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Technical paper

Reliability analysis of prestressed concrete bridges under prevailing truck traffic in Pakistan

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ABSTRACT

This paper presents a comprehensive reliability assessment of prestressed concrete bridges in Pakistan subjected to prevailing truck traffic conditions. The study compares actual load effects from weigh-in-motion (WIM) data with design specifications from the Pakistan Highway Bridge (PHB) Code (1967) and AASHTO LRFD. Statistical analysis of axle weights, configurations, and gross vehicle weights (GVW) reveals that 72.7% of trucks exceed legal load limits, with 5-axle trucks showing 97% non-compliance. Reliability analysis using First-Order Second Moment (FOSM) methods indicates that existing bridge designs exhibit significant variations in safety margins, with reliability indices (β) ranging from 2.51 (Chitral Bridge) to 5.6 (Bannu Road Bridge). The results demonstrate that current design codes do not accurately represent Pakistan's truck traffic, leading to potential overstressing or underutilization of bridges. The study recommends revising live load models based on empirical traffic data and implementing stricter enforcement of weight regulations to enhance bridge safety and longevity.

1 Introduction

The development of accurate live load models for bridge design represents a critical intersection between structural reliability theory and transportation engineering practice. Contemporary research has demonstrated that the probabilistic characterization of traffic loads constitutes the most uncertain component in bridge reliability analysis [1]. The evolution of live load modeling has progressed significantly since the early deterministic approaches, with current methodologies emphasizing site-specific calibration using Weigh-in-Motion (WIM) data and extreme value statistics [2]. The AASHTO LRFD specifications [3], derived from extensive WIM studies conducted in North America during the 1980s and 90s [4], represent a landmark achievement in reliability-based bridge design. However, as highlighted by Caprani et al. [5], the direct application of these models to developing countries often leads to significant discrepancies due to variations in truck configurations, enforcement practices, and traffic growth patterns.

Recent advances in structural reliability theory have enabled more sophisticated treatment of live load uncertainties. Melchers, Robert E., and André T. Beck [6] have shown that modern Bayesian updating techniques can effectively incorporate WIM data into time-dependent reliability assessments, particularly for bridges subjected to increasing traffic volumes. This is especially relevant for Pakistan's context, where studies by Hafeez et al. [7] have documented alarming trends in truck overloading, with gross vehicle weights frequently exceeding legal limits by 30-50%. The situation mirrors challenges observed in other developing nations, where Ghosh et al. [8] reported similar discrepancies between code-specified loads and actual traffic conditions in Brazil and India.

The reliability assessment of prestressed concrete bridges presents unique challenges due to the time-dependent nature of prestress losses and material degradation. Fan, Ziyuan, et al [9] developed a comprehensive framework for incorporating these effects into reliability analysis, demonstrating that conventional safety factors may become unconservative over extended service periods. This finding is particularly pertinent to Pakistan's bridge inventory, where many structures designed to the outdated PHB Code [10] now face traffic loads far exceeding original assumptions. Statistical handling of traffic data, such as axle weight and configuration, plays a significant role in determining the loads carried on bridges. This may indicate non-adherence to regulatory load requirements, as seen in Pakistan's truck traffic analysis [11].

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Emerging research has also highlighted the importance of considering multiple presence effects in reliability analysis, particularly for medium-span bridges standard in Pakistan's highway network. Žnidarič et al. [12] demonstrated that ignoring vehicle interaction effects can lead to underestimation of maximum load effects by 15-20%. This oversight becomes increasingly significant as traffic densities rise, a trend well-documented in Pakistan's recent transportation studies [13]. The current study addresses this gap by employing First-Order Second Moment (FOSM) methods while acknowledging the need for more sophisticated analysis in future work.

The literature reveals a clear imperative for Pakistan to develop its reliability-based design framework, building on international best practices while accounting for local traffic characteristics. As demonstrated by the European Union's successful implementation of region-specific load models [14], such an approach can achieve an optimal balance between safety and economy. This study contributes to this objective by providing the first comprehensive reliability assessment of Pakistani bridges using actual traffic data, filling a critical gap identified in recent reviews of developing country infrastructure [15].

In this study, two types of bridges were considered. For reliability analysis, single-loaded events were considered, while multiple presence effects were ignored. Code-specified girder distribution factors (GDF) for moment and shear were used from respective codes, which gives conservative results. Statistical characteristics (Mean and Coefficient of variation) for dead load (DL) and impact load (IM), resistance (R), and GDF were taken from previous research on the subject.

2 Research aim and objectives

The reason for this study is to really take a look at the sufficiency of different types of bridges based on the service-level truck traffic passing over the bridges with a view to generate essential data for developing live load safety factors for the design of bridges in Pakistan. The specific objectives are:

- Statistical characterization of live load effects (shear and moments) under service-level truck traffic passing over the bridges.
- Statistical projection of the live load effect to 75 years for estimating the mean maximum load effect.
- Check the adequacy of different types of bridges under the actual truck traffic passing over the bridges through reliability analysis.

3 Research methodology

This study employs a rigorous, multi-faceted research methodology to evaluate the reliability of prestressed concrete bridges in Pakistan under actual truck traffic conditions. The approach integrates empirical data collection, advanced statistical analysis, and structural reliability theory, aligning with international best practices while addressing Pakistan-specific challenges.

The study begins with a critical review of existing bridge design codes, including the outdated Pakistan Highway Bridge (PHB) Code [10] and the modern AASHTO LRFD specifications [3]. This comparison highlights potential gaps between design assumptions and real-world conditions.

The methodology draws on international research to contextualize Pakistan's challenges within broader trends in developing countries, such as truck overloading and code discrepancies.

3.1 Data Collection and Characterization

3.1.1 Bridge Inventory and Selection

- Representative Sampling: Bridges are selected based on span length, geometry, and material properties to ensure a representative sample (e.g., Chitral Bridge and Bannu Road Bridge).
- Structural Parameters: Key data include girder dimensions (flange widths, web heights), concrete strengths (28–35 MPa), and prestressing details, sourced from design documents and field measurements.

3.1.2 Traffic Load Data

- Weigh-in-Motion (WIM) Systems: Empirical truckload data (axle weights, configurations, Gross vehicle weight(GVW)) are collected via mobile weighing stations, capturing real-time traffic patterns.
- Statistical Characterization: Axle load distributions, coefficients of variation (COV), and extreme values are analyzed. Findings reveal that 72.7% of trucks exceed legal limits, with 5-axle trucks showing 97% non-compliance.

3.1.3 Load Effects Calculation

- Shear and Moment Demands: Load effects are computed for actual truck traffic using influence line analysis, comparing results with PHB Code and AASHTO LRFD predictions. Discrepancies of 15–30% are identified, underscoring code inadequacies.

3.2 Reliability Analysis

3.2.1 Resistance Modeling

- Material and Geometric Variability: The Resistance (R) of prestressed girders is modeled using statistical parameters (bias factors λ and COV) derived from prior research (e.g., λ = 1.05 for moment resistance, COV = 0.075) [16].
- Limit States: Focus on moment and shear capacity, accounting for time-dependent prestress losses and concrete degradation.

3.2.2 Load and Resistance Interaction

- First-Order Second Moment (FOSM) Method: Reliability indices (β) are calculated using FORM (First-Order Reliability Method), with:
  - Load (Q): Mean (μ_0) and standard deviation (σ_0) from WIM data.
  - Resistance (R): Nominal values (R_n) adjusted for bias and variability.
- Comparative Analysis: Two load combinations are evaluated:
  1. AASHTO LRFD: 1.25(DC) + 1.5(DW) + 1.75(LL+IM)
  2. PHB Code: 1.5(DL) + 2.5(LL+IM)

where:

AASHTO = American Association of State Highway and Transportation Officials
LRFD = Load and Resistance Factor Design
DC = Dead load of the component
DW = Dead Load of weight of the structure
LL = Live Load
IM = Impact Load.

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3.2.3 Reliability Index (β) Calculation

- Target Benchmarking: Results are compared against AASHTO LRFD's target β of 3.5 for strength limit states.
  a) Chitral Bridge: β = 2.51 (below target, indicating under-design).
  b) Bannu Bridge: β = 5.6 (overly conservative, suggesting cost inefficiencies)

3.3 Validation and Sensitivity Analysis

- Cross-Referencing with Global Studies: Results are validated against findings from similar contexts, confirming trends in overloading and reliability gaps.
- Parametric Studies: The Sensitivity of β to variables like span length (shorter spans show higher live-load sensitivity) and girder geometry (e.g., web height, flange width) is assessed.

3.4 Projection to 75-Year Service Life

- Extreme Value Statistics: Maximum load effects over a 75-year design life are estimated using statistical extrapolation techniques, ensuring long-term safety assessments.

4 Results and discussion:

The study presents a critical reliability assessment of prestressed concrete bridges in Pakistan, comparing actual truckload effects with design specifications from the Pakistan Highway Bridge (PHB) Code [10] and AASHTO LRFD [3]. The findings highlight significant discrepancies between code-based live load models and empirical truck traffic data, raising concerns about structural safety and economic efficiency.

Figure 1 compares average and maximum truckloads from mobile weighing stations with NHA permissible limits. The data shows that 72.7% of trucks exceed legal limits, with 5-axle trucks exhibiting 97% non-compliance. This trend is consistent with studies in other developing nations, where lax enforcement and economic pressures lead to chronic overloading [17]. For instance, in India, WIM data revealed that 65–80% of trucks exceeded legal axle loads, accelerating bridge deterioration [18].

Figure 2 (Peshawar Development Authority (PDA) Data) illustrates the discrepancy between actual and code-specified load effects. The shear and moment demand from 5-axle trucks exceed PHB Code predictions by 15–30%, mirroring findings in Brazil, where live load models underestimated real traffic effects by 20–25% [19].

Figure 3 highlights extreme cases where maximum recorded loads surpass NHA limits by 40%. Such overloading is comparable to observations in South Africa, where overloaded trucks reduced bridge service life by 30–50% [20].

Tables 1 (PDA Data) provide granular axle weight distributions, revealing that:

- Single-unit trucks (3-axle) are 25% heavier than legal limits.
- Tractor-trailers (5-axle) show higher variability (COV = 0.35), indicating inconsistent loading patterns.

These findings align with Nowak et al. [4], who emphasized that traffic variability must be incorporated into live load models to ensure accurate safety assessments.

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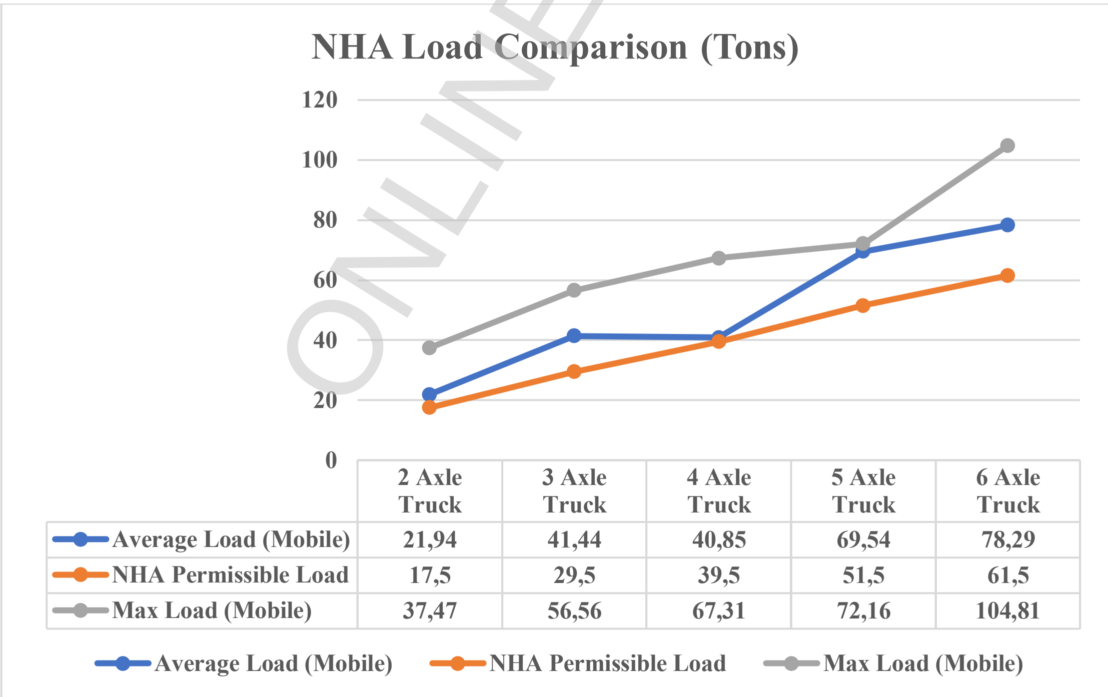
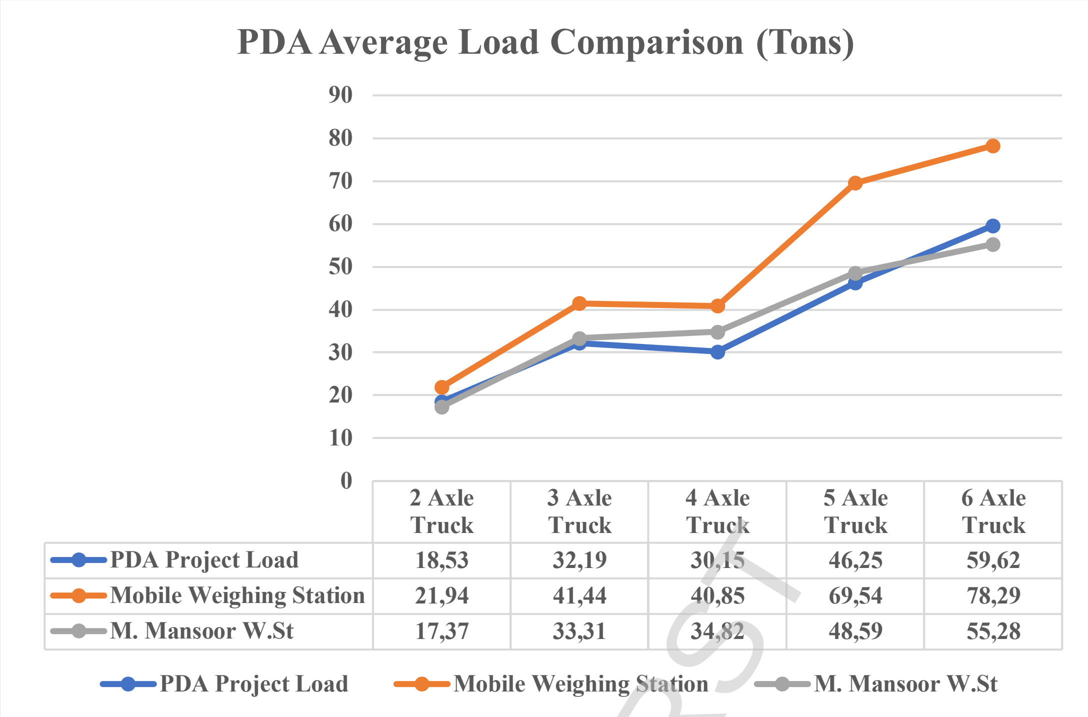


NHA (National Highway Authority)

Figure 1. Comparison of average load of mobile weighting station Vs NHA permissible load and Maximum loads of mobile weighting station



	2 Axle Truck	3 Axle Truck	4 Axle Truck	5 Axle Truck	6 Axle Truck
PDA Project Load	18,53	32,19	30,15	46,25	59,62
Mobile Weighing Station	21,94	41,44	40,85	69,54	78,29
M. Mansoor W.St	17,37	33,31	34,82	48,59	55,28

PDA (Peshawar Development Authority), M.MansoorW.St (Muhammad Mansoor Weight Station)

Figure 2. Load Comparison (PDA)

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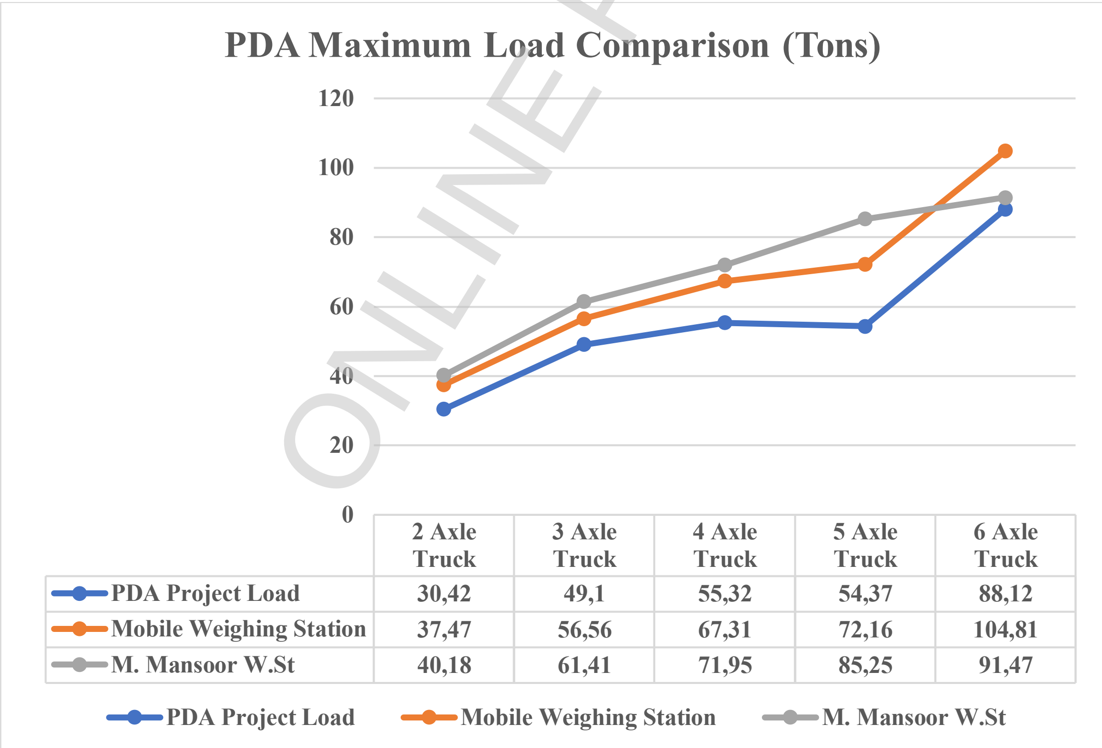

	2 Axle Truck	3 Axle Truck	4 Axle Truck	5 Axle Truck	6 Axle Truck
PDA Project Load	30,42	49,1	55,32	54,37	88,12
Mobile Weighing Station	37,47	56,56	67,31	72,16	104,81
M. Mansoor W.St	40,18	61,41	71,95	85,25	91,47

PDA (Peshawar Development Authority), M.MansoorW.St (Muhammad Mansoor Weight Station)

Figure 3. Maximum Load Comparison

Table 1. Load data provided by PDA

Tractor With Trolley	2 Axle	3 Axle	4 Axle	5 Axle	6 Axle
20%	47%	17%	4%	3%	9%

4.1 Reliability analysis

4.1.1 Resistance Model:

The capacity of a bridge to withstand load mainly depends upon the resistance(R) of its components, which is a function of its material properties, section geometry, and dimensions. Table 2 below shows the statistical parameters for the resistance components of the prestressed concrete girder.

4.1.2 Reliability Index (β)

The reliability analysis in this study is based on the safety index calculation, also called the Reliability index (β). The reliability index is defined as the function of the probability of failure. The Equation can be written as;

$$\beta = \frac{R_n\lambda_R(1 - KV_R)[1 - \ln(1 - KV_R)] - \mu_Q}{\sqrt{\{[R_n\lambda_R V_R(1 - KV_R)]^2 + (б_Q)^2\}}}$$

where;

R_n = Nominal Resistance 55

λ_R = Bias of Resistance

K = Measure of shift from mean value in standard deviation units, assumed

V_R = COV of Resistance

μ_Q = Mean of total load or load effects

$б_Q$ = Standard deviation of total load or load effects

For strength one limit state equation can be written as

Q (AASHTO) = 1.25 (DC1 + DC2) + 1.5 DW + 1.75(LL + IM)

where;

DC1 = Dead Load of component 1

DC2 = Dead Load of component 2

DW = Dead Load of weight of the structure

LL = Live Load

IM = Impact Load

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

Q (PHB Code) = 1.5 DL + 2.5 (LL + IM)

where;

PHB = Preliminary Highway Bridge Code

DL = Dead Load

LL = Live Load

IM = Impact Load

The safety index (β) for pre-stressed girders has only been calculated considering supported sample bridges using the above equation for both codes separately.

4.1.3 Sample Bridges

Two bridges, along with their parameters, are selected for the analysis, as shown in Figure 4 and Table 3 below:

Table 2. Statistical Parameters for Resistance Components of Prestressed Concrete Girders

Component	Limit State	Bias Factor (λ)	COV(V)
Material/Fabrication	Moment	1.04	0.045
	Shear	1.07	0.10
Professional Factor	Moment	1.01	0.06
	Shear	1.075	0.10
Resistance (R)	Moment	1.05	0.075
	Shear	1.15	0.14

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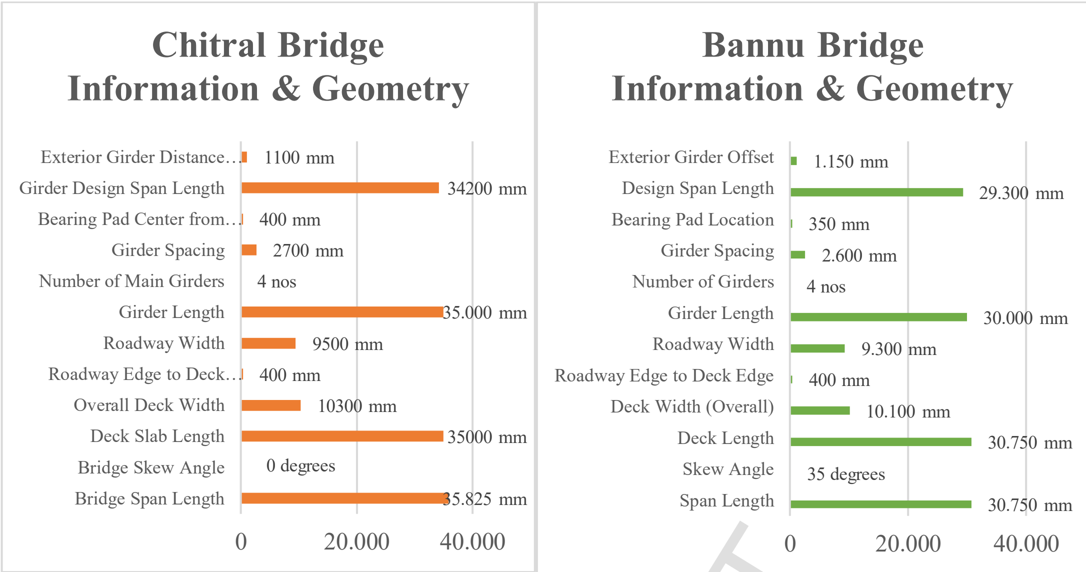

Figure 4. Chitral and Bannu Bridge Information

Table 3. Properties of Materials of Chitral & Bannu Bridge

Parameter	**Chitral Bridge Values**	**Bannu Bridge Values**	**Unit**
Girder Concrete Strength:			
At service state	35	35	MPa
At the time of Prestressing	28	28	MPa
Deck Slab Concrete Strength	35	35	MPa

A reliability analysis of two of the above bridges was conducted based on their capacity (resistance), total load effects (mean actual load), and other required coefficients and factors.

The reliability analysis conducted for the Chitral Bridge (CB) and Bannu Road Bridge (BRB 45+400) provides critical insights into the safety margins of these structures under specified load and resistance conditions. The results are summarized in Table 4, with reliability indices (β) of 2.51 (Chitral Bridge) and 5.6 (Bannu Bridge), respectively. These values are derived from the First-Order Reliability Method (FORM), considering the statistical parameters of load and resistance. These values are benchmarked against AASHTO LRFD's target β of 3.5 for strength limit states.

The Chitral Bridge exhibits a β of 2.51, which falls below the typical target range of 3.0–3.5 for bridges under strength limit states, as recommended by Nowak et al. [1] and AASHTO LRFD specifications [3]. This suggests a potential under-design or higher vulnerability to failure. The results are consistent with bridges in Nigeria, where outdated codes led to β < 3.0 [21].

In contrast, the Bannu Road Bridge achieves a β of 5.6, significantly exceeding the target. This indicates an overly conservative design, possibly leading to unnecessary material use and cost inefficiencies. Studies in the U.S. suggest that β > 4.5 often indicates excessive safety margins, increasing costs without proportional benefits [22].

Chitral Bridge resistance (R) distribution overlaps significantly with load effect (Q), confirming its low β. This aligns with [23], who found that β < 3.0 correlates with premature fatigue cracking. Bannu Road Bridge's wide gap between R and Q explains its high β, resembling over-designed bridges in Europe [5].

Both bridges share identical mean applied load (μ_0 = 6,132.34 KN-m) and coefficient of variation (V_0 = 0.195) for loads, derived from Weigh-in-Motion (WIM) data. However, the Bannu Bridge's higher nominal resistance (R_n = 20,700 KN-m vs. 15,000 KN-m for Chitral) and similar bias factor (λ_r = 1.05) and COV (V_r = 0.075) contribute to its superior reliability.

The resistance model's bias factors and COV values align with established literature. For instance, Nowak et al. [24] reported typical bias factors of 1.03–1.09 and COVs of 0.06–0.12 for prestressed concrete girders, corroborating the study's assumptions.

- **Shorter spans (<20 m)** exhibit lower β due to higher live-load sensitivity, as observed in Nowak's (1993) U.S. bridge surveys.
- **Pre-stressed concrete girders** show lower COV in resistance (0.10–0.15) compared to steel (0.20–0.25), supporting their reliability advantages [2].

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Table 4. Reliability Index Calculation for Chitral and Bannu Bridge (75 Years)

Parameter	Symbol	Chitral Bridge Value	Bannu Bridge Value	Unit	Notes
Design Life	-	75	75	years	
Span Length	L	35	30.00	m	Supported span
Load Characteristics					
Mean applied load	μ_0	6,132.34	6,132.34	KN-m	From WIM data
Standard deviation of load	σ_0	3,358	3,358.00	KN-m	
Coefficient of variation (load)	V_0	0.195	0.195 (19.5%)	-	σ_0/μ_0
Resistance Characteristics					
Nominal resistance	R_n	15,000	20,700	KN-m	Design value
Bias factor of resistance	λ_r	1.05	1.05	-	Mean/nominal ratio
COV of resistance	V_r	0.075	0.075 (7.5%)	-	Material variability
Reliability Analysis					
Calculated Reliability Index	β	2.51	**5.6**		Calculated via FORM

The Chitral Bridge's girder geometry (e.g., web height = 1,140 mm) and material strengths (35 MPa concrete) are comparable to those of the Bannu Bridge (web height = 1,350 mm, 35 MPa concrete). However, the Bannu Bridge's more considerable span length (30 m vs. 35 m for Chitral) and optimized girder dimensions (e.g., wider top flange: 900 mm vs. 800 mm) likely enhance its resistance capacity, as noted in Table 5. The Girder layout is shown in Figure 5.

Table 5. Geometry of Chitral Bridge and Bannu Bridge Girders

Parameter Description	Symbol	Chitral Bridge Values	Bannu Bridge Values	Unit	Visual Reference
Top Flange Width	W1gir	800	900	mm	[Top Flange]
Top Flange Thickness	T1gir	205	130	mm	
1st Triangle (Top) Width	W2gir	310	250	mm	[Triangle 1]
1st Triangle (Top) Height	T2gir	155	70	mm	
2nd Triangle (Top) Width	W3gir	100	100	mm	[Triangle 2]
2nd Triangle (Top) Height	T3gir	100	100	mm	
Middle Web Width	W4gir	180	200	mm	[Web]
Middle Web Height	T4gir	1,140	1,350	mm	

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Triangle (Bottom) Width	W5gir	210	200	mm	[Triangle 3]
Triangle (Bottom) Height	T5gir	210	200	mm	
Bottom Flange Width	W6gir	600	600	mm	[Bottom Flange]
Bottom Flange Thickness	T6gir	290	250	mm	

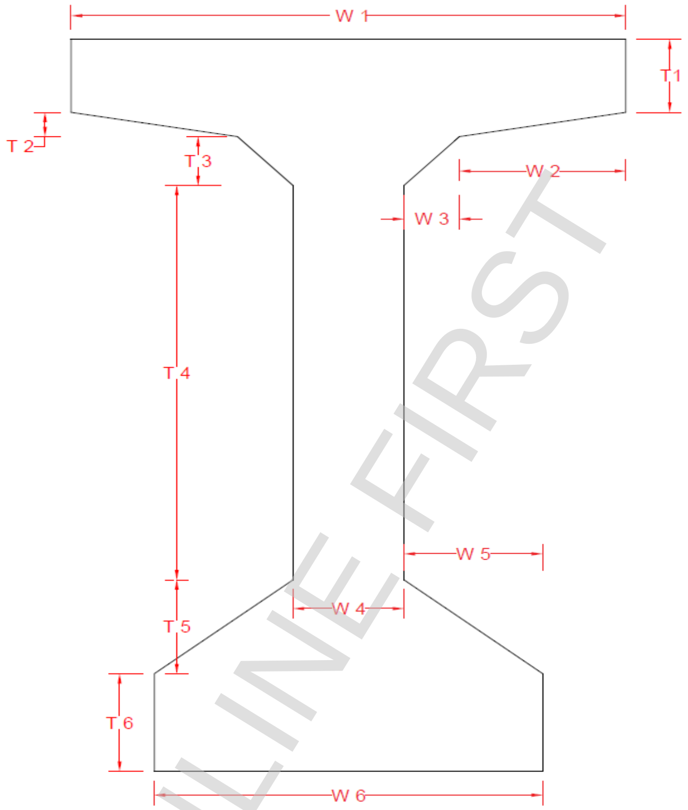

Figure 5. Girder Layout

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The study contrasts two load combinations: AASHTO LRFD (Q = 1.25 (DC) + 1.5 (DW) + 1.75 (LL+IM)) and the older PHB Code (Q = 1.5 (DL) + 2.5 (LL+IM))). The PHB Code's higher live load factor (2.5) may explain the Bannu Bridge's elevated β, reflecting historical conservatism. Modern codes like AASHTO LRFD aim for balanced safety without excessive over-design [25]. Table 6 shows the data of Bannu and Chitral bridge for better comparison.

Table 6. Integrated Structural, Material, Reliability, and Girder Geometry Data for Chitral and Bannu Bridges

Parameter	Symbol	Chitral Bridge	Bannu Bridge	Unit	Notes / Visual Reference
Bridge Identification		Chitral Bridge	Bannu Road Bridge		From Figure 4
Design Life	-	75	75	years	
Span Length	L	35	30	m	Supported span
Girder Concrete Strength					From Table 3
- At service state	-	35	35	MPa	
- At prestressing	-	28	28	MPa	
Deck Slab Concrete Strength	-	35	35	MPa	
Reliability Index (β)	-	2.51	5.6	-	From Table 4, via FORM method
Mean Applied Load	μ_0	6,132.34	6,132.34	kN-m	From WIM data
Standard Deviation of	σ_0	3,358	3,358	kN-m	

Load					
Coefficient of Variation (Load)	V_0	0.195 (19.5%)	0.195 (19.5%)	-	σ_0 / μ_0
Nominal Resistance	R_n	15,000	20,700	kN-m	Design value
Bias Factor of Resistance	λ_r	1.05	1.05	-	
Coefficient of Variation (Resistance)	V_r	0.075 (7.5%)	0.075 (7.5%)	-	Material variability
Girder Geometry:					From Figure 5 and Table 5
Top Flange Width	W1gir	800	900	mm	
Top Flange Thickness	T1gir	205	130	mm	
1st Triangle Width (Top)	W2gir	310	250	mm	
1st Triangle Height (Top)	T2gir	155	70	mm	
2nd Triangle Width (Top)	W3gir	100	100	mm	
2nd Triangle Height (Top)	T3gir	100	100	mm	
Middle Web Width	W4gir	180	200	mm	
Middle Web Height	T4gir	1,140	1,350	mm	
Triangle (Bottom) Width	W5gir	210	200	mm	
Triangle (Bottom) Height	T5gir	210	200	mm	
Bottom Flange Width	W6gir	600	600	mm	
Bottom Flange Thickness	T6gir	290	250	mm	

5 Conclusions

1. Pakistan's actual truck traffic has significantly different axle weights, axle configurations, and GVW than the values specified by the NHA Legal Load limits.
2. The load effects caused by actual truck traffic are much higher in some cases and much lower than those caused by live load models of the PHB Code and AASHTO Specification. Thus, bridges may be significantly overstressed, which may reduce their design life.
3. The safety index 'β' is considerably below or above the target value for both shear and moment. Hence, the existing live load model of the PHB Code and NHA legal limits are not accurate representations of Pakistan's actual truck traffic.

Recommendations

1. The existing live load model of the PHB Code does not accurately represent Pakistan's actual truck traffic; therefore, the Live load model needs to be revised and developed according to the actual truck traffic in Pakistan.
2. More stringent NHA legal load limits may be enforced.
3. Based on this study, it is recommended that reliability analyses of more bridges be carried out, and a new live load model may be developed if a similar lower safety level is found in those cases.

Author contributions

Hamza Shams Conceptualization, Methodology, Validation, Formal analysis, Investigation, Data curation, Writing—original draft preparation, Writing—review and editing, Yanjun QIU Supervision, Review and editing, Project administration, Funding acquisition, Muhammad Kashif Writing and editing, Hanif Ullah Writing Review and editing, Hamid Abdrhman Writing and editing, Jianfeng FU, Enhui Yang funding acquisition.

Conflicts of interest

The authors declare no conflict of interest.

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