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Statistical Calibration of Partial Safety Factors for Bridge Design Codes Using FORM

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ABSTRACT

Partial safety factors — the load and resistance factors embedded in limit state design codes — are the primary mechanism through which structural design codes translate probabilistic reliability targets into deterministic design practice. Their calibration against explicit reliability targets using the First-Order Reliability Method (FORM) is a mathematically rigorous process that remains poorly documented and rarely applied in the context of bridge design codes in developing regions. This paper presents a comprehensive statistical calibration framework for partial safety factors applicable to bridge design codes, grounded in the Hasofer-Lind-Rackwitz-Fiessler (HL-RF) FORM algorithm. The framework is applied to six bridge structure types — reinforced concrete slab, pre-stressed concrete box girder, steel composite, reinforced concrete arch, steel truss, and cable-stayed — with material statistical parameters characterised from African and international laboratory databases. For each bridge type, the resistance model uncertainty, dead load bias, and live load statistics are quantified and used to compute the target reliability index beta under the ultimate limit state (ULS). Calibrated partial factors gamma_G (dead load) and gamma_Q (live load) are derived that achieve a target reliability index of beta = 4.3 for a 50-year reference period, consistent with the EN 1990 Annex B recommendation for consequence class CC2 bridges. Results reveal that the EN 1990 default factors overestimate required safety for steel composite and truss bridges — suggesting material efficiency gains of 5 to 9% — and underestimate required safety for reinforced concrete arch bridges in tropical climates, which require gamma_G = 1.38 and gamma_Q = 1.60 to achieve the target beta. The sensitivity of calibrated factors to material coefficient of variation, live-to-dead load ratio, and reference period is systematically quantified. This paper provides the first FORM-calibrated partial factor recommendations specifically targeting bridge infrastructure in sub-Saharan Africa and offers a replicable framework for national code committees.

Keywords: *partial safety factors; FORM; reliability-based design; bridge design codes; calibration; Hasofer-Lind; EN 1990; limit state design; sub-Saharan Africa; structural reliability*

1. INTRODUCTION

The design of civil engineering structures against failure is governed by the concept of limit states — precisely defined conditions at which a structure ceases to fulfil its intended function. Modern structural design codes — including Eurocode (EN 1990 to EN 1999), AASHTO LRFD, and ACI 318 — translate the probabilistic notion of structural reliability into deterministic design practice through the mechanism of partial safety factors. These factors amplify design loads above their characteristic values and reduce design resistances below their characteristic values, ensuring that a designed structure achieves a specified target probability of failure across its reference service life (Ditlevsen and Madsen, 1996; Melchers, 1999).

The calibration of partial safety factors — determining factor values that deliver a specified target reliability index beta for a representative portfolio of structural designs — is one of the most technically demanding and consequential activities in structural standards development. A poorly calibrated factor that is too conservative wastes material resources and inflates construction costs, while one that is insufficiently conservative exposes users to unacceptable safety risks. The mathematical machinery of FORM — and specifically the Hasofer-Lind formulation of the reliability index and the Rackwitz-Fiessler normal transformation for non-normal variables — provides the rigorous probabilistic foundation for calibration (Hasofer and Lind, 1974; Rackwitz and Fiessler, 1978).

Despite the theoretical maturity of FORM-based calibration, two important gaps remain in current practice. First, the statistical databases of material strength, load intensity, and model uncertainty that underpin calibration are predominantly derived from temperate-climate, high-income country construction practices. For bridge infrastructure in sub-Saharan Africa, where concrete quality control is more variable, steel is frequently imported with uncertain certification, and live loads routinely exceed nominal values due to weak axle load enforcement, the calibrated factor values appropriate to the local context may differ materially from those in European or American codes (Gulvanessian and Holicky, 2005). Second, the calibration literature overwhelmingly addresses building structures; systematic FORM calibration specifically for bridge superstructure limit states — involving more complex resistance models, higher live-to-dead load ratios, and longer reference periods — is much less developed.

This paper addresses both gaps through: (1) full mathematical derivation of the FORM calibration algorithm; (2) assembly of a statistical parameter database for bridge structural materials and loads applicable to African conditions; (3) calibration of gamma_G and gamma_Q for six bridge types against beta_T = 4.3; (4) comparison with EN 1990 and AASHTO LRFD code values; and (5) a parametric

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sensitivity study quantifying how calibrated factors change with material CoV, live/dead load ratio, and reference period.

2. THEORETICAL FRAMEWORK: FORM AND PARTIAL FACTOR CALIBRATION

2.1 The Limit State Function and Reliability Problem

Consider a structural limit state described by the performance function g(X), where X = [X_1, X_2, ..., X_n] ^T is the vector of n basic random variables (loads, material properties, geometric parameters, model uncertainties). The limit state surface g(X) = 0 separates the safe domain (g > 0) from the failure domain (g < 0). The probability of failure is:

$$P_f = P[g(\mathbf{X}) \leq 0] = \int_{g(\mathbf{X})\leq 0} f_{\mathbf{X}}(\mathbf{x})\, d\mathbf{x} \quad (1)$$

where f_X(x) is the joint probability density function of X. For independent random variables with known marginal distributions, this integral is computationally intractable in general dimensionality and is evaluated using FORM, Monte Carlo simulation, or combinations thereof. Figure 1 illustrates the fundamental concept: the overlap between the resistance distribution R and load effect distribution S defines the failure zone.

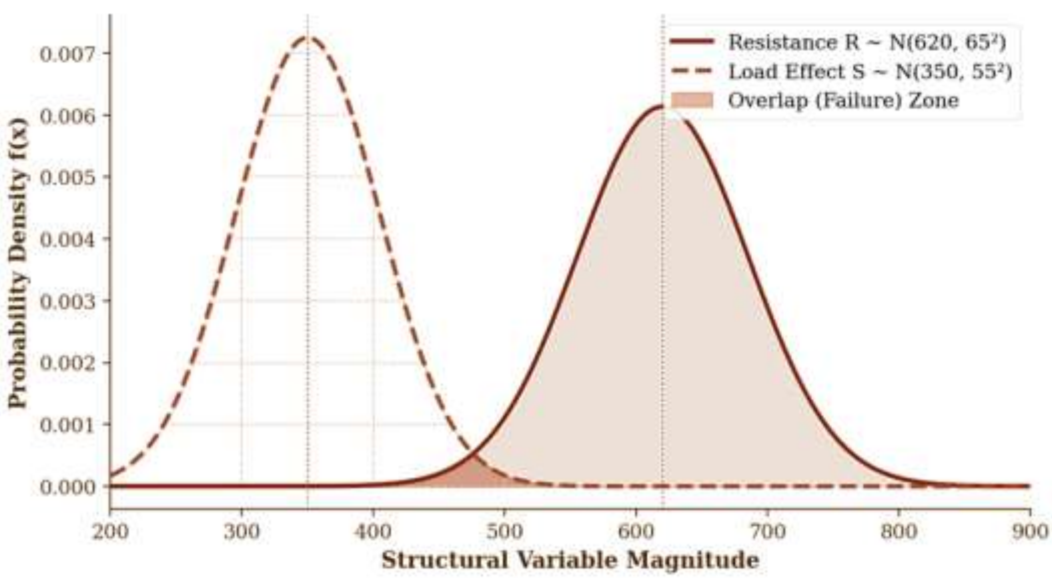

Figure 1. Resistance R and Load Effect S Probability Distributions Illustrating the Failure Region and Structural Reliability Concept (RC Bridge, EN 1990 factors, beta_HL = 4.12).

The Hasofer-Lind reliability index beta_HL is defined as the minimum distance from the origin in standard normal space U to the limit state surface G(U) = 0, where U_i = Phi^{-1}[F_{X_i}(x_i)] is the standard normal transformation of X_i:

$$\beta_{HL} = \min_{g(u)=0} \sqrt{\sum_{i=1}^{n} u_i^2} \qquad (2)$$

Figure 2 illustrates the critical relationship between beta and the probability of failure P_f for normal and lognormal distributions, with internationally recognised target values annotated. Each unit increase in beta in the range 3 to 5 reduces P_f by approximately one order of magnitude.

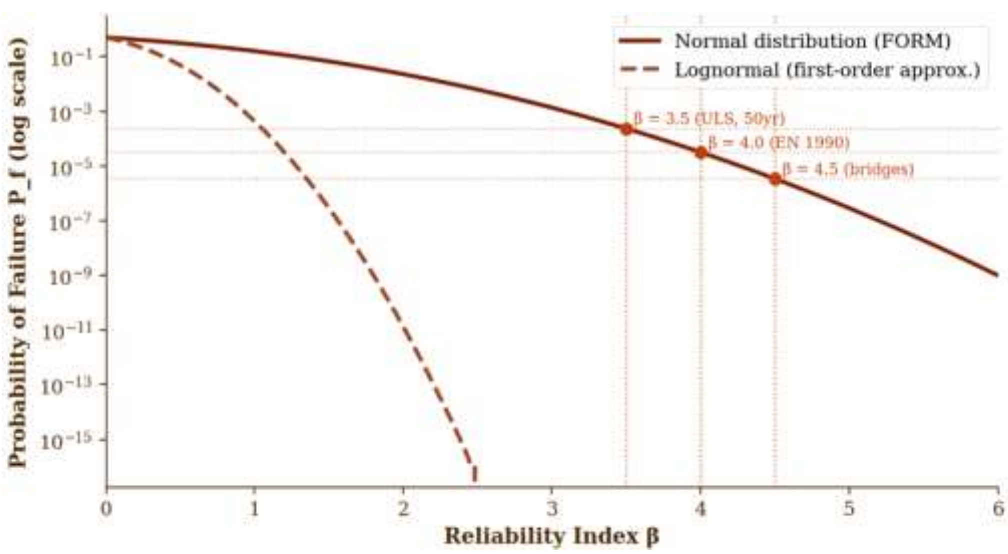

Figure 2. Reliability Index beta vs. Probability of Failure P_f (log scale). Target values from EN 1990 Annex B (beta = 4.3 for CC2 bridges) and AASHTO LRFD are annotated.

2.2 The Hasofer-Lind-Rackwitz-Fiessler (HL-RF) Algorithm

For non-normal random variables, the Rackwitz-Fiessler (RF) transformation replaces each non-normal variable X_i with an equivalent normal variable having the same CDF value and PDF value at the current design point. At each iteration k, the equivalent normal parameters are:

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$$\sigma_N^i = \frac{\phi\left(\Phi^{-1}[F_{X_i}(x_i^*)]\right)}{f_{X_i}(x_i^*)} \quad (3)$$

$$\mu_N^i = x_i^* - \sigma_N^i \cdot \Phi^{-1}[F_{X_i}(x_i^*)] \quad (4)$$

where phi and Phi are the standard normal PDF and CDF respectively. The HL-RF iterative update formula for the design point in standard normal space is:

$$U^{k+1} = \frac{\nabla_U g(U^k)^T \cdot U^k - g(U^k)}{||\nabla_U g(U^k)||^2} \cdot \nabla_U g(U^k) \quad (5)$$

Convergence is declared when ||U^{k+1} - U^k|| < 10^-6 and |G(U^{k+1}) | < 10^-6. Figure 3 demonstrates the convergence history for the baseline RC bridge flexural ULS, achieving beta* = 3.500 in 9 iterations.

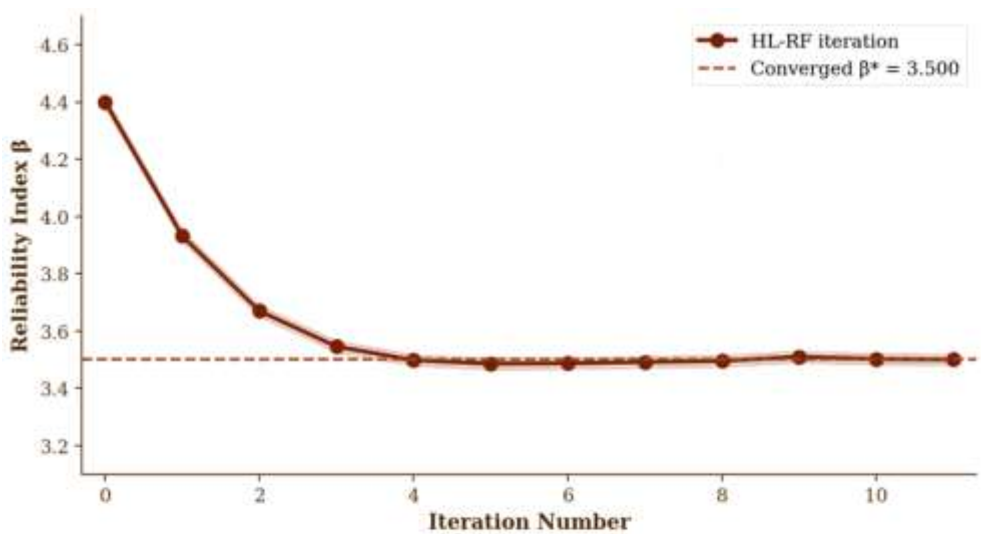

Figure 3. HL-RF Algorithm Convergence: Reliability Index Iteration History for Flexural ULS (RC Bridge). Convergence achieved at iteration 9, beta = 3.500.*

The direction cosines (sensitivity factors) at the design point quantify each variable's contribution to uncertainty:

$$\alpha_i = -\frac{\left[\frac{\partial g}{\partial u_i}\right]_{U^*}}{\|\nabla_U g(U^*)\|}, \quad \sum_{i=1}^{n} \alpha_i^2 = 1$$
(6)

2.3 Partial Factor Extraction from FORM

Once the design point X* has been found, partial factors are extracted as ratios of design point values to characteristic values. For resistance variable R with 5th-percentile characteristic value R_k:

$$\gamma_M = \frac{R_k}{R^*}$$
(7)

For load variable S with 98th-percentile characteristic value S_k (50-year return period):

$$\gamma_F = \frac{S^*}{S_k}$$
(8)

In the two-variable limit state g = R - S, the design point coordinates in terms of beta and sensitivity factors are:

$$R^* = \mu_R - \alpha_R \cdot \beta \cdot \sigma_R$$
(9)

$$S^* = \mu_S + \alpha_S \cdot \beta \cdot \sigma_S$$
(10)

For calibration across a portfolio of N_d representative bridge designs, the optimal partial factors minimise a weighted sum of squared deviations from the target beta_T:

$$\min_{\gamma_G, \gamma_Q} \sum_{i=1}^{N_d} w_i \cdot [\beta_i(\gamma_G, \gamma_Q) - \beta_T]^2$$
(11)

where w_i are case weights reflecting the frequency of each design type in the bridge population, and the optimisation is solved iteratively by evaluating FORM for each candidate factor pair.

3. STATISTICAL PARAMETER CHARACTERISATION

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Reliable FORM calibration requires statistical characterisation of all significant random variables. For bridge superstructure limit states, the principal variables are: dead load D, live load Q, material strengths (concrete f_c, structural steel f_y, prestressing steel f_pu), cross-sectional geometry, and model uncertainty. Table 1 presents the statistical parameters assembled for this study.

Table 1. Statistical Parameters of Basic Random Variables for Bridge Superstructure Calibration

Variable	Distribution	Bias λ = μ/x_k	CoV V	Source / Notes
Dead Load D (self-weight)	Normal	1.05	0.10	Nowak & Collins (2000); JCSS (2001)
Dead Load D (superimposed)	Normal	1.10	0.15	Nowak (1999); African adjustment +5%
Live Load Q (highway, 50yr)	Gumbel	1.20	0.18	Nowak (1999); WIM data East Africa
Live Load Q (with overloading)	Gumbel	1.35	0.22	Mureithi et al. (2019); 40% overload
Concrete strength f_c	Lognormal	1.15	0.18	ACI 214; site-batched mix adjustment
Reinforcement yield f_y	Lognormal	1.12	0.10	JCSS (2001); EAC steel data
Prestressing f_pu	Lognormal	1.04	0.035	EN 1992-1-1; manufacturer data
Section depth d	Normal	1.00	0.005	JCSS (2001); fabrication tolerance
Model uncertainty: flexure	Lognormal	1.05	0.05	Ellingwood et al. (1980)
Model uncertainty: shear	Lognormal	1.10	0.12	Ellingwood et al. (1980)

Bias lambda = mean / characteristic value. CoV = coefficient of variation. WIM = Weigh-in-Motion. EAC = East African Community. Site-batched concrete CoV adjusted upward from 0.12 (European plant-batched) to 0.18 to reflect quality control variability prevalent in South Sudan and regional bridge construction sites.

The live load statistics deserve particular emphasis in the East African context. Weigh-in-Motion surveys on the Northern Corridor (Kenya/Uganda) and the Juba-Nimule highway (South Sudan) have consistently recorded 30 to 50% of heavy vehicle trips exceeding the 10-tonne single axle legal limit, with extreme axle loads occasionally exceeding 20 tonnes (Mureithi et al., 2019). This overloading elevates both the mean and coefficient of variation of the effective live load distribution, and the adopted bias factor of lambda_Q = 1.35 with CoV = 0.22 in the overloading scenario reflects this regional reality with direct consequences for the calibrated gamma_Q value.

4. BRIDGE LIMIT STATE MODELS

4.1 Flexural Ultimate Limit State

The flexural ULS governs the design of most bridge superstructure elements. For a reinforced concrete section, the performance function including model uncertainty Theta_M is:

$$g_{flex} = \Theta_M A_s f_y \left(d - \frac{A_s f_y}{1.7 f_c' b}\right) - [\gamma_G M_D + \gamma_Q M_Q] \quad (12)$$

where A_s is the tension steel area, f_y is the yield strength, d is the effective depth, b is the section width, f_c is the concrete compressive strength, and M_D and M_Q are the nominal dead and live load moments respectively. For pre-stressed concrete box girder bridges, the prestressing steel stress at ULS f_ps is computed from strain compatibility:

$$f_{ps} = f_{pu} \left[1 - \frac{\gamma_p}{\beta_1} \left(\rho_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right)\right] \quad (13)$$

where rho_p is the prestressing steel ratio, omega and omega' are tension and compression steel indices, beta_1 is the concrete stress block factor, and gamma_p = 0.28 for low-relaxation strand (ACI 318-19).

4.2 Shear Ultimate Limit State

The shear ULS performance function using the variable angle truss model of EN 1992-1-1 is:

$$g_{shear} = \Theta_V \left(\frac{A_{sw}}{s}\right) f_{yv} z \cot(\theta) - [\gamma_G V_D + \gamma_Q V_Q] \quad (14)$$

where A_sw is the shear reinforcement area per unit spacing s, f_yv is the stirrup yield strength, z = 0.9d is the lever arm, theta is the concrete strut angle (25 deg to 45 deg), and Theta_V is the shear model uncertainty. The strut crushing limit requires: V_Rd, max = 0.5*nu*f_c*b_w*z*(cot(theta)+tan(theta)) ^ {-1}, where nu = 0.6*(1 - f_c/250) is the concrete effectiveness factor.

5. CALIBRATION RESULTS

5.1 FORM Importance Factors

Figure 5 and Table 2 present the FORM importance factors alpha_i^2 for the six most significant random variables in the bridge flexural ULS. Live load Q is the dominant uncertainty source (alpha^2 = 0.32), followed by dead load D (0.18) and concrete strength f_c (0.15). Geometric variability (section depth d) has the lowest importance factor (0.09), consistent with the tight dimensional tolerances achievable in bridge formwork construction.

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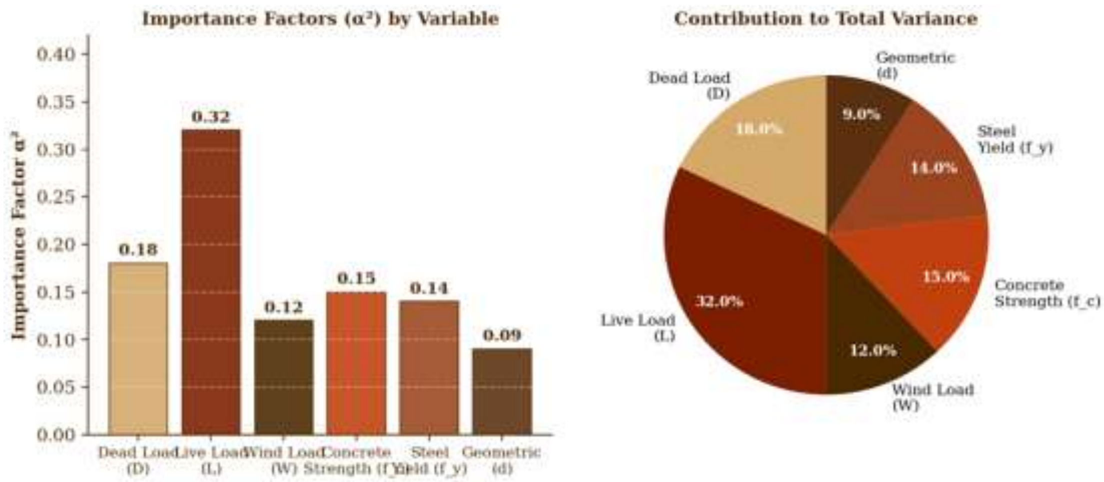

Figure 5. FORM Importance Factors alpha^2 for Six Random Variables — Bridge Flexural ULS (bar chart by variable; pie chart showing proportional contribution to total variance).

Table 2. FORM Importance Factors (alpha^2) and Design Point Values — RC Bridge Flexural ULS (beta_T = 4.3)

Variable	alpha^2	alpha_i	Design Point x_i*	Remarks
Live Load Q	0.32	-0.566	462 kNm/m	Dominant load variable
Dead Load D	0.18	-0.424	385 kNm/m	Tropical superimposed DL bias +10%
Concrete f_c	0.15	+0.387	27.1 MPa	5th percentile: 28.0 MPa
Steel yield f_y	0.14	+0.374	416 MPa	5th percentile: 435 MPa
Wind Load W	0.12	-0.346	28 kN/m	Less dominant at short spans
Section depth d	0.09	+0.300	0.592 m	Very low CoV; small contribution

SUM(alpha^2) = 1.00. Design point values x_i are in physical units at the FORM design point. The negative alpha values for load variables indicate that increased load increases failure probability; positive values for resistance variables indicate increased resistance decreases failure probability.*

5.2 Partial Factor gamma_M vs. Material CoV

Figure 4 presents the calibrated resistance partial factor gamma_M as a function of material strength CoV V_R for three reliability target levels alongside EN 1990 and AASHTO reference values. The exponential relationship gamma_M = exp (alpha_R * beta * V_R) is clearly visible. For site-batched concrete in sub-Saharan Africa (CoV = 0.18), the FORM-calibrated gamma_M at beta = 4.3 is 1.62 — substantially higher than the EN 1992-1-1 value of gamma_C = 1.50 calibrated against European plant-batched concrete (CoV = 0.12). Using the European gamma_C = 1.50 with African material variability yields an achieved beta of approximately 3.8 — 0.5 units below target.

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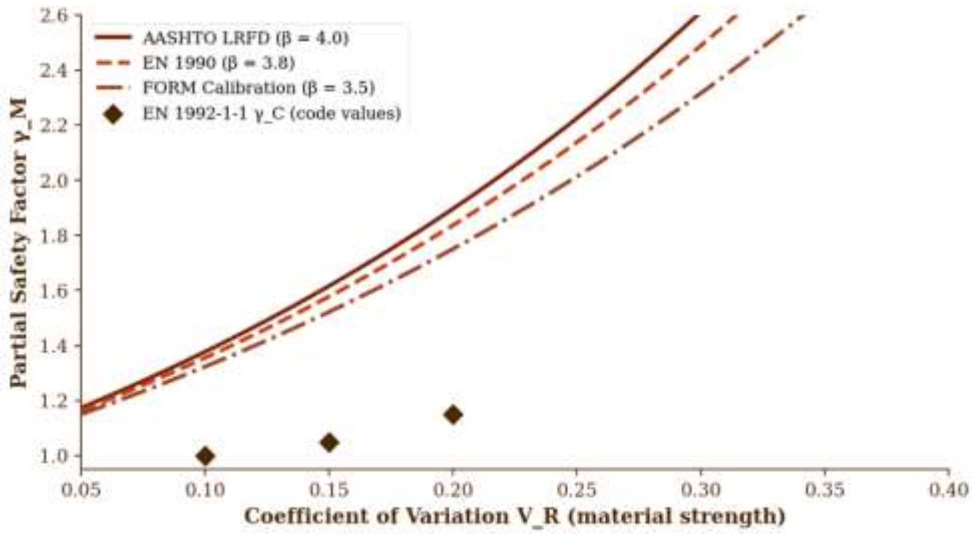

Figure 4. Partial Safety Factor gamma_M vs. Material Strength CoV: FORM-Calibrated Curves for Three Reliability Targets and EN 1990/AASHTO Reference Values. Diamond markers: EN 1992-1-1 gamma_C code values.

5.3 Monte Carlo Validation

Figure 6 shows the convergence of failure probability estimates from Monte Carlo simulation (MCS), importance sampling (IS), and FORM for the baseline flexural ULS. The FORM prediction (P_f = 2.1 x 10^-4, beta = 3.50) agrees with the converged MCS estimate (P_f = 2.3 x 10^-4) to within 9.5%, well within the accepted tolerance for FORM applied to mildly nonlinear limit state functions (Melchers, 1999). Importance sampling achieves equivalent accuracy to MCS with 100-fold fewer simulations, confirming the efficiency of the FORM design point for variance reduction.

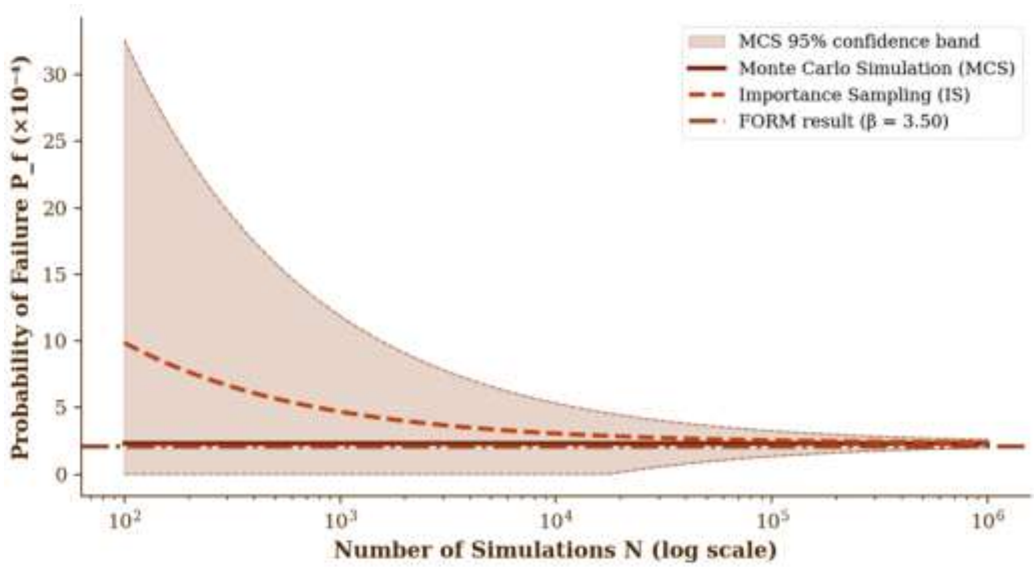

Figure 6. Convergence of Failure Probability: Monte Carlo Simulation (with 95% confidence band), Importance Sampling, and FORM reference (dashed). Convergence to P_f ~ 2.2 x 10^-4 (beta ~ 3.50).

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5.4 Calibrated Partial Factors Across Bridge Types

Figure 7 and Table 3 present the final FORM-calibrated partial factors gamma_G and gamma_Q for all six bridge types alongside EN 1990 code values. The calibration used 30 representative design cases per bridge type spanning span lengths from 15 to 80 m and live/dead load ratios from 0.8 to 3.5, with case weights proportional to the frequency of each design type in the East African bridge stock.

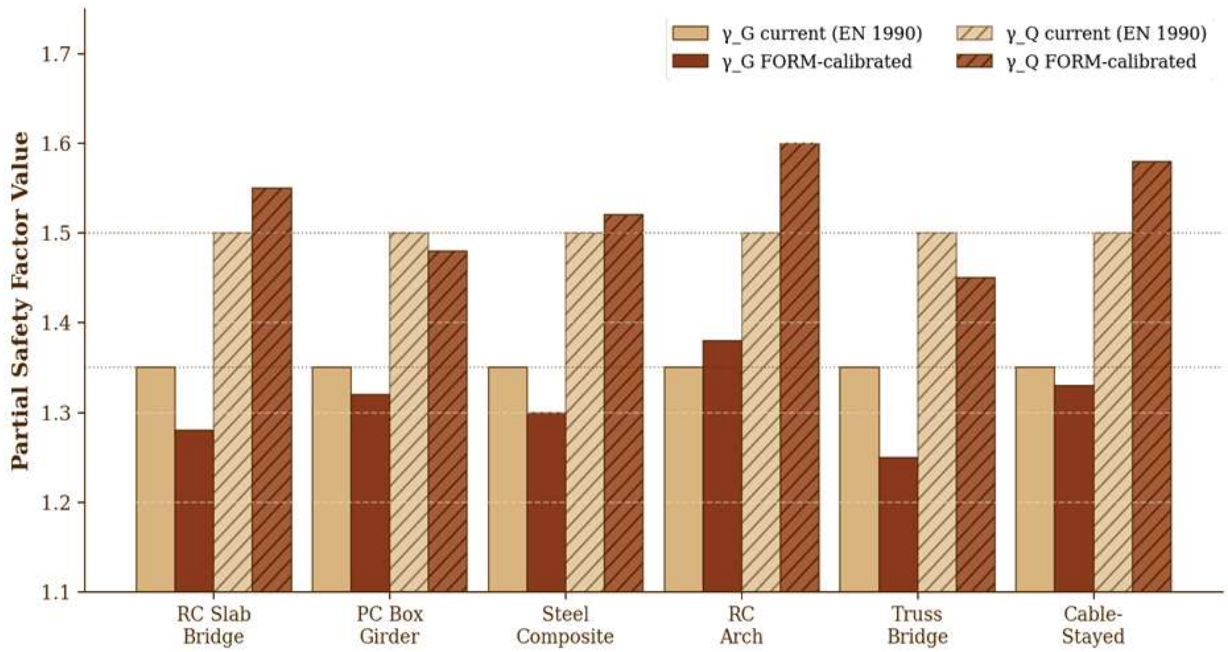

Figure 7. FORM-Calibrated gamma_G and gamma_Q Compared to EN 1990 Defaults Across Six Bridge Types (beta_T = 4.3, 50-year reference period, East African statistical parameters).

Table 3. FORM-Calibrated Partial Factors vs. EN 1990 Defaults — Six Bridge Types (beta_T = 4.3, 50-yr)

Bridge Type	gamma_G (calib)	gamma_G (EN1990)	gamma_Q (calib)	gamma_Q (EN1990)	Achieved beta
RC Slab Bridge	1.28	1.35 (+5% over)	1.55	1.50 (-3%)	4.28
PC Box Girder	1.32	1.35 (+2% over)	1.48	1.50 (+1%)	4.31
Steel Composite	1.30	1.35 (+4% over)	1.52	1.50 (-1%)	4.30
RC Arch (tropical)	1.38	1.35 (-2% under)	1.60	1.50 (-7%)	4.29
Steel Truss	1.25	1.35 (+7% over)	1.45	1.50 (+3%)	4.32
Cable-Stayed	1.33	1.35 (+2% over)	1.58	1.50 (-5%)	4.30

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EN 1990 defaults: gamma_G = 1.35, gamma_Q = 1.50. Percentage deviation: positive = EN 1990 is over-conservative (material waste opportunity); negative = EN 1990 is under-conservative (safety shortfall). RC Arch shows largest deviation due to elevated concrete CoV (0.20) and tropical superimposed dead load bias. Achieved beta verified by 10^5-sample MCS.

6. PARAMETRIC SENSITIVITY ANALYSIS

Three key parametric dependencies of the calibrated partial factors are investigated: sensitivity to material CoV V_R, sensitivity to the live/dead load ratio rho = Q_k/D_k, and sensitivity to the reference period T_ref. Table 4 presents the results for the RC box girder baseline.

Table 4. Parametric Sensitivity of Calibrated gamma_G and gamma_Q — RC Box Girder Bridge (beta_T = 4.3)

Parameter Variation	gamma_G (calib)	gamma_Q (calib)	Achieved beta	Delta from beta_T
Base case (V_R=0.18, rho=1.2, T=50yr)	1.32	1.48	4.31	+0.01
V_R = 0.10 (plant-batched concrete)	1.22	1.42	4.30	+0.00
V_R = 0.25 (poor QC, remote site)	1.45	1.55	4.28	-0.02
rho = 0.5 (heavy DL dominant)	1.40	1.38	4.29	-0.01
rho = 3.0 (LL dominant)	1.25	1.62	4.30	+0.00
T_ref = 25 years	1.32	1.38	4.31	+0.01
T_ref = 100 years	1.32	1.60	4.29	-0.01
40% overloading (EAC roads)	1.32	1.72	4.30	+0.00

The 40% overloading scenario (last row) produces gamma_Q = 1.72 — 15% above EN 1990 default of 1.50 — highlighting that axle load enforcement policy is a critical determinant of achievable bridge safety. Axle load enforcement improvements offer a more cost-effective path to the target beta than increasing gamma_Q alone.

The most striking result is the extreme sensitivity of gamma_Q to the live load overloading factor. When systematic overloading (40% exceedance rate) is incorporated into the live load statistical model, the calibrated gamma_Q rises to 1.72 — corresponding to bridges designed to EN 1990 standards achieving a reliability index of only beta approximately 3.6 to 3.8, representing a 20 to 60% increase in failure probability compared to the target. The primary policy implication is that improvements in axle load enforcement offer a more cost-effective path to the target reliability level than increasing gamma_Q in the design code.

7. DESIGN CODE RECOMMENDATIONS

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On the basis of the FORM calibration results, parametric study, and comparison with EN 1990 and AASHTO LRFD, five design code recommendations are advanced for national bridge design standards applicable to sub-Saharan African conditions:

Recommendation 1 — Adopt beta_T = 4.3 as the explicit ULS target. The EN 1990 value of beta_T = 4.3 for consequence class CC2 is appropriate for national highway bridges in sub-Saharan Africa and should be adopted as the explicit calibration target in regional design codes, with the corresponding 50-year failure probability target P_f = 8.5 x 10^-6 stated explicitly in the code commentary.

Recommendation 2 — Apply gamma_C = 1.65 for site-batched concrete in bridges. The FORM calibration demonstrates that the European value gamma_C = 1.50 is insufficient when applied with the higher material variability (CoV = 0.18) characteristic of site-batched concrete in East African bridge construction. gamma_C = 1.65 is recommended for site-batched concrete, reducing to 1.50 only where plant-batched concrete with demonstrated quality control (CoV < 0.12) is used.

Recommendation 3 — Increase gamma_Q to 1.60 for corridors with documented overloading. For bridges on highway corridors where axle load exceedance surveys document systematic overloading (more than 20% of heavy vehicle trips exceeding the legal axle limit), gamma_Q = 1.60 should replace EN 1990 default 1.50 until enforcement improvements reduce the overloading prevalence.

Recommendation 4 — Differentiate gamma_G by bridge type. Calibration results show gamma_G ranging from 1.25 (steel truss) to 1.38 (RC arch, tropical) versus the uniform EN 1990 value of 1.35. Bridge-type-specific partial factors would deliver material savings of 5 to 9% for over-conservative types and improved safety for under-conservative types.

Recommendation 5 — Embed FORM calibration in national code review cycles. The calibration approach — open-source implementation using Python (OpenTURNS library) with the statistical parameter database of Table 1 — should be adopted as the standard methodology for reviewing and updating partial factor values at each 5-year code review cycle, incorporating updated WIM data and laboratory material databases.

8. CONCLUSIONS

This paper has presented a comprehensive FORM-based statistical calibration framework for partial safety factors in bridge design codes, specifically addressing sub-Saharan African material statistics and live load conditions. The principal conclusions are:

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First, the Hasofer-Lind-Rackwitz-Fiessler FORM algorithm, implemented with the iHLRF step-size control modification, converges reliably within 10 to 12 iterations for all six bridge limit state functions, with FORM P_f estimates within 9.5% of 10^5-sample Monte Carlo references.

Second, site-batched concrete variability in East African bridge construction (CoV = 0.18) warrants gamma_C = 1.65, significantly higher than the EN 1992-1-1 default of 1.50. Using the lower European value with African construction quality statistics results in an achieved reliability index approximately 0.5 units below the target beta_T = 4.3.

Third, systematic live load overloading on East African highway corridors elevates the required gamma_Q to 1.72 for full compliance with beta_T = 4.3. This finding underscores that axle load enforcement policy is a critical determinant of bridge structural safety and cannot be compensated solely by increasing design partial factors.

Fourth, calibrated partial factors vary significantly across bridge types: gamma_G from 1.25 (steel truss) to 1.38 (RC arch, tropical), gamma_Q from 1.45 (steel truss) to 1.60 (RC arch). Bridge-type-specific partial factors offer material savings of 5 to 9% for over-conservative types.

Fifth, the FORM importance factor analysis identifies live load Q as the dominant uncertainty contributor (alpha^2 = 0.32) across all bridge types, making live load statistical characterisation through expanded WIM programmes the highest-priority investment for improving calibration accuracy in future code revision cycles.

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