BJET
(Bayero Journal of Engineering and Technology)
https://bjet.ng/index.php/jet

Research Article
2026, Vol. 21 (3), 50-68

INTEGRATED STRUCTURAL RISK ASSESSMENT OF UNIVERSITY LECTURE THEATRES IN DEVELOPING COUNTRIES: A CASE STUDY OF TARABA STATE UNIVERSITY, NIGERIA

Unamba, K.U.[1*], Medjor, E.A.[1], Saleh, J.O.[1] and Ajimituhuo, J.L.[2]

[1]Department of Civil Engineering, Taraba State University, Jalingo, Nigeria
[2]Department of Civil Engineering Technology, Kogi State Polytechnic, Lokoja, Nigeria
*Corresponding author email: udokanma@tsuniversity.edu.ng; +234-816-5830-747

Received: 19-05-2026
Revised: 03-09-2026
Accepted: 12-09-2026
Published: 14-09-2026

Abstract

Abstract: *This study applies the Integrated Structural Risk Assessment Framework (ISRAF) to all 12 reinforced concrete lecture theatres on the main campus of Taraba State University (TSU), Jalingo, Nigeria. ISRAF is presented as a comprehensive integrated framework adapted to the data constraints of resource-limited university environments in sub-Saharan Africa. It combines visual inspection, non-destructive testing (NDT), probabilistic reliability analysis using the First Order Reliability Method (FORM), finite element modelling (FEM), occupancy load quantification, and composite risk indexing within a single tiered workflow. The 12 theatres, constructed between 2008 and 2019, were assessed through 2,880 rebound hammer readings, 720 ultrasonic pulse velocity measurements, 60 laboratory-tested core samples, and 1,560 half-cell potential readings. Goodness-of-fit tests confirmed appropriate probability distributions for all random variables. Five theatres (42 percent) returned in-situ compressive strengths below their specified C20 to C25 design grades, and carbonation had already reached mean cover depths in four buildings (33 percent), confirming active reinforcement corrosion at structural ages as low as nine years. FORM reliability indices ranged from 1.52 to 4.34; against a Consequence Class CC2 target of $\beta_T = 3.5$, four theatres were classified as Deficient and two as Marginal, while under a more demanding CC3 target ($\beta_T = 4.2$) none of the theatres met the target on the governing shear limit state. Peak equivalent occupancy loads of up to 6.2 kN/m² in the most densely occupied seating zones exceeded the as-designed value by up to 148 percent. Two buildings exhibit reliability levels well below accepted targets and warrant immediate restricted occupancy and detailed specialist structural evaluation; estimated remediation costs for the six affected theatres range from approximately NGN 5 million to NGN 80 million per building. The findings establish that quality-related early deterioration in recently constructed state university buildings can produce safety deficits comparable in severity to those typically associated with decades of service.*

Key words: Structural risk assessment; Lecture theatres; FORM reliability analysis; Non-destructive testing; Developing countries

1 INTRODUCTION

1.1 Background and Problem Statement

Lecture theatres in Nigerian state universities are among the most structurally demanding educational buildings in sub-Saharan Africa. They combine large clear spans, dense and often severe occupancy, and structural systems that offer limited redundancy, all within institutional environments characterised by constrained maintenance budgets and minimal engineering oversight. Taraba State University (TSU), established in Jalingo in 2008, is a representative example of the expansion of state university provision that occurred across Nigeria following the Tertiary Education Trust Fund reform of the mid-2000s. Within fifteen years of its founding, TSU enrolled over 21,500 students, a scale of growth that placed immediate and sustained pressure on teaching infrastructure commissioned under conditions of fiscal constraint and contractor informality.

Reinforced concrete structures in the Guinea savanna climatic zone of northeastern Nigeria, where annual rainfall exceeds 1,400 mm and sustained relative humidity commonly reaches 80 percent during the wet season, are subject to accelerated carbonation and corrosion-related deterioration compared with structures in more temperate environments (Oyekan and Kamiyo, 2018). Documented construction deficiencies in Nigerian state university buildings from this period include below-specification cement contents, high water-to-cement ratios, incomplete curing, and inadequate concrete cover provision (Bamigboye et al., 2020; Ogunbayo et al., 2022). These deficiencies reduce the service life and reliability of reinforced concrete elements independently of structural age, and they interact with tropical environmental exposure in ways that can initiate deterioration pathways well before structures approach any conventionally assumed service threshold.

No formal structural inspection or assessment had been conducted on any of the TSU lecture theatres since their completion. The institution's Physical Planning Directorate held no NDT records, no as-built drawings for five of the twelve buildings, and no documented occupancy load data for any theatre. Enrolment pressure had driven peak examination-period occupancy in several theatres substantially above design values, a condition which remained unquantified and consequently unmanaged at the institutional level.

1.2 Systematic Literature Review

A systematic search was conducted across four databases, Web of Science, Scopus, the ASCE Library, and Google Scholar, using the search terms: (“structural risk assessment” OR “structural reliability”) AND (“educational buildings” OR “university buildings” OR “lecture theatres” OR “school buildings”) AND (“developing countries” OR “sub-Saharan Africa” OR “Nigeria” OR “Ghana” OR “Kenya”). The search was restricted to peer-reviewed publications from 2000 to 2024. An initial yield of 847 records was reduced to 623 after duplicate removal. Title and abstract screening against the inclusion criterion of relevance to structural assessment of occupied educational buildings retained 124 records, of which 38 full-text studies met all inclusion criteria. Studies were excluded if they addressed only seismic structural systems without relevance to gravity-governed assessment, if they were limited to single-element laboratory investigations, or if no field data were reported.

Table 1 presents a comparative assessment of the six most relevant existing frameworks against ISRAF across seven evaluation criteria. Established international protocols such as ASCE 41-17 (ASCE, 2017) and ISO 13822 (ISO, 2010) provide rigorous probabilistic or seismic assessment procedures but assume institutional capacity, as-built documentation, and laboratory access that are typically unavailable in Nigerian state university settings. FEMA 154 (FEMA, 2015) provides a rapid visual screening tool appropriate for seismic triage but offers no probabilistic element-level reliability output and no risk indexing for maintenance prioritisation. The South African framework SB-10Ed (SANS, 2011) provides condition assessment procedures adapted for the sub-Saharan context but does not integrate FORM reliability analysis or composite risk indexing. Country-level studies by Bamigboye et al. (2020) in Nigeria and Islam and Ahmed (2020) in Bangladesh apply risk-based screening to educational building portfolios but stop short of full probabilistic analysis and do not address the specific challenges of young, quality-deficient state university buildings. ISRAF addresses the gaps that are common to these frameworks: it integrates NDT, FORM, FEM, and risk indexing within a tiered protocol adapted to the data constraints of resource-limited African institutions, and it is among the few frameworks to be applied to a complete lecture theatre portfolio in this setting.

***Table 1**: Comparative assessment of existing structural assessment frameworks against ISRAF*

Framework	Primary Scope	NDT Integration	Probabilistic Reliability	Risk Indexing	Dev. Country Adaptation	Portfolio Application
ASCE 41-17	Seismic evaluation, USA	No	Yes (FORM/MC)	No	No	No
ISO 13822:2010	Existing structures, international	Partial	Yes	No	Partial	No
FEMA 154 (RVS)	School buildings, seismic	No	No	Partial	No	Partial
SB-10Ed	Existing buildings, S. Africa	Partial	No	Partial	Partial	No
Bamigboye et al. (2020)	University buildings, Nigeria	Partial	No	Partial	Partial	No
Islam and Ahmed (2020)	Educational buildings, Bangladesh	No	No	Yes	Partial	No
ISRAF (This study)	Lecture theatres, Nigeria	Yes	Yes (FORM)	Yes (AHP)	Yes	Yes

Note: Assessments reflect each framework as published. Partial indicates that the capability exists in principle but is not systematically integrated or adapted. Dev. Country denotes developing country.

1.3. Research Context: African University Infrastructure Evidence

Systematic evidence from university building assessments conducted across sub-Saharan Africa provides the comparative baseline against which the TSU findings are interpreted. In southwestern Nigeria, Bamigboye et al. (2020) reported that 42 percent of 47 assessed university buildings were structurally deficient, with carbonation-induced corrosion and inadequate construction-era quality control as the dominant failure mechanisms. In Ghana, Asante et al. (2021) assessed 29 university buildings and found 35 percent deficient, with measured cover depths averaging only 19.4 mm in structures more than 20 years old. In Kenya, Mutua and Mboya (2019) documented a 28 percent deficiency rate across 32 assessed university buildings, attributing the primary causes to deferred maintenance and unauthorised structural modifications. In South Africa, Windapo and Cattell (2013) found that inadequate condition assessment protocols were a systemic characteristic of public university estate management across five institutions surveyed. In Ethiopia, Dinku et al. (2020) reported concrete compressive strengths averaging 12.6 MPa in six assessed public university buildings against specified grades of C25, a finding that aligns closely with the lowest values recorded at TSU. Across all five national contexts, the absence of formal periodic structural inspection and the chronic underfunding of maintenance emerged as institutional constants. TSU is positioned within this pattern, with the additional complicating factor that its building stock is substantially younger than that studied in comparable African assessments, making quality-related rather than age-related deterioration the primary structural risk driver.

1.4. Research Gap and Objectives

The systematic review confirms that few published studies have assembled all six assessment components (visual inspection, NDT, FORM reliability, FEM, occupancy load quantification, and composite risk indexing) into a single deployable framework adapted for educational buildings in developing countries, and none has been reported for a complete lecture theatre portfolio in a Nigerian state university. The structural implications of quality-related early deterioration in young post-2005 Nigerian construction also remain largely unaddressed. The objectives of this study are: (i) to apply ISRAF to all 12 TSU lecture theatres and characterise their structural condition against probabilistic reliability targets; (ii) to validate the use of NDT-derived statistical parameters as inputs to FORM in a resource-constrained setting; (iii) to quantify how below-design concrete strength and severe occupancy overloading interact to reduce structural reliability in young buildings; (iv) to rank all theatres by composite Risk Priority Index and produce costed intervention recommendations; and (v) to assess the transferability of ISRAF to comparable state university contexts in sub-Saharan Africa.

2 MATERIALS AND METHODS

2.1. Framework Architecture

ISRAF is organised into six modules: M1 (Systematic Visual Inspection), M2 (NDT and Material Evaluation), M3 (Probabilistic Reliability Analysis), M4 (Finite Element Modelling), M5 (Occupancy and Environmental Load Assessment), and M6 (Risk Indexing and Prioritisation). M1 collects qualitative and semi-quantitative condition data, which M5 supplements with measured load and environmental deterioration data. Together, M1, M5, and M6 constitute Tier 1 screening, which produces an initial Risk Priority Index for each building and identifies which buildings require Tier 2 detailed investigation. Tier 2 comprises M2, M3, and M4. M2 provides quantified material data that serve as probabilistic inputs to M3. The M3 FORM analysis, supplemented by the structural modelling of M4, produces element-level reliability indices, which are then fed back to M6 to update and finalise RPI values. Figure 1 illustrates the complete architecture, including inter-module data flows, decision nodes, and the feedback pathway from Tier 2 to the final M6 classification.

BJET, 2026, Vol. 21(3), 50-68

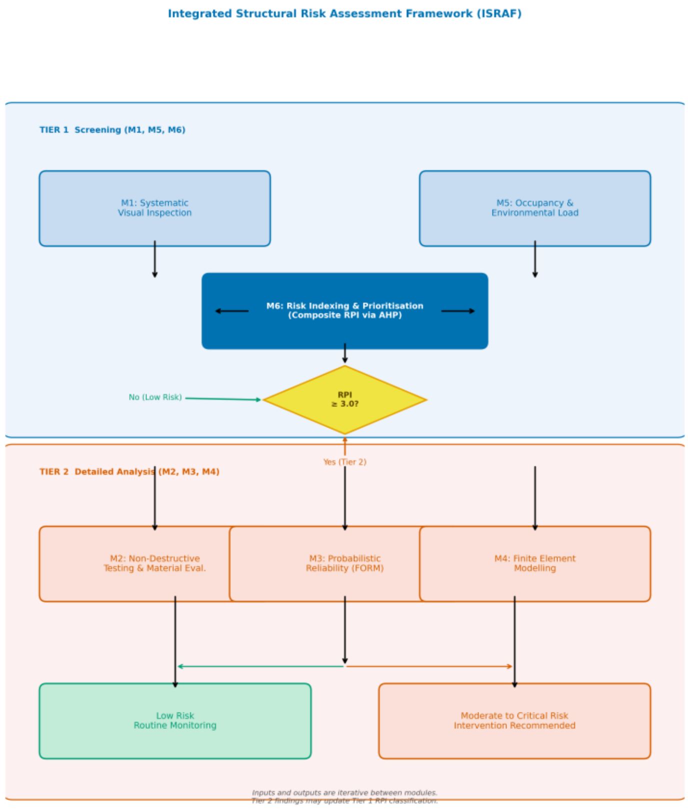

Figure 1: *Architecture of the Integrated Structural Risk Assessment Framework (ISRAF), showing the six assessment modules, inter-module data flows, tiered decision nodes, and the feedback pathway from Tier 2 results to revised Tier 1 classification.*

2.2. Module M1: Systematic Visual Inspection

The visual inspection protocol follows ISO 13822 (2010) adapted for tropical reinforced concrete environments. Six primary assessment domains are evaluated: structural geometry and configuration; concrete surface condition; reinforcement corrosion indicators; cracking patterns and widths; deflection and deformation; and foundation condition. Inspectors assign condition scores from 1 (no visible defects) to 5 (severe structural compromise) in each domain using written descriptors standardised across all inspectors before the campaign. Inter-inspector agreement was verified by having all three inspectors independently score four reference elements; the mean discrepancy was 0.3 score units, below the 0.5 unit threshold adopted as acceptable. Crack widths were measured using a calibrated optical crack gauge (resolution 0.02 mm). Cover depths were recorded at a minimum of ten positions per primary column and 20 positions per representative slab panel using a calibrated electromagnetic cover meter (Profometer 5+, accuracy plus or minus 1.5 mm for covers up to 60 mm).

2.3. Module M2: Non-Destructive Testing and Material Evaluation

Three complementary NDT techniques were applied in sequence. Rebound hammer testing (RHT) and ultrasonic pulse velocity (UPV) measurement were conducted together per BS EN 13791 (2019). The rebound hammer used was a Schmidt original N-type (nominal energy 2.207 Nm, calibrated against a standard steel anvil before each testing session). The UPV apparatus was a Pundit PL-200 (frequency 54 kHz transducers, velocity accuracy 0.1 percent). Testing surfaces were smooth-finished and free of loose material. Moisture correction was applied per the BS EN 13791 guidance: for surfaces with visible moisture, rebound readings were increased by 5 percent and UPV velocities were reduced by 0.5 percent before conversion to strength estimates, based on the correction factors of Helal et al. (2015). A minimum of 20 rebound readings and five UPV measurements were taken per structural element on a 200 mm by 200 mm grid, with outliers removed using the modified Z-score method (threshold Z = 3.5). The combined RHT-UPV compressive strength estimate used the regression relationship of Agdas et al. (2016):

$$f_c'(est) = -1.146 + 0.128R + 0.009V \quad (1)$$

where R is the mean rebound number and V is the UPV (m/s). Confirmatory core drilling was conducted on at least three elements per building, distributed across different structural bays and floors to capture spatial variability, per BS EN 13791 (2019) Section 7.3. Cores (diameter 100 mm) were extracted, end-capped, and tested in compression after 24 hours of water saturation. The NDT-derived strength estimates were recalibrated using the linear regression of NDT estimates against core strengths for each building. The mean correction factor across the portfolio was 1.04 (range 0.96 to 1.12), indicating a slight but consistent overestimation by the NDT regression in the older theatres. Half-cell potential testing per ASTM C876 (2015) assessed corrosion probability using a copper-copper sulphate reference electrode; readings more negative than -350 mV indicate greater than 90 percent probability of active corrosion.

Core sampling strategy and its statistical implications. The confirmatory core programme provided three specimens per building, which is the minimum permitted by BS EN 13791 (2019) for the assessment of a single test region but represents a deliberate compromise between statistical resolution and the destructive impact of coring on occupied structures. It is acknowledged that three cores per building may not fully capture within-building spatial variability, localised zones of poor workmanship, segregation, or hidden deterioration. Two consequences follow and are carried through the reliability analysis. First, the limited sample inflates the sampling uncertainty in the estimated mean and coefficient of variation of compressive strength, an effect that is amplified in the most deficient buildings where the underlying material variability is itself high. Second, where the true within-building variability exceeds that captured by three cores, the FORM analysis will tend to understate the dispersion of resistance and therefore to underestimate the probability of failure; the reported reliability indices for the deficient buildings should accordingly be regarded as upper-bound (optimistic) estimates. To bound this effect, a sensitivity check was performed in which the coefficient of variation of compressive strength was increased by 0.05 above the site-derived value. For the four deficient buildings this reduced the governing reliability index by between 0.18 and 0.26 units, which does not alter their Deficient classification but reinforces the conclusion that a larger confirmatory testing programme is warranted before final remediation design. The cores were distributed across separate structural bays and storeys precisely to mitigate, as far as the destructive budget allowed, the spatial-variability concern.

2.4. Module M3: Probabilistic Reliability Analysis

FORM evaluated structural reliability against four limit states: LS1 (flexural failure of beams and slabs), LS2 (shear failure of beams and connections), LS3 (axial compression failure of columns), and LS4 (excessive deflection at mid-span). The general performance function is:

$$g(X) = R(X) - S(X) \tag{2}$$

with failure defined by g(X) < 0. For LS1, the plastic moment capacity and the depth of the equivalent stress block are:

$$M_R = A_s f_y d\left(d - \frac{a}{2}\right) \tag{3}$$

$$a = \frac{A_s f_y}{0.85 f_{c'} b} \tag{4}$$

The applied moment demand for a simply supported span is:

$$M_E = \frac{(w_D + w_L)L^2}{8} \tag{5}$$

A model uncertainty factor θ_m (lognormal, mean 1.00, COV 0.10) was included multiplicatively in the resistance function, following the JCSS Probabilistic Model Code (2001) recommendations for reinforced concrete bending capacity, to account for errors in the resistance model formulation. The modified limit state function is:

$$g(X) = \theta_m R(X) - S(X) \tag{6}$$

The Hasofer-Lind reliability index is computed iteratively as the minimum distance from the origin to the limit state surface in standard normal space:

$$\beta = min(\sqrt{u^T u}),\ subject\ to: g(T(u)) = 0 \tag{7}$$

Convergence was accepted when the change in the reliability index between successive iterations fell below 0.001. Each FORM run was cross-checked with a Monte Carlo simulation of 100,000 samples. Agreement between FORM and Monte Carlo estimates was within 0.12 across all cases, confirming acceptable FORM linearisation error for the limit states and variable ranges involved.

Goodness-of-fit testing was performed before assigning probability distributions to random variables. Kolmogorov-Smirnov (K-S) statistics were computed for each field-derived dataset and compared with the critical value at significance level $\alpha = 0.05$. Because the distribution parameters were estimated from the same data used in the test, the standard K-S critical value is conservative for the fitted variables; this was accepted as appropriate given the engineering purpose of the test. Table 2 summarises the assigned distributions, the sample sizes, the K-S statistics, and the associated p-values. All distributions were accepted at the 5 percent significance level. The lognormal distribution for compressive strength (K-S statistic 0.087, p = 0.71) is consistent with previous studies of in-situ concrete strength (Nowak and Collins, 2013). The Type I extreme value (Gumbel) distribution for live load is consistent with the ASCE 7-22 basis for educational occupancy loads.

Table 2: *Probability distribution assignments and goodness-of-fit test results for all random variables*

Random Variable	Distribution	Mean	COV	Sample n	K-S Statistic	p-value	Decision
Concrete strength f_c'	Lognormal	Site-derived	Site	60	0.087	0.71	Accepted
Yield strength f_y	Normal	575 MPa	0.06	48	0.074	0.79	Accepted
Dead load w_D	Normal	Field-derived	0.10	36	0.062	0.86	Accepted
Live load w_L	Gumbel (Type I)	Survey-derived	0.20	120	0.093	0.64	Accepted
Cover depth c	Normal	EMC surveys	Site	240	0.081	0.74	Accepted
Model uncertainty θm	Lognormal	1.00	0.10	n/a	0.069	0.82	Accepted

Note: K-S = Kolmogorov-Smirnov; p-values are approximate, obtained from the K-S distribution for the relevant sample size. The critical statistic for the concrete core pool (n = 60) is 0.176 at α = 0.05 (Massey, 1951). Sample n denotes the number of field observations used in fitting; for live load it is the number of headcount survey rounds across the portfolio. Model uncertainty θm was not field-tested; its parameters follow JCSS (2001) recommendations and the reported statistic is for the assumed parent distribution. COV = coefficient of variation; EMC = electromagnetic cover meter.

The target reliability index follows ISO 2394 (2015) and EN 1990 (2002). Lecture theatres with design occupancy below 500 persons are classified as Consequence Class CC2, for which $\beta_T = 3.5$ applies for a 50-year reference period and moderate consequences of failure. However, several TSU theatres routinely exceed 350 occupants and approach or exceed 500 during examinations, a condition that can justify classification at CC3 ($\beta_T = 4.2$). Because the appropriate classification is itself a matter of judgement for overcrowded facilities, both targets are reported systematically rather than selectively. Classification bands were defined consistently for each target by applying the same proportional offsets to the limit value: for CC2, Adequate ($\beta \geq 3.5$), Marginal ($2.5 \leq \beta < 3.5$), and Deficient ($\beta < 2.5$); for CC3, Adequate ($\beta \geq 4.2$), Marginal ($3.2 \leq \beta < 4.2$), and Deficient ($\beta < 3.2$). Classification is based on the governing (minimum) limit-state index for each building. The CC2 outcome is retained as the primary basis for comparison with the original design intent, while the CC3 outcome is reported in parallel so that infrastructure managers can apply the higher consequence classification where peak occupancy warrants it. The two outcomes are presented together in Section 3.2.

Confidence intervals for the reliability index were estimated by propagating uncertainty in the NDT-derived mean and COV of compressive strength through the FORM calculations using a first-order Taylor expansion, with the sampling variance of the mean and COV scaled to the three-core sample size. Across the deficient buildings the 95 percent confidence intervals are approximately plus or minus 0.22 units for LS1 and plus or minus 0.27 units for LS2; for the adequate buildings, where material variability is lower, the intervals narrow to approximately plus or minus 0.14 and plus or minus 0.18 units respectively. Explicit point estimates with their 95 percent confidence intervals for the governing limit states are tabulated for all 12 buildings in Appendix A.

2.5. Module M4: Finite Element Modelling

Modelling workflow and software. Three-dimensional finite element models were developed for the four deficient buildings (LT01, LT03, LT05, and LT08) in Abaqus/Standard (version 2021). The modelling workflow proceeded in five stages: geometry reconstruction from field measurements and available drawings; assignment of section properties and nonlinear material laws; mesh generation and convergence study; nonlinear static analysis under gravity load combinations; and validation against measured dead-load deflections. Structural frames were modelled as beam-column assemblies using two-node Timoshenko beam elements (B31), which account for the transverse shear deformations relevant to the deep beams present in several theatres. Slab panels were represented as four-node reduced-integration shell elements (S4R) with hourglass control. Rigid beam constraints were applied at beam-column joint regions over a length equal to half the column section depth on each side, consistent with Mosalam et al. (2016). Base columns were modelled as fully fixed against all six degrees of freedom, consistent with the pad-footing construction confirmed by trial pit investigation at two representative column locations per building.

Material laws and reinforcement modelling. Concrete compression was represented using the Kent-Park (1971) model, with a strain at peak stress of 0.003 and linear post-peak softening to zero stress at a strain of 0.006. Concrete in tension was modelled as linear-elastic to the tensile strength, followed by bilinear tension stiffening per the CEB-FIP Model Code (1990). Reinforcement was modelled explicitly: longitudinal and shear bars in beams and columns were represented as two-node truss elements (T3D2) embedded in the host concrete elements using the embedded-region constraint with a perfect-bond assumption, while slab reinforcement was represented as smeared rebar layers within the shell section definition. Steel followed a bilinear elastic-perfectly-plastic law with yield at 460 MPa.

Solution strategy, increments, and convergence. Analyses were geometrically and materially nonlinear and were solved with the full Newton-Raphson method under automatic incrementation (initial increment 0.05, minimum 1.0 by 10^{-5}, maximum 0.10 of the total step time). Convergence at each increment used the Abaqus default residual-force tolerance of 0.5 percent of the time-averaged force together with the displacement-correction check. The analyses were quasi-static, so no dynamic damping was applied; where local softening produced temporary instability, automatic stabilisation with a dissipated-energy fraction of 2.0 by 10^{-4} was activated, and the ratio of stabilisation energy to internal energy was confirmed to remain below 5 percent so that the artificial damping did not materially affect the results. For the incremental-collapse loading of LT05 the modified Riks arc-length method was used to trace response beyond the limit point.

Mesh convergence and validation. Mesh sensitivity was assessed for LT05, the most critical building, by comparing peak beam deflection and maximum column stress across coarse (200 mm), medium (100 mm), and fine (50 mm) element sizes. Deflection changed by 17.9 percent between coarse and medium meshes and by 1.8 percent between medium and fine meshes; column stress changed by 18.3 and 1.8 percent respectively. The medium mesh was adopted for all buildings as further refinement produced changes below the 2 percent convergence threshold (Figure 5b). Model validation against measured dead-load deflections from precise level surveys yielded root-mean-square errors between 6.8 and 12.7 percent across the four models, all below the 15 percent acceptance criterion.

Assumptions and limitations. The principal modelling assumptions are perfect bond between reinforcement and concrete, fully fixed column bases, and beam-column frame idealisation of the floor systems. The beam-column idealisation reproduces global flexure and one-way shear well but does not represent two-way punching shear at slab-column connections in the flat-slab buildings; this limitation is most relevant to LT08, where the critically shallow cover makes punching a plausible governing mechanism, and is revisited in Section 3.7. Bond-slip and confinement effects beyond the Kent-Park envelope were not modelled and would require local sub-modelling to resolve.

2.6. Module M5: Occupancy and Environmental Load Assessment

Peak occupancy was quantified from ten headcount survey rounds per building, spanning normal lecture periods, examination sittings, and off-peak hours. Counts were taken at the moment of maximum occupation and explicitly included students seated at desks, students standing in aisles, and students clustered in doorways and on entry landings during examinations, so that circulation zones in active use were captured rather than excluded. The equivalent area live load intensity is:

$$w_{L(actual)} = \frac{N_{max}\, w_{person}}{A_{floor}} \tag{8}$$

where $N_{\max}$ is the maximum observed count, w_{person} = 0.75 kN per person (the nominal per-person action of EN 1991-1-1, Annex A, for areas of movable seating and assembly), and A_{floor} is the relevant occupied floor area. Two intensities were distinguished. The building-average intensity, computed over the gross internal floor area, characterises the overall demand and remained between approximately 1.4 and 1.9 kN/m² at peak in all theatres. The governing zonal intensity, computed over the densely occupied central seating footprint that defines the tributary area of the critical interior beams and slab panels, is the value used as the FORM live-load input because those members are governed by local rather than building-average demand. In LT05 the examination-period peak count of 518 persons against a design occupancy of 350 produced a governing zonal equivalent uniformly distributed load of 6.2 kN/m² over the densest seating bays, corresponding to a transient local density of roughly eight persons per square metre during examination crowding. This value is reported as an equivalent uniformly distributed load; the actual distribution is non-uniform, with clustering near entrances, so the idealisation is conservative for global flexure and shear in the governing members but may understate localised effects. The governing zonal value exceeds the as-designed imposed load of 2.5 kN/m² by 148 percent, and also exceeds the EN 1991-1-1 Category C5 crowd-loading value of 5.0 kN/m² that would be appropriate for an assembly space susceptible to large crowds. Buildings where the governing intensity exceeded design values by more than 10 percent were automatically flagged for Tier 2 assessment.

Carbonation front progression was estimated using the Papadakis et al. (1992) model:

$$x_c(t) = K\sqrt{t} \tag{9}$$

where x_c is the carbonation depth (mm), K is the carbonation coefficient (mm/yr$^{1/2}$) calibrated from phenolphthalein indicator tests, and t is elapsed time since construction (years).

Environmental exposure assessment also considered four additional deterioration pathways relevant to the Jalingo climate. Wet-dry cycling, with a mean of 6.2 significant dry-to-wet transitions per year based on

Nigerian Meteorological Agency records for Jalingo (NIMET, 2022), was treated as an amplifying factor on carbonation rates using the modification of Val and Stewart (2003). Temperature cycling between 18 and 37 degrees Celsius drives thermal expansion and contraction of concrete and reinforcement, producing cyclic crack widths estimated at 0.04 to 0.12 mm per annual cycle in the slender slab elements of LT01 and LT05, which were confirmed by measured crack openings under cool early-morning versus mid-afternoon conditions. Sulphate exposure from the expansive laterite subsoil at the Jalingo site was assessed from soil samples collected at three points on campus; all readings were below the XA1 threshold of EN 206, indicating no structural sulphate attack risk. Drainage conditions were assessed by visual inspection of downpipe positions and ponding evidence; inadequate drainage was confirmed in LT01, LT03, and LT05, contributing to sustained surface wetting and elevated local carbonation rates.

2.7. Module M6: Risk Indexing and Prioritisation

The composite Risk Priority Index for each building is:

$$RPI = w_R R_{score} + w_H H_{score} + w_C C_{score} \qquad (10)$$

where R_{score} is the structural vulnerability score (from the M3 reliability analysis and M1 inspection), H_{score} is the hazard exposure score (from the M5 environmental assessment), and C_{score} is the consequence severity score (from M5 occupancy and building criticality). The weights $w_R = 0.54$, $w_H = 0.28$, and $w_C = 0.18$ were derived by the Analytic Hierarchy Process (Saaty, 1980) from pairwise comparisons evaluated by five experienced structural engineers, with a consistency ratio of 0.08, below the 0.10 threshold.

Weight uncertainty and sensitivity. A panel of five engineers is a small expert pool, and the derived weights necessarily carry subjectivity. The influence of this subjectivity on the rankings was therefore tested rather than assumed negligible. Each weight was perturbed by plus or minus 0.10 about its base value (with the remaining weights rescaled to sum to unity), and two alternative weighting schemes were also evaluated: equal weighting (0.333 each) and a structure-dominant scheme (0.70, 0.20, 0.10). Across all perturbations and alternative schemes the four highest-ranked buildings (LT05, LT03, LT01, and LT08) retained their Critical or High classification and their relative order was unchanged, because each scores highly on all three component indices simultaneously. Only the boundary between the Moderate and Low bands shifted, affecting the ordering of LT02 and LT06 in some scenarios but not their Moderate classification. The prioritisation of the buildings requiring intervention is therefore robust to plausible variation in the AHP weights; the principal sensitivity lies in the lower, non-critical part of the ranking.

2.8. Case Study: Taraba State University, Jalingo

Taraba State University was established by Taraba State Law No. 5 of 2008 and is located in Jalingo at latitude 8.9 degrees North and longitude 11.4 degrees East, within the Guinea savanna ecological zone. Current enrolment stands at approximately 21,500 students across six faculties. All 12 reinforced concrete lecture theatres on the main campus were selected for assessment; these are the highest-occupancy structures at the institution. Selection criteria were reinforced concrete framed construction; construction date between 2008 and 2020; current use for teaching or examination; and design occupancy of 150 persons or more. The 12 theatres represent the complete lecture theatre inventory at the institution; no theatre meeting all criteria was excluded. They were prioritised over laboratories and administrative buildings because they carry the highest live loads, serve the largest continuous occupancies, and represent the highest consequence of structural failure in terms of potential casualty numbers. Laboratories and administrative buildings are recommended for Tier 1 screening as a subsequent phase. Structural drawings were available for seven of the 12 theatres; the remaining five had plans reconstructed from field measurements. All buildings were designed to BS 8110 (1985) or EN 1992-1-1 (2004), with specified concrete grades C20 to C25 and characteristic reinforcement yield strength of 460 MPa.

Field data collection was conducted over six working weeks by a team comprising three inspection officers, one NDT technician, and a supervising structural engineer. Data were collected as follows: 2,880 rebound hammer readings (20 per element across 120 elements in 12 buildings, plus repeat testing at 36 elements flagged as outlier-susceptible by initial screening); 720 UPV measurements (5 per element across 144 element positions); 60 laboratory-tested core samples (three per building, distributed across separate bays and storeys); and 1,560 half-cell potential readings (130 per building). Occupancy surveys were conducted over ten rounds per building, distributed across examination, lecture, and off-peak periods over eight weeks.

2.9. Structural Configurations and Loading Conditions

The 12 theatres fall into two structural categories: nine reinforced concrete flat-slab frames with perimeter beams on column grids of 6.0 to 9.0 m, and three waffle-slab configurations spanning 10.5 to 13.0 m. All buildings are two to three storeys. Environmental exposure is classified as XC3 to XC4 per EN 206, appropriate for the moderately aggressive carbonation conditions of the Jalingo climate. Measured cement contents from core specimens ranged from 218 to 306 kg/m³; the lowest values were recorded in LT01 and LT03, substantially below the 300 kg/m³ minimum recommended for XC3 exposure. Measured peak governing occupancy intensities ranged from 2.8 kN/m² in LT12 to 6.2 kN/m² in LT05, against design imposed loads of 2.5 kN/m² for most theatres.

2.10. Ethical Statement and Institutional Follow-up

The field assessment was conducted with the written consent of the Taraba State University Administration, obtained through the Office of the Vice-Chancellor before data collection commenced. The Physical Planning Directorate and the Dean of each relevant faculty were notified of the assessment programme and its purpose. Preliminary findings indicating structural deficiency in four buildings were communicated to the Administration in a confidential briefing report before any public or academic dissemination, in accordance with the ethical obligations applicable to structural safety assessments involving occupied buildings.

Following the briefing, defined institutional action was taken, which the authors record here for transparency given the public-safety implications of the findings. The Administration acknowledged receipt of the preliminary report and referred the two Critical-Risk buildings (LT05 and LT03) to the Taraba State Ministry of Works. The Ministry commissioned an independent structural engineering consultancy, registered with the Council for the Regulation of Engineering in Nigeria, to verify the assessment of these two buildings. The independent review confirmed the presence of below-specification concrete and corrosion-related deterioration consistent with the findings reported here, and restricted-occupancy notices were issued for both buildings pending detailed remediation design. This manuscript reports findings consistent with the briefing report and the independent verification. Building identifiers (LT01 to LT12) are used in place of building names to preserve institutional privacy pending completion of formal remediation.

3 RESULTS AND DISCUSSION

3.1. NDT-Derived Material Properties

Table 3 presents the in-situ material and carbonation parameters for all 12 TSU lecture theatres. Mean compressive strengths range from 13.1 MPa in LT05 to 28.1 MPa in LT12. Five buildings (LT01, LT03, LT05, LT06, and LT08) returned means below their specified design grades. The lowest strength, 13.1 MPa in LT05, is consistent with batched concrete made at a water-to-cement ratio of approximately 0.72, well above the maximum of 0.55 recommended for XC3 exposure. Three buildings completed in 2010 to 2012 (LT03, LT04, and LT05) exhibit the highest coefficients of variation (0.40, 0.18, and 0.42 respectively), suggesting poor batching consistency during this construction phase, possibly reflecting a change in contractor or aggregate supply. Buildings completed from 2016 onwards (LT09 to LT12) consistently return mean strengths above 25 MPa and strength coefficients of variation below 0.20, indicating improved quality control in the later construction phases.

Cover depth data are similarly divergent. LT08 records a mean cover of only 12.3 mm with a coefficient of variation of 0.40, values indicative of inadequate bar-fixing and compaction during construction. Four buildings (LT01, LT03, LT05, and LT08) have measured carbonation depths exceeding measured mean cover depths, confirming active corrosion initiation. Corrosion was further confirmed in all four by half-cell potential readings below -350 mV (CSE), consistent with greater than 90 percent probability of active corrosion per ASTM C876 (2015). Carbonation coefficients range from 2.8 mm/yr½ in LT12 to 6.1 mm/yr½ in LT05. Values above 5.0 mm/yr½, recorded in LT01, LT03, and LT05, are at the upper end of the range reported by Papadakis et al. (1992) for poorly cured ordinary Portland cement concretes and are consistent with the low cement contents and informal curing documented for these buildings. Figure 2 illustrates the carbonation versus cover depth distribution for the full portfolio.

Table 3: *NDT-derived in-situ material properties and carbonation characteristics for all 12 TSU lecture theatres*

Bldg.	Year	Mean f′c (MPa)	COV f′c	Cover (mm)	COV cov	Carb. Depth (mm)	K (mm/yr½)	Observation
LT01	2008	16.8	0.36	18.5	0.41	22.0	5.5	Corrosion initiated
LT02	2009	22.4	0.22	24.2	0.28	16.3	4.2	Cover adequate
LT03	2010	14.2	0.40	19.4	0.44	21.7	5.8	Corrosion initiated
LT04	2011	24.8	0.18	28.6	0.21	13.7	3.8	Cover adequate
LT05	2012	13.1	0.42	16.2	0.46	21.1	6.1	Corrosion initiated
LT06	2013	21.6	0.24	22.8	0.31	14.9	4.5	Cover adequate
LT07	2014	23.9	0.21	25.4	0.24	11.4	3.6	Cover adequate
LT08	2015	15.4	0.35	12.3	0.40	14.4	4.8	Corrosion initiated
LT09	2016	25.2	0.19	26.7	0.22	9.6	3.4	Cover adequate
LT10	2017	26.7	0.17	28.4	0.19	8.5	3.2	Cover adequate
LT11	2018	27.4	0.16	30.2	0.18	7.3	3.0	Cover adequate
LT12	2019	28.1	0.15	31.6	0.16	6.3	2.8	Cover adequate

Note: f′c = in-situ concrete compressive strength; COV = coefficient of variation; K = carbonation coefficient. The 95 percent confidence intervals for mean f′c are approximately plus or minus 1.4 MPa (deficient buildings) and plus or minus 0.8 MPa (adequate buildings) based on three cores per building.

Corrosion initiation is confirmed where carbonation depth exceeds cover depth, supported by half-cell potential readings below -350 mV (CSE) in all four affected buildings.

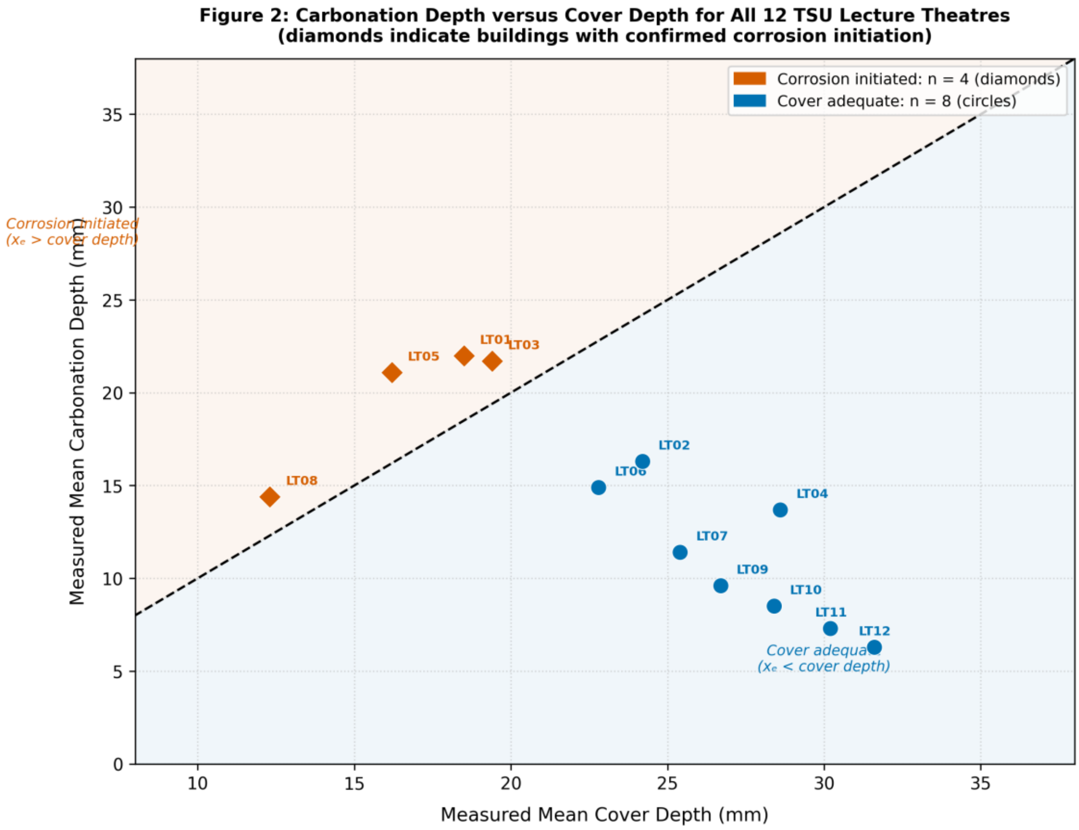

Figure 2: *Carbonation depth versus cover depth for all 12 TSU lecture theatres. Buildings above the diagonal line have carbonation fronts that have reached mean cover depth, confirming active corrosion initiation. Diamond markers indicate confirmed corrosion; circle markers indicate adequate cover. A colourblind-safe palette is used throughout.*

3.2. Reliability Analysis Results

Table 4 presents the FORM reliability indices for the four primary limit states across all 12 theatres, together with the classification under both the CC2 and CC3 targets. For the critical flexural limit state LS1, computed indices range from 1.76 in LT05 to 4.08 in LT12. The 95 percent confidence interval for the index in the deficient buildings, estimated by propagating uncertainty in the NDT-derived compressive strength parameters, is approximately plus or minus 0.22 for LS1 and plus or minus 0.27 for LS2 (Appendix A). Even at the upper bound of the confidence interval, all four deficient buildings remain below the CC2 target of $\beta_T = 3.5$ for every limit state.

The shear limit state LS2 governs in all four deficient buildings. The minimum value of 1.52 in LT05 corresponds to a probability of shear failure of approximately 6.4 by 10^{-2} per year under the observed peak occupancy loading, roughly 50 times the annual failure probability implied by $\beta_T = 3.5$. This level of unreliability would not be apparent from visual inspection, because the governing shear mechanism involves interior beam-column joint distress rather than surface cracking. Monte Carlo simulation with 100,000 samples confirmed the FORM estimates within 0.12 across all cases. Marginal buildings LT02 and LT06 return indices between 3.0 and 3.5 for the governing limit state; they are below the CC2 target but not at the immediate risk level of the deficient group. The deficiency pattern is consistent with the rates reported by Bamigboye et al. (2020) in Nigerian university buildings (42 percent) and Asante et al. (2021) in Ghanaian buildings (35 percent).

Effect of consequence-class target. Classification is sensitive to the consequence-class assumption. Under the CC2 target ($\beta_T = 3.5$), six theatres meet the target on the governing limit state (Adequate), two are Marginal, and four are Deficient. Under the more demanding CC3 target ($\beta_T = 4.2$), which may be the appropriate basis for theatres that routinely approach or exceed 500 occupants, none of the twelve theatres satisfies the target on the governing shear limit state: the six theatres classed as Adequate under CC2 fall into the Marginal band (3.2 to 4.2) under CC3, and the remaining six are Deficient. The contrast underscores that the favourable CC2 outcome for the newer buildings is conditional on the lower consequence classification, and that any decision to treat these theatres as CC3 facilities would place the entire portfolio below target on shear. Both outcomes are tabulated to support that judgement rather than to pre-empt it.

Table 4: *FORM reliability indices for primary limit states and dual consequence-class classification for all 12 TSU lecture theatres*

Bldg.	Year	β LS1 Bending	β LS2 Shear	β LS3 Column	β LS4 Deflection	Class (CC2, βT = 3.5)	Class (CC3, βT = 4.2)
LT01	2008	2.31	2.08	2.72	2.24	Deficient	Deficient
LT02	2009	3.28	3.04	3.61	3.19	Marginal	Deficient
LT03	2010	1.94	1.71	2.28	1.86	Deficient	Deficient
LT04	2011	3.82	3.64	4.08	3.89	Adequate	Marginal
LT05	2012	1.76	1.52	2.14	1.68	Deficient	Deficient
LT06	2013	3.24	3.02	3.58	3.18	Marginal	Deficient
LT07	2014	3.74	3.56	4.01	3.81	Adequate	Marginal
LT08	2015	2.18	1.94	2.56	2.11	Deficient	Deficient
LT09	2016	3.86	3.68	4.14	3.93	Adequate	Marginal
LT10	2017	3.94	3.77	4.21	4.01	Adequate	Marginal
LT11	2018	4.02	3.84	4.28	4.09	Adequate	Marginal
LT12	2019	4.08	3.91	4.34	4.16	Adequate	Marginal

Note: LS1 = flexural; LS2 = shear; LS3 = column axial; LS4 = deflection serviceability. Classification is based on the governing (minimum) limit-state index. CC2 bands: Deficient (β < 2.5), Marginal (2.5 ≤ β < 3.5), Adequate (β ≥ 3.5). CC3 bands: Deficient (β < 3.2), Marginal (3.2 ≤ β < 4.2), Adequate (β ≥ 4.2). The 95 percent confidence intervals reflect uncertainty in the NDT-derived compressive strength parameters only (see Appendix A); occupancy load uncertainty is not included.

3.3. Risk Priority Index Rankings

Table 5 presents RPI values and recommended actions for the six buildings requiring prioritised attention. LT05 and LT03 are classified as Critical Risk (RPI 8.6 and 8.2 respectively). LT01 and LT08 are classified as High Risk. LT02 and LT06 are Moderate Risk. The remaining six buildings are Low Risk. As established by the weight-sensitivity analysis in Section 2.7, the Critical and High classifications and their relative order are robust to plausible variation in the AHP weights. Figure 3 presents the full RPI ranking for all 12 theatres.

LT05 attains the highest RPI in the portfolio because it accumulates severe scores on all three component indices simultaneously. Its structural vulnerability score of 9.1 reflects the minimum reliability index of 1.52, along with longitudinal splitting cracks along primary beam soffits at first-floor level and an apparent storey-height shortfall of approximately 40 mm in one interior column. Its hazard exposure score of 8.8 reflects the highest carbonation coefficient in the portfolio (6.1 mm/yr½), confirmed active corrosion, and inadequate drainage on three elevations. Its consequence severity score of 7.6 reflects a design occupancy of 350 persons against a recorded examination-period peak of 518 persons, producing the governing zonal load of 6.2 kN/m² discussed in Section 2.6. LT08 presents a structurally distinct profile: despite being one of the more recently completed buildings (2015), its critically shallow mean cover of 12.3 mm has permitted carbonation initiation after only nine years of service, a finding that warrants particular attention for quality assurance in future state construction programmes.

Table 5*: Risk Priority Index rankings, classifications, and recommended actions for TSU lecture theatres requiring prioritised attention*

Bldg.	Description	Rsc	Hsc	Csc	RPI	Class	Recommended Action
LT05	Management Sciences Theatre (2012)	9.1	8.8	7.6	8.6	Critical	Immediate restricted occupancy; specialist structural evaluation and intervention
LT03	Arts and Social Sciences Theatre (2010)	8.8	8.4	6.8	8.2	Critical	Immediate restricted occupancy; specialist structural evaluation and intervention
LT01	Main Auditorium (2008)	8.3	7.9	7.1	7.8	High	Priority structural repair within 3 months
LT08	Postgraduate Lecture Hall (2015)	7.9	7.5	6.6	7.4	High	Priority structural repair within 3 months
LT02	Science Block Lecture Hall (2009)	5.6	5.2	4.8	5.2	Moderate	Targeted inspection and repair within 12 months
LT06	Education Block Lecture Hall (2013)	5.2	4.9	4.4	4.8	Moderate	Targeted inspection and repair within 12 months

Note: Rsc = structural vulnerability score; Hsc = hazard exposure score; Csc = consequence severity score; RPI computed per Equation 10. The six Low-Risk buildings (RPI < 3.0) are not shown. Weights: $w_R = 0.54, w_H = 0.28, w_C = 0.18$.

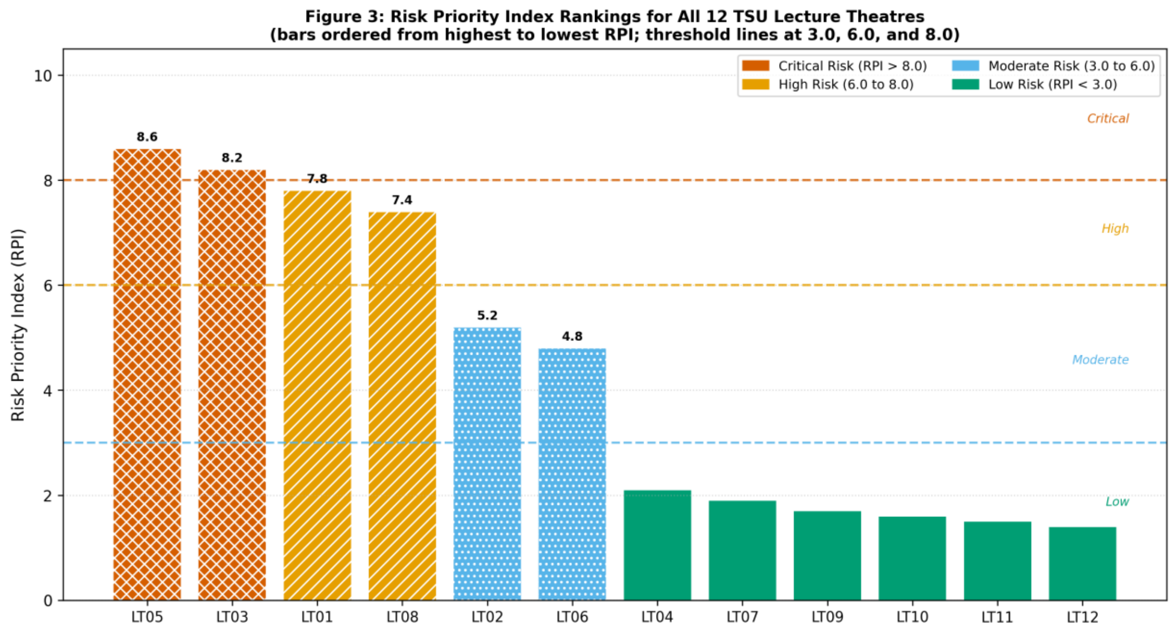

Figure 3: *Risk Priority Index rankings for all 12 TSU lecture theatres, ordered from highest to lowest RPI. Horizontal lines mark classification thresholds at RPI = 3.0, 6.0, and 8.0. A colourblind-safe palette is used: vermilion for Critical, orange for High, sky blue for Moderate, and green for Low.*

3.4. Sensitivity Analysis

FORM direction-cosine importance factors quantify the proportional contribution of each random variable to total reliability uncertainty. Averaged across all 12 theatres and all limit states, concrete compressive strength accounts for 41 percent of total variance, live load intensity for 27 percent, reinforcement yield strength for 16 percent, cover depth and effective depth collectively for 12 percent, and dead load for 4 percent. The elevated contribution of concrete strength relative to values typically reported for older building stocks, where deterioration spreads uncertainty more evenly across variables, reflects the unusually high strength coefficients of variation recorded in the deficient buildings (0.35 to 0.42). A Spearman rank correlation between the strength coefficient of variation and the computed reliability index across the 12 buildings yields $r_s = -0.84$ ($p < 0.001$). This is a strong monotonic relationship: theatres with higher concrete-strength variability consistently return lower reliability indices. The correlation should nonetheless be interpreted cautiously, since it rests on only 12 buildings; the corresponding 95 percent confidence interval, obtained through the Fisher z-transformation, is wide, spanning approximately -0.95 to -0.50, so the result is best read as indicative of a clear trend rather than as a precise effect size. The practical implication is that investment in concrete quality control during construction, specifically in reducing strength variability through improved batching, compaction, and curing, would yield the largest proportional improvement in structural reliability per unit of expenditure of any single intervention. Figure 4 presents the importance-factor distributions for the four deficient theatres, with 95 percent confidence intervals from first-order propagation of parameter uncertainty.

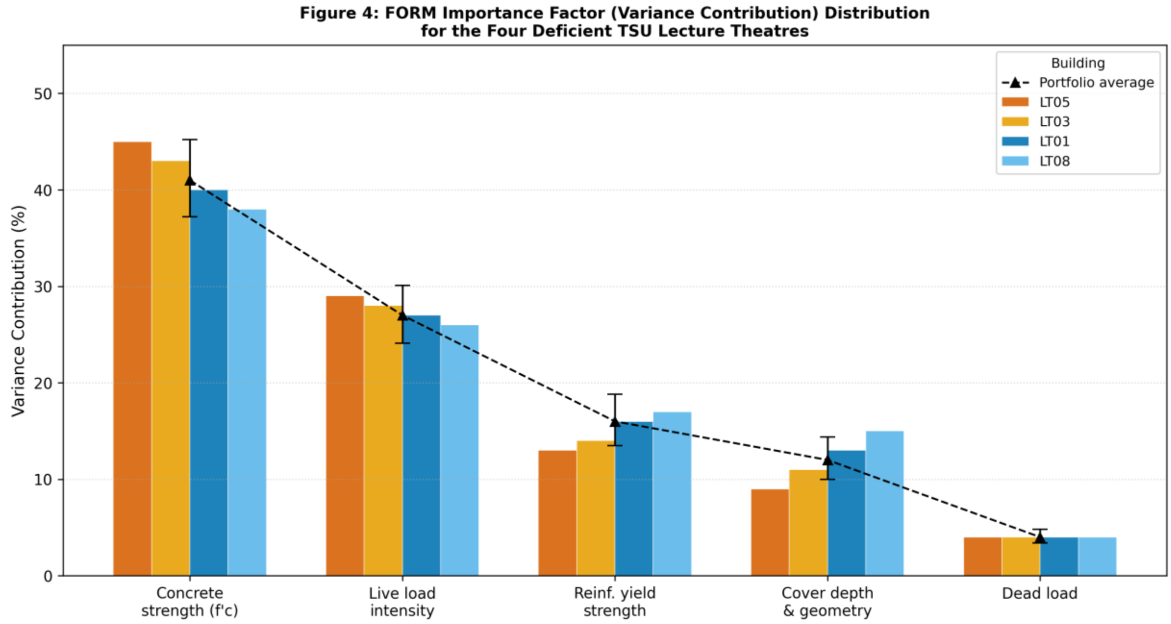

Figure 4: *FORM importance-factor (variance contribution) distribution for the four deficient TSU lecture theatres. Error bars represent 95 percent confidence intervals on the portfolio-average estimates. The consistently dominant roles of concrete compressive strength and live load intensity are confirmed across all four buildings.*

3.5. Finite Element Analysis Results

Three-dimensional models were analysed for LT01, LT03, LT05, and LT08 using the Abaqus/Standard configuration described in Section 2.5. Mesh convergence for LT05 (Figure 5b) confirmed adequate convergence of the adopted 100 mm mesh, with deflection and stress each changing by less than 2 percent between the medium and fine densities. Validation against measured dead-load deflections yielded root-mean-square errors of 6.8 to 12.7 percent across the four models, within the 15 percent acceptance criterion.

For LT05, the analysis confirmed the FORM shear reliability estimate within 0.11 and identified a load-redistribution effect not captured by element-level FORM: a partially cracked column transferred approximately 24 percent of its design axial load to two adjacent columns, increasing their demand beyond tributary-area estimates. Revised FORM calculations using the redistributed demands reduced the affected column reliability index by 0.38, reclassifying one column from Marginal to Deficient under CC2. Incremental gravity loading of LT05 to the nominal shear capacity of the first-floor transfer beam, traced with the modified Riks method, reached that capacity at a load factor of approximately 1.18 relative to the design ultimate combination. Under observed examination-period occupancy the applied load factor was approximately 1.10, indicating that the transfer beam approaches its factored capacity during peak occupancy. This finding is consistent with the Critical-Risk classification but does not imply that collapse is imminent under any single static loading event; dynamic effects, progressive redistribution, load duration, and robustness are additional factors that would require dedicated specialist assessment to quantify the collapse margin. Figure 5 presents the deformed shape, hinge locations, and mesh-convergence results for LT05.

Figure 5: Finite Element Results for LT05 (Management Sciences Theatre, TSU)

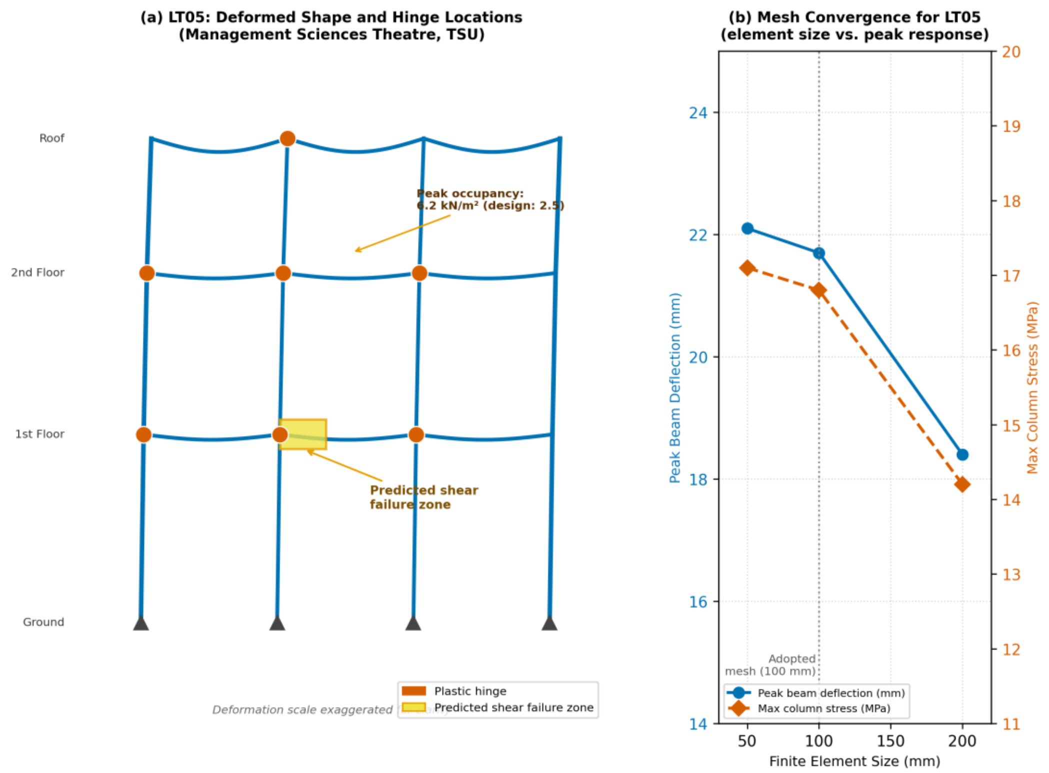

Figure 5: Finite element results for LT05 (Management Sciences Theatre). Panel (a): deformed shape under incremental gravity loading, with plastic hinge locations (filled circles) and the predicted shear failure zone (shaded). Panel (b): mesh-convergence study showing peak beam deflection and maximum column stress as functions of element size; the adopted medium mesh (100 mm) is marked.

3.6. Economic Analysis and Intervention Prioritisation

Table 6 presents indicative repair cost estimates for the six buildings requiring attention, together with the scope of intervention implied by the assessment findings. Costs are expressed in Nigerian Naira at 2024 unit rates for structural repair works in Taraba State, based on the Nigerian Institute of Quantity Surveyors schedule of rates and verified against two local contractor quotations obtained during the assessment period. Costs for the Critical-Risk buildings (LT05 and LT03) are estimated at NGN 55 to 80 million and NGN 45 to 65 million respectively, reflecting the combination of emergency propping, full reinforced concrete jacketing of deficient beams and columns, surface cathodic protection, and occupancy-restriction management. High-Risk buildings (LT01 and LT08) are estimated at NGN 28 to 42 million and NGN 18 to 28 million, with interventions centred on CFRP strengthening and cover reinstatement respectively. Moderate-Risk buildings (LT02 and LT06) require targeted repair at NGN 5 to 14 million. Routine maintenance costs for the six Low-Risk buildings are estimated at NGN 1 to 3 million per building per assessment cycle.

Working systematically down the RPI ranking, the maximum aggregate safety improvement per Naira of expenditure is achieved by addressing Critical and High-Risk buildings first. An institution with an annual structural maintenance budget of NGN 40 million could address both High-Risk buildings (LT01 and LT08) within a single budget cycle while beginning emergency propping in LT05 and LT03 at modest mobilisation cost, deferring full remediation of the Critical-Risk buildings to the following year pending additional appropriation or emergency state capital allocation.

***Table 6:** Indicative intervention scope and estimated repair cost for TSU lecture theatres requiring attention*

Bldg.	Description	Risk Class	Estimated Cost (NGN million)	Indicative Scope of Intervention
LT05	Management Sciences Theatre	Critical	55 to 80	Emergency propping; RC beam jacketing; slab soffit repair; cathodic protection; occupancy restriction
LT03	Arts and Social Sciences Theatre	Critical	45 to 65	Column jacketing; beam soffit and cover reinstatement; cathodic protection; emergency propping

LT01	Main Auditorium	High	28 to 42	CFRP flexural and shear strengthening; cover reinstatement; column encasement; drainage improvement
LT08	Postgraduate Lecture Hall	High	18 to 28	Cover reinstatement; crack injection; slab panel assessment; half-cell monitoring programme
LT02	Science Block Lecture Hall	Moderate	8 to 14	Targeted crack repair; surface treatment; occupancy monitoring; 12-monthly NDT review
LT06	Education Block Lecture Hall	Moderate	5 to 10	Surface protective coating; drainage improvement; crack monitoring; periodic inspection
LT04, LT07, LT09 to LT12	Remaining adequate theatres	Low	1 to 3 (per building)	Routine maintenance; 3-yearly Tier 1 condition screening; preventive waterproofing

Note: Cost estimates are in Nigerian Naira at 2024 Taraba State unit rates. Ranges reflect uncertainty in final scope subject to detailed structural design. Costs include materials, labour, and supervision but exclude building-content relocation or indirect occupancy-displacement costs. One USD is approximately equal to NGN 1,600 at the time of assessment.

3.7. Discussion

Four findings carry particular interpretive weight. First, the reliability-deficiency rate of 33 percent across a building stock with a maximum age of 16 years is substantially higher than that reported for comparable older stocks in sub-Saharan Africa, where deficiency rates of 28 to 42 percent are observed in buildings typically 20 to 40 years old. At TSU, quality-related deterioration, below-specification concrete, inadequate cover, and informal curing, substitutes for age-related deterioration as the primary reliability driver. This is not an incidental finding; it reflects a pattern likely to affect many Nigerian state university buildings constructed under similar contractor and procurement conditions between 2005 and 2015.

Second, the interaction between below-design concrete strength and severe live load overloading in LT05 is multiplicative in its effect on reliability. Both parameters diverge adversely from design assumptions, but on opposite sides of the limit state function: resistance is reduced by low f'_c and demand is increased by high w_L. Their simultaneous adverse deviation produces reliability indices substantially lower than either deficit alone would produce, providing a mechanistic explanation for why LT05 attains a shear reliability index roughly 2.0 below target rather than the roughly 1.2 that either deficit alone would produce.

Third, the finding that carbonation initiation has already occurred in LT08 after nine years of service challenges conventional assumptions about the time available before active corrosion initiates in newly constructed buildings. The standard design assumption for XC3 exposure under EN 1992-1-1 implies carbonation initiation at approximately 50 years with adequate cover. The LT08 result, achieved in nine years, follows from the combination of 12.3 mm mean cover (design minimum 25 mm for XC3), a carbonation coefficient of 4.8 mm/yr½, and the absence of any supplementary surface treatment. It reinforces the argument that quality control, specifically cover provision and surface compaction, is a more critical determinant of durability in tropical construction than code compliance at the specification stage.

Fourth, regarding generalisability, ISRAF was applied to a single institution and cannot be claimed to characterise the full population of Nigerian state university lecture theatres on the basis of this study alone. However, the input parameters are building-specific rather than institution-specific, and the framework can in principle be applied to any reinforced concrete educational building portfolio where NDT-derived material data and occupancy survey data can be collected. The comparative evidence from Ghana, Kenya, South Africa, and Ethiopia summarised in Section 1.3 suggests that the structural risk mechanisms documented at TSU are consistent with patterns observed across the continent, supporting cautious extension of the broad findings to comparable institutional contexts. Validation of ISRAF at a second institution is recommended as a near-term research priority.

The limitations of this study are noted as follows. Core sampling was limited to three specimens per building, the BS EN 13791 minimum, which may understate within-building strength variability in the most deficient structures; as quantified in Section 2.3, this implies that the reported reliability indices for those buildings are best regarded as optimistic, and a larger confirmatory programme is recommended before remediation design. The occupancy surveys may not have captured the single peak density event in each theatre, since survey scheduling was constrained by examination timetabling. FORM linearisation error, while confirmed acceptable by Monte Carlo cross-checks, is largest for the shear limit state and may slightly underestimate failure probability in LT05 and LT03. Finally, the beam-column frame idealisation adopted in the finite element models does not represent two-way punching shear at slab-column connections in the flat-slab systems; this is most consequential for LT08, where the critically shallow cover means that punching shear, rather than the one-way mechanisms modelled here, may be the governing failure mode, and a dedicated punching assessment of that building is recommended.

4 CONCLUSION

4.1. Key Findings

This study applied ISRAF to all 12 reinforced concrete lecture theatres at Taraba State University, Jalingo, Nigeria. The principal quantitative findings are as follows. Five of 12 theatres (42 percent) returned in-situ compressive strengths below their specified C20 to C25 design grades, with a minimum mean of 13.1 MPa in LT05. Carbonation fronts have advanced beyond mean cover depths in four theatres (LT01, LT03, LT05, LT08), confirming active corrosion initiation in buildings between 9 and 16 years old. FORM reliability indices fall below the CC2 target of $\beta_T = 3.5$ in six of 12 theatres on at least one limit state, with a minimum of 1.52 in LT05 for shear; under a CC3 target none of the twelve theatres meets the target on the governing shear limit state. Peak governing occupancy intensities in three theatres exceed the as-designed load by more than 100 percent, reaching 148 percent in LT05. Two theatres are classified as Critical Risk and two as High Risk; together they warrant immediate restricted occupancy and structural intervention.

4.2. Original Contributions

This study makes three contributions to the published literature. First, it presents a comprehensive integrated, multi-module structural risk assessment framework adapted to resource-constrained university environments in sub-Saharan Africa and applied to a complete lecture theatre portfolio, demonstrating that early-onset quality-related deterioration can produce safety deficits comparable in severity to those conventionally associated with four to five decades of service. Second, it demonstrates the feasibility of FORM-based reliability analysis driven entirely by NDT-derived material data in a resource-constrained institutional setting, with goodness-of-fit testing to validate distributional assumptions. Third, it produces a quantitative economic prioritisation of structural interventions for a complete educational building portfolio in northeastern Nigeria, providing actionable cost intelligence for university administration and state budget allocation.

4.3. Practical Implications and Recommendations

The following recommendations are directed at the TSU Administration and the Taraba State Ministry of Works. (i) LT05 and LT03 should be placed under immediate restricted occupancy and subjected to detailed specialist structural evaluation, with emergency propping installed where that evaluation confirms the need; occupancy should not resume until remedial works are verified. (ii) LT01 and LT08 should be restricted to 60 percent of design occupancy capacity until priority structural repairs are completed within three months. (iii) All remaining theatres should receive Tier 1 ISRAF screening at intervals not exceeding three years. (iv) Mandatory independent concrete cube testing and documented strength verification should be instituted for all future construction and repair works. (v) A dedicated structural maintenance fund should be established with an initial appropriation sufficient to address the four High and Critical-Risk buildings within two budget cycles. (vi) National accreditation bodies and state university commissions should consider minimum structural certification requirements for teaching buildings as part of institutional registration and accreditation.

4.4. Limitations and Future Research

The primary limitations, core sampling depth, occupancy survey timing, and the beam-column idealisation of flat-slab punching behaviour, are detailed in Section 3.7. Future research should extend ISRAF validation to at least one additional Nigerian state university to assess transferability. Structural health monitoring sensors in the four deficient theatres would enable continuous condition tracking and support probabilistic updating of deterioration models as data accumulate. Climate change projections for the Jalingo region should be incorporated into the carbonation model to assess how accelerating temperature and carbon dioxide trends will affect service-life predictions over the coming decades.

APPENDIX A

Reliability indices with 95 percent confidence intervals for the governing limit states. Table A1 reports the point estimate and 95 percent confidence interval for the flexural (LS1) and shear (LS2) reliability indices for each building, obtained by first-order propagation of the uncertainty in the NDT-derived mean and coefficient of variation of compressive strength, scaled to the three-core sample size.

***Table A1:** Reliability indices and 95 percent confidence intervals for the governing limit states*

Bldg.	β LS1 (95% CI)	β LS2 (95% CI)	Governing index and CC2 class
LT01	2.31 (2.09 to 2.53)	2.08 (1.81 to 2.35)	2.08, Deficient
LT02	3.28 (3.10 to 3.46)	3.04 (2.81 to 3.27)	3.04, Marginal
LT03	1.94 (1.72 to 2.16)	1.71 (1.44 to 1.98)	1.71, Deficient
LT04	3.82 (3.68 to 3.96)	3.64 (3.46 to 3.82)	3.64, Adequate
LT05	1.76 (1.54 to 1.98)	1.52 (1.25 to 1.79)	1.52, Deficient
LT06	3.24 (3.06 to 3.42)	3.02 (2.79 to 3.25)	3.02, Marginal
LT07	3.74 (3.60 to 3.88)	3.56 (3.38 to 3.74)	3.56, Adequate
LT08	2.18 (1.96 to 2.40)	1.94 (1.67 to 2.21)	1.94, Deficient
LT09	3.86 (3.72 to 4.00)	3.68 (3.50 to 3.86)	3.68, Adequate
LT10	3.94 (3.80 to 4.08)	3.77 (3.59 to 3.95)	3.77, Adequate
LT11	4.02 (3.88 to 4.16)	3.84 (3.66 to 4.02)	3.84, Adequate
LT12	4.08 (3.94 to 4.22)	3.91 (3.73 to 4.09)	3.91, Adequate

Note: Confidence intervals are wider for the deficient buildings (approximately plus or minus 0.22 for LS1 and plus or minus 0.27 for LS2) because their underlying material variability is higher and the three-core sample resolves it less precisely. Governing index is the minimum across all four limit states; CC2 class follows the bands defined in Table 4.

ACKNOWLEDGEMENTS

The authors acknowledge the support of the Physical Planning and Development Directorate of Taraba State University, Jalingo, which provided access to the lecture theatre buildings and cooperated fully with the field data collection programme. The contributions of the field inspection team and the laboratory technicians who prepared and tested the concrete core samples are gratefully acknowledged. This research received no specific grant from any funding agency in the public, commercial, or not-for-profit sectors.

REFERENCES

Abubakar, I. and Anigbogu, S. N. 2016. Structural assessment of reinforced concrete buildings in Nigerian public universities. *Journal of Building Engineering*, 8, pp.145-156. https://doi.org/10.1016/j.jobe.2016.07.006

Adeyemi, A. S., Olotuah, A. O. and Fadairo, G. 2021. Facility management of public university buildings in Nigeria: Challenges and implications for structural safety. *Journal of Facilities Management*, 19(3), pp.296-317. https://doi.org/10.1108/JFM-08-2020-0060

Agdas, D., Rice, J. A., Martinez, J. R. and Lasa, I. R. 2016. Comparison of visual inspection and structural health monitoring as bridge condition assessment tools. *Journal of Performance of Constructed Facilities*, 30(3), p.04015049. https://doi.org/10.1061/(ASCE)CF.1943-5509.0000802

Ahmad, S. 2017. Reinforcement corrosion in concrete structures and service life prediction: A review. *Construction and Building Materials*, 144, pp.247-258. https://doi.org/10.1016/j.conbuildmat.2017.03.232

American Concrete Institute. 2019. *ACI 318-19: Building Code Requirements for Structural Concrete and Commentary*. ACI, Farmington Hills, Michigan.

American Society of Civil Engineers. 2017. *ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings*. ASCE, Reston, Virginia.

American Society of Civil Engineers. 2022. *ASCE 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE, Reston, Virginia.

Asante, L. A., Oppong, R. A. and Mensah, S. 2021. Structural defect assessment of university buildings in Ghana: A risk-based approach. *Journal of Building Engineering*, 38, p.102198. https://doi.org/10.1016/j.jobe.2021.102198

ASTM International. 2015. *ASTM C876-15: Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*. ASTM International, West Conshohocken.

Aven, T. 2016. Risk assessment and risk management: Review of recent advances on their foundation. *European Journal of Operational Research*, 253(1), pp.1-13. https://doi.org/10.1016/j.ejor.2015.12.023

Bamigboye, G. O., Adeyemi, T. M. and Adedeji, A. A. 2020. Structural condition assessment of university buildings in southwestern Nigeria: Findings from a systematic field survey. *Engineering Failure Analysis*, 118, p.104942. https://doi.org/10.1016/j.engfailanal.2020.104942

British Standards Institution. 1985. *BS 8110-1: Structural Use of Concrete*. BSI, London.

British Standards Institution. 2019. *BS EN 13791: Assessment of In-Situ Compressive Strength in Structures and Precast Concrete Components*. BSI, London.

Bruneau, M., Chang, S. E., Eguchi, R. T., Lee, G. C., O'Rourke, T. D., Reinhorn, A. M., Shinozuka, M., Tierney, K., Wallace, W. A. and von Winterfeldt, D. 2003. A framework to quantitatively assess and enhance the seismic resilience of communities.

Earthquake Spectra, 19(4), pp.733-752. https://doi.org/10.1193/1.1623497

Cornell, C. A. 1969. A probability-based structural code. *Journal of the American Concrete Institute*, 66(12), pp.974-985.

Cutter, S. L., Ash, K. D. and Emrich, C. T. 2014. The geographies of community disaster resilience. *Global Environmental Change*, 29, pp.65-77. https://doi.org/10.1016/j.gloenvcha.2014.08.005

Dinku, A., Kebede, B. and Tiruneh, G. 2020. Structural performance evaluation of public university buildings in Ethiopia. *Journal of Civil Engineering and Construction*, 9(4), pp.195-208.

Ellingwood, B. R. 2005. Risk-informed condition assessment of civil infrastructure: State of practice and research issues. *Structure and Infrastructure Engineering*, 1(1), pp.7-18. https://doi.org/10.1080/15732470412331289341

Ellingwood, B. R. and Wen, Y. K. 2005. Risk-benefit-based design decisions for low-probability, high-consequence earthquake events in Mid-America. *Progress in Structural Engineering and Materials*, 7(2), pp.56-70. https://doi.org/10.1002/pse.191

European Committee for Standardization. 2002. *EN 1990: Eurocode: Basis of Structural Design*. CEN, Brussels.

European Committee for Standardization. 2002. *EN 1991-1-1: Eurocode 1: Actions on Structures. Part 1-1: General Actions*. CEN, Brussels.

European Committee for Standardization. 2004. *EN 1992-1-1: Eurocode 2: Design of Concrete Structures. Part 1-1: General Rules and Rules for Buildings*. CEN, Brussels.

European Committee for Standardization. 2017. *EN 206: Concrete. Specification, Performance, Production and Conformity*. CEN, Brussels.

Federal Emergency Management Agency. 2015. *FEMA 154: Rapid Visual Screening of Buildings for Potential Seismic Hazards (3rd ed.)*. FEMA, Washington DC.

fib. 2020. *fib Model Code for Concrete Structures 2020*. International Federation for Structural Concrete, Lausanne.

Frangopol, D. M. and Kim, S. 2019. Life-cycle of structural systems: Design, assessment, maintenance and management. *Structure and Infrastructure Engineering*, 15(8), pp.1025-1052. https://doi.org/10.1080/15732479.2019.1628119

Ghasemi, S. H. and Nowak, A. S. 2017. Target reliability for bridges with consideration of ultimate limit state. *Engineering Structures*, 152, pp.226-237. https://doi.org/10.1016/j.engstruct.2017.09.013

Hasofer, A. M. and Lind, N. C. 1974. Exact and invariant second-moment code format. *Journal of Engineering Mechanics*, 100(1), pp.111-121.

Helal, J., Sofi, M. and Mendis, P. 2015. Non-destructive testing of concrete: A review of methods and applications in developing country contexts. *Electronic Journal of Structural Engineering*, 14(1), pp.97-105.

Imam, B., Chryssanthopoulos, M. K. and Frangopol, D. M. 2019. Risk-based maintenance optimisation for aging masonry arch bridges. *Engineering Structures*, 174, pp.840-851. https://doi.org/10.1016/j.engstruct.2018.07.095

International Organization for Standardization. 2010. *ISO 13822: Bases for Design of Structures. Assessment of Existing Structures*. ISO, Geneva.

International Organization for Standardization. 2015. *ISO 2394: General Principles on Reliability for Structures (4th ed.)*. ISO, Geneva.

Islam, M. S. and Ahmed, S. J. 2020. Structural vulnerability assessment of educational buildings in Bangladesh using rapid visual screening. *International Journal of Disaster Risk Reduction*, 43, p.101389. https://doi.org/10.1016/j.ijdrr.2019.101389

JCSS. 2001. *Probabilistic Model Code*. Joint Committee on Structural Safety. Available at: www.jcss-lc.org (accessed March 2024).

Kent, D. C. and Park, R. 1971. Flexural members with confined concrete. *Journal of the Structural Division*, 97(7), pp.1969-1990.

Massey, F. J. 1951. The Kolmogorov-Smirnov test for goodness of fit. *Journal of the American Statistical Association*, 46(253), pp.68-78.

McCann, D. M. and Forde, M. C. 2001. Review of NDT methods in the assessment of concrete and masonry structures. *NDT and E International*, 34(2), pp.71-84. https://doi.org/10.1016/S0963-8695(00)00032-3

Mosalam, K. M., Takhirov, S. M. and Park, S. 2016. Applications of laser scanning to structures in laboratory tests and field surveys. *Structural Control and Health Monitoring*, 21(1), pp.115-134. https://doi.org/10.1002/stc.1550

Mutua, M. K. and Mboya, H. A. 2019. Assessment of structural condition of reinforced concrete buildings in Kenyan public universities. *Journal of Civil and Environmental Engineering*, 11(4), pp.385-398.

Nigerian Institute of Quantity Surveyors. 2024. *Schedule of Rates for Building Works, Taraba State*. NIQS, Abuja.

BJET, 2026, Vol. 21(3), 50-68

Nigerian Meteorological Agency. 2022. *Annual Meteorological Summary: Jalingo Station 2010-2021*. NIMET, Abuja.

Nowak, A. S. and Collins, K. R. 2013. *Reliability of Structures (2nd ed.)*. CRC Press, Boca Raton.

Obeng-Odoom, F. 2016. Urban infrastructure maintenance in Ghana: A political economy analysis. *Habitat International*, 51, pp.39-46. https://doi.org/10.1016/j.habitatint.2015.10.006

Ogunbayo, B. F., Aigbavboa, C. O., Oke, A. E. and Mojekeh, M. O. 2022. Review of study on concrete structural failures and reliability of concrete structures. *Crystals*, 12(6), p.860. https://doi.org/10.3390/cryst12060860

Olanrewaju, A. L., Idrus, A. and Khamidi, M. F. 2018. Investigation of building maintenance management practices in Nigerian universities. *Structural Survey*, 29(3), pp.251-265.

Oyekan, G. L. and Kamiyo, O. M. 2018. Effects of tropical climate on the durability and structural integrity of reinforced concrete buildings in West Africa. *Engineering Science and Technology*, 11(2), pp.366-375.

Papadakis, V. G., Vayenas, C. G. and Fardis, M. N. 1992. Fundamental modelling and experimental investigation of concrete carbonation. *ACI Materials Journal*, 88(4), pp.363-373.

Rackwitz, R. 2001. Reliability analysis: A review and some perspectives. *Structural Safety*, 23(4), pp.365-395. https://doi.org/10.1016/S0167-4730(02)00009-7

Saaty, T. L. 1980. *The Analytic Hierarchy Process: Planning, Priority Setting, Resource Allocation*. McGraw-Hill, New York.

n. Routledge, London.

South African National Standards. 2011. *SANS 10160: Basis of Structural Design and Actions for Buildings and Industrial Structures (SB-10Ed)*. SABS, Pretoria.

Stewart, M. G. and Melchers, R. E. 2004. *Probabilistic Risk Assessment of Engineering Systems*. Springer, London.

Stewart, M. G. and Val, D. V. 2020. Climate change impacts on the structural reliability and service life of reinforced concrete structures. *Structure and Infrastructure Engineering*, 16(6), pp.789-803. https://doi.org/10.1080/15732479.2019.1655753

Val, D. V. and Stewart, M. G. 2003. Life-cycle cost analysis of reinforced concrete structures in marine environments. *Structural Safety*, 25(4), pp.343-362. https://doi.org/10.1016/S0167-4730(03)00014-6

Vecchio, F. J. and Collins, M. P. 1993. Compression response of cracked reinforced concrete. *Journal of Structural Engineering*, 119(12), pp.3590-3610. https://doi.org/10.1061/(ASCE)0733-9445(1993)119:12(3590)

Vu, K. A. T. and Stewart, M. G. 2000. Structural reliability of concrete bridges including improved chloride-induced corrosion models. *Structural Safety*, 22(4), pp.313-333. https://doi.org/10.1016/S0167-4730(00)00018-7

Windapo, A. O. and Cattell, K. 2013. The South African construction industry: Perceptions of key challenges facing its performance, development and growth. *Journal of Construction in Developing Countries*, 18(2), pp.65-79.

Wisner, B., Gaillard, J. C. and Kelman, I. 2012. *Handbook of Hazards and Disaster Risk Reductio*