage of live load. as per AASHTO LRFD code 1994, 33% of live load was taken as impact allowance. Similarly, impact factor allowance as described in PHB code was applied for the normalized values of shear and moment with class a vehicle. impact factor as given in PHB code 1967 was used for reliability analysis:

$$IM = 15 / (L + 20) \leq 30\% \quad (5)$$

Table 5. Statistical parameters of dead load.

No	Components	Bias Factor (Mean value/Nominal value)	COV
1	DC 1 (Factory made elements)	1.03	0.08
2	DC 2 (Cast in place concrete members)	1.05	0.1
3	DW 1 (Wearing Surface)	1.0 (3.5 inch mean)	0.25
4	DW 2 (Miscellaneous components)	1.03 - 1.05	0.08 – 0.1

where *L* is the length of bridge in feet. COV of impact factor is taken from previous research as 0.10.

7.2. GIRDER DISTRIBUTION FACTOR (GDF)

Accurate percentage of GDF is required to assess the live load effect on an individual girder. According to AASHTO LRFD code 1994, GDF for moment of interior girders for single lane of traffic is given as:

$$GDF_{MI} = 0.06 + (S/4300)^{0.4} \times (S/L)^{0.3} \times (K_g/Lt_s^3)^{0.1} \quad (6)$$

where, *S* is the beam spacing in mm with range of applicability, between 1100 and 4900 mm, *L* is the span length in mm, between 6000 and 73000 mm, t_s is the thickness of deck in mm, between 110 and 300 mm, K_g is the beam stiffness Parameters, Number of girders $\geq$ 4, *M* is the Moment, *I* is the interior Girder. Shear distribution factor for interior girder for single lane loaded is given in the following equation:

$$GDF_{VI} = 0.36 + S/7600 \quad (7)$$

Distribution factor given in West Pakistan Code for highway bridges 1967 is:

$$GDF = S / 5.5 \quad (8)$$

COV of GDF is also taken from previous research as 0.13.

7.3. RELIABILITY ANALYSIS

Target reliability index is 3.5. Target reliability index of 3.5 means approximately two failures in 10,000 events. Like the previous research, total load components have been distributed normally and the resistance components as log normally.

(1) Resistance model. load carrying capacity of bridge depends upon the resistance (*R*) of its components. resistance of these components is the function of material properties, section geometry and dimensions. all these functions are considered deterministic for design purpose but in reality some sort of uncertainties are associated with each function. therefore resistance is considered random variable. in reliability analysis, *r* is considered the product of nominal resistance (R_n), material factor (*M*), fabrication factor (*F*) and professional factor (*P*):

$$R = R_n \times M \times F \times P \quad (9)$$

where *M* is the parameters reflecting variation in strength of material, *F* is the variables reflecting uncertainties in dimensions, *P* is the professional factor, which accounts for uncertainty arising from the method of analysis used, and R_n is the nominal resistance specified by code.

Statistical parameters for the resistance in given as:

Mean of R, $\mu_R = R_n \times \mu_M \times \mu_F \times \mu_P$ (10)

Bias of R, $\lambda_R = \lambda_R \times \lambda_F \times \lambda_P$ (11)

COV of R, $V_R = \sqrt{(V_M)^2 + (V_F)^2 + (V_P)^2}$ (12)

where, μ_R, λ_R, and V_R are the mean, the bias, and the coefficient of variation of resistance, respectively. Statistical parameters of the material, fabrication and professional factors is determined from available literature and values are given in the **Table 6**.

(2) Reliability index (*β*). *β* in this study is based on the safety index calculation. *β* is defined as the function of probability of failure. equation 13 is used to calculate the reliability index as given in previous research:

$$\beta = \frac{R_n\lambda_R(1-KV_R)[1-ln(1-KV_R)]-\mu_Q}{\sqrt{\left\{[R_n\lambda_R V_R(1-KV_R)]^2+(\delta_Q)^2\right\}}} \quad (13)$$

where *K* is the measured shift from mean value in standard deviation units, assumed equal to 2, μ_Q is the mean of total load or load effects and 6_Q is the standard deviation of total load or load effects. The total factored force for design of girders as per AASHTO LRFD code and PHB Code is given in the following equation (For strength 1 limit state):

$$Q = \sum n_i j_i Q_i \leq \phi R_n \tag{14}$$

where n_i is the load modification factor. j_i, Qi, and ϕ are the load factor, the force effect and the resistance factor, respectively. For strength 1, limit state equation can be written as

$$Q\ (AASHTO) = 1.25\ (DC_1 + DC_2) + 1.5\ DW + 1.75(LL + IM) \tag{15}$$

For pre-stressed design the equation as given in PHB code 1967 is

$$Q\ (PHB\ Code) = 1.5\ DL + 2.5\ (LL + IM) \tag{16}$$

Reliability index (β) for pre-stressed girders only has been calculated considering simply supported sample bridges using above equation and for both the codes separately. The reliability index determined for all the three bridges is summarized in **Table 7**. Target β was 3.50 as was done during the calibration of AASHTO LRFD code and same value was used in the previous research. 3. 50 value of reliability index means two failures in 10,000 events. After the analysis, it was found that reliability index values in all three bridges are lower than target reliability value of 3.50. Values are between 13 and 37% lower than target value for all the cases, which shows that the bridges do not have desired safety level. Values of β estimated based on AASHTO live load and resistance model are greater than those estimated based on PHB code 1967 live load and resistance model, which shows that AASHTO code as a whole is more conservative than PHB code 1967. The value of β for moment is greater than shear for same set of loadings and provisions.

Table 6. Statistical parameters of material, fabrication, professional and resistance.

Type of structure		Material and fabrication factor F and M		Professional factor P		Resistance R	
		λ_F	V_F	λ_P	V_P	λ_R	V_R
Pre-Stressed Concrete Girders	Moment	1.04	0.045	1.01	0.06	1.05	0.075
	Shear	1.07	0.1	1.075	0.1	1.15	0.14

Table 7. Reliability index β – moment.

Bridge	Span	Truck data	Design vehicle	β for Moments (75 Years)	β for Shear (75 Years)
Sangjani – bridge	47.2	Sangjani	HL-93	3.04	2.75
			Class A	2.43	2.32
Mansoor – bridge	47	Mansoor	HL-93	2.83	2.59
			Class A	2.24	2.20
Peshawar - bridge	12.8	Peshawar	HL-93	2.74	2.51
			Class A	2.5	2.24

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8. CONCLUSION

Conclusions are in the following

(1) Actual truck traffic of Pakistan is significantly different in axle weights, axle configuration and GVW than the values specified by NHA Legal Load limits. Load effects caused by actual truck traffic are much higher than those caused by live load models of PHB Code and AASHTO Specification. The average calculated moments were from 10.62 to 44.8% higher as compared to design truck moments. Whereas, the maximum value of moment was about 1.52 to 2.96 times higher than the moment produced by HL-93 and Class A truck.

(2) Similar, trend was observed in case of shear and the calculated shear of actual trucks at all the three sites were found significantly higher than the design truck shear. The average calculated shear values were from 11.20 to 42.8% higher as compared to design truck shear values. Whereas, the maximum value of shear was from 1.62 to 2.99 times higher than the shear produced by HL-93 and Class A truck.

(3) After carrying out the reliability analysis it was found that reliability index β value for both the cases (Moment and Shear) was less than the target reliability index of 3.50 for all the three bridges.

Moreover, β value for shear was found lower than the moment for same set of conditions and bridges and β value using PHB code 1967 were also lower than the AASHTO LRFD code provisions and live load model. Safety index β (an alternative way of judging the safety of structures) is considerably below the target value for both shear and moment. Hence, existing live load model of PHB Code and NHA legal limits are not the true representations of actual truck traffic of Pakistan. Existing code provisions of both PHB 1967 and AASHTO may not be adequate for safe and economical designing.

9. THE AUTHOR'S NOTES

The authors declare that there is no conflict of interest regarding the publication of this article. Authors confirmed that the data and the paper are free of plagiarism.

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