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Reliability Appraisal of Nominal Eccentricity of Short Reinforced Concrete Column Designed to BS 8110 and Eurocode (EN: 2)-Ultimate Limit State on Fatigue

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ABSTRACT

Nowadays assessment of fatigue and reliability of structures have increased dramatically. This is confirmed by the recommendations of the standards and constitutive model of concrete in which the rules and requirements to ensure safety, serviceability and durability of the structure are stated. This study is directed to the reliability assessment of reinforced concrete column and fatigue comparison of (BS: 8110, 1997) and Eurocode 2 (EN: 2, 2004) ultimate limit state requirements on nominal eccentricity of short column resisting moments and forces. The column was modelled as one end fixed to resist moment reaction and free at the other end. It was then examined on fatigue and probabilistically assessed with the variables relating to the uncertainty loading conditions. The First-Order Reliability Method (FORM 5) encoded in CalREL was employed to estimate the implied probability of failure by varying load ratio and reinforcement ratio. And was verified with numerical simulation on CONCRETE MODEL OF 3 Dimension (COM 3). The results obtained have shown that the column assessed lost its flexural and shear carrying capacity gradually as the percentage load increased especially at the joint. Reinforced concrete column's performance may be dependent on the applied load and could fail if it carries a lot more than the designed loads. It is therefore necessary to perform fatigue investigation to double check the resistant capacity of the column.

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1. Introduction

Many uncertainties can be found in all real-life problems. In the light of present state of ambiguities in the various factors required for the assessment and design of structures, it is hence worrisome to measure the absolute safety of structures using deterministic approach. Therefore, one of the significant ways for ensuring the safety of the structures quantitatively is its reliability or probability of failure [1]. Reinforced concrete structures need to satisfy the requirements for safety and serviceability for an appointed time. Such demands give details on high risk events like the total collapse of a structure also events less severe such as maximum deflection or vibration requirements [2]. Reinforced concrete structures designed and erected before modern seismic code came into place in the early 1970's and is prone to collapse during natural disasters. Standards adoption in structural reliability is scanty around the world [3–7]. More so, ability to reduce design estimation to safe cost and produce efficient and elegant structures cannot be undermined hence, this study compares the risk associated in adopting (BS:8110, 1997) and Eurcode (EN:2, 2004), codes for design of concrete structures, to check reliability of concrete column under uncertain loadings when nominal eccentricity of short column resist moments and axial forces.

Engineering community, users and owners of buildings always expect structure and its foundation to be designed with a reasonably safe margin. In practice, these expectations are achieved by following the provisions in the design codes which is based on experience, practice and judgment, [8]. However, this approach lacks systematic basis for evaluating the degree of conservativeness and may result in inadequate or uneconomical designs. To assess the safety and enforce the safe margins, it is essential to identify all major sources of uncertainties associated with the analysis and designs of structural systems.

Reliability is the probability that a system will perform its proposed function over a specified period under specific operating conditions [1]. Hence, it is very important to minimize probability of failure before construction of a reinforced concrete. In assessing the uncertainty in the concrete column under the implied loads, CalREL, a universal-multipurpose structural analysis program was adopted. Its aim is to ascertain how reliable or otherwise is the failure probability of structural systems. The different methods of reliability analysis are first order Reliability Method (FORM), Second order Reliability Method (SORM), and Monte Carlo Simulation (MCS) [9]. In a reliability-based study, uncertainties that are associated with the characteristics of materials, environmental factors, loads etc. are considered by treating the parameters as random variables or processes, [10]. In this investigation, reliability assessment of the column was performed using FORM which is programmed in FORTRAN programming language with all the variables generated during calculation. There is counterpart to reliability called probability of failure (P_f). It is defined as the probability that a structural system will fail under the given load conditions. Hence, reliability and probability of failure form two extremes related to the safety of structural systems. Probability theory states that the sum of reliability and probability of failure is always equal to unity [4]. This rule makes it possible to evaluate one quantity if the other is known.

2. Fatigue analysis model

Material constituents of concrete always contribute to its behavior when load is applied [11], especially during hydration of cement with mineral particles which form the micro pore structures that contributed to fatigue behavior. Direct-path integral scheme for fatigue simulation under constitutive laws that take account of small amplitude, but high fatigue has been formulated, [12]. This considered compression along cracking, tension normal to cracks and shear transfer along crack plane. Concrete loses its capacity gradually under uniaxial tension and shear, reinforcement start yielding when stresses are transferred close to the crack region, however, some steels remain elastic because of bond interaction. This is the constitutive law of concrete under fatigue [13]. Fatigue is a localized and advanced ways in which structural damages amass ceaselessly owing to the application of external loadings such as vehicles for concrete bridges, winds for high-rise structures, waves for offshore platforms, temperature for turbine engines and seismic activities for concrete structures [14]. Fatigue is one of the damages potentially observed in concrete elements which are most critical. Concrete located at earthquake-prone regions are more vulnerable to seismic collapse, some of the structures subjected to ground motion around the earthquake inclined area such as Turkey, Japan etc. Have been examined [15,16].

2.1 Structural design process

The aim of Structural design according to both BS: 8110 and Eurocode 2 (EN, 2004) "is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an acceptable degree of safety, they should sustain all the loads and deformations of normal construction and use, as well as have adequate durability and resistance to misuse and fire." Structural design of a building involves the determination of various types, sizes and locations of structural elements namely; foundations, columns, beams, and slabs, and the amounts of steel, which can safely combine with concrete to sustain the loads of the building – where reinforced concrete is the material used [17–23]. Standard professional practice is adhered to using relevant codes of practice. In Nigeria, the British standard codes are generally used however, the use of Eurocodes give more room for material economy and adequate capacity. The later have been adopted lately in parts of the world, some of the advantages offered is now being espoused in Nigeria.

2.1.1. Limit state of structural reliability

Limit state is the state at which a structure's performance changes from acceptable to unacceptable. Reynolds *et al.,* (2008) [21], explained that the permissible stress method in which designs are obtained by applying a safety factor to the ultimate strength of materials does not possess some inconsistencies resulting from the arbitrary way the permissible stress is defined and that it is not suitable for semi-plastic materials, non-linear structures and stability of structures subjected to overturning forces. The introduction of probability-based safety analysis gives room for advancements in the former methods of obtaining probability of failure through systematic analysis of the uncertainties in all variables. This proves better than the permissible stress and load factor methods which does not consider rational reliability concept in their derivation, although structural design through the application of safety factors is necessary [24–28]. Hooman *et al.,* (2017) [29], proposed non-normal reliability indices for the limit state function according to convolution theory using Gaussian function. Target reliability with reference to experience are sometime used to adjudge the cost-failure function of structures during construction [30].

The traditional design concept of system account for the limit state performance transforms from satisfactory to unsatisfactory. There are several types of limit states: serviceability limit states, ultimate limit state, and serviceability limit state of fatigue. Any of these conditions can be defined by limit state function given in eqn. (1).

$$g(x_i) = R - D \quad (1)$$

The structural resistance, R and its demand response, D, are both function of design variables which are random in nature. A structure is said to be save when the value of D is less than R, [28]. The performance function $g(x_i)$ can also be expressed as given in eqn. (2)

$$g(x_i) = g(x_1, x_2, x_3, \ldots, x_n) = 0 \quad (2)$$

where 'x' represents the basic design random variable.

If $g(x_i) < 0$, it leads to unauthorized breakage of constructions and its performance, and if $g(x_i)) \geq 0$, the structures performance is satisfied. In the case of ultimate limit states, R represents structure capacity, while D represents the load. In the case of serviceability limit states, R may represent a maximum allowable deflection of the structure, while D can represent deflection under load. However, the limit state function may be more complex (e.g. nonlinear) and its parameters can be variable in time hence the risk association is high.

Since R and D, are random variables, their ratio must also be random in nature; this is implied by the existing decimal places in the value which renders the deterministic approach valueless in actualizing appropriate factor of safety, F_s, this is always leads to over or under design of engineering component.

Hence, eqn. (1) can be rewritten as given in eqn. (3).

$$R \geq F_s D \quad (3)$$

F_s is always preferred since the nominal values of both S and R cannot be determined with certainty [25]. In addition, since the two variables are normal random distribution, their product will also produce a random variable of normal probability distribution. Hence the factor of safety F_s is reviewed to be a random variable as given in eqn. (4).

$$Y = \frac{D}{R} < 1 \quad (4)$$

Because most engineering designs are made without complete benefit information, the aim of reliability design entails the realization of acceptable probability that a design system will fulfill its intended purpose within the limit of economy and under uncertainty condition [31].

If the system fails, the probability of failure according to [2] is given in eqn. (5) as:

$$P_f = P(X \in F) = P(g(x_i) \leq 0) = \int_{g(x_i)\leq 0} dF_x(x) \tag{5}$$

However, the probability of survival or reliability is given in eqn. (6) as;

$$R_v = 1 - P_f \tag{6}$$

If R_v=0, the system is a total collapse. This will happen when the maximum demand of the system D_{max} surpasses the minimum strength R_{min}, which makes both the distribution to coincide. But if P_f= 0, the system is trustworthy.

3. Methodology

3.1. COM 3 model of concrete column

Finite element model of a typical concrete column subjected to axial loading was made Figure 1, its constituent's concrete materials and properties as per BS:8110 and EN:2004 was adapted to the COM 3, a quantitative as well as constitutive simulating machine. The geometry, characteristic strengths and applied load of the column are as stated in section 3.3, for COM 3 model, reinforcement ratio of 1.55, estimated reinforcement diameter of 16 mm and total length of 2 m were considered. The column was modelled monolithically with the rigid footing to resist free movement during application of load.

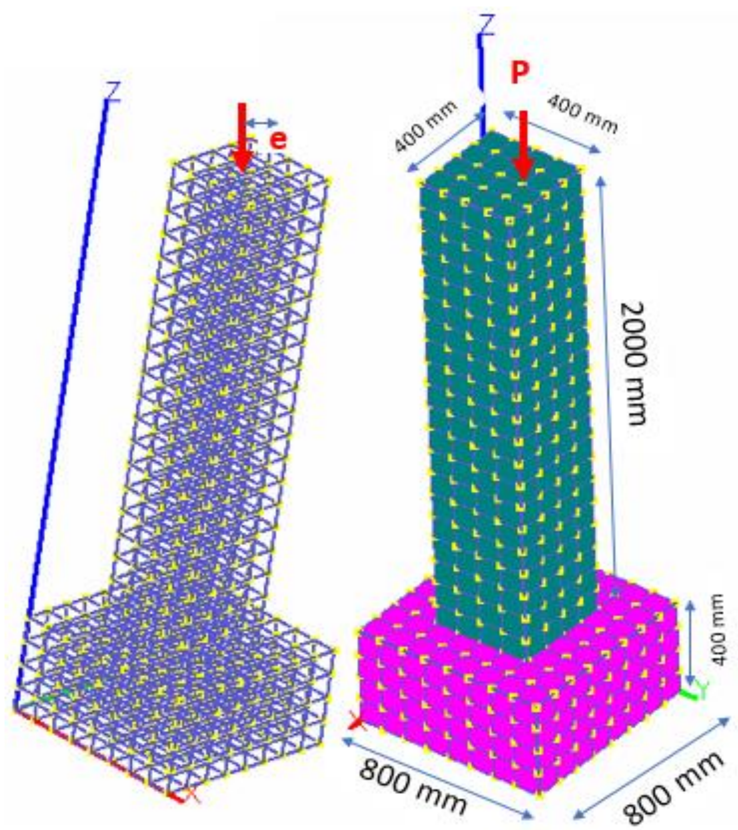


Fig. 1. COM 3 FEM Model of Concrete Column.

3.2. Data generation

In order to effectively estimate the level of the probability of failure on nominal eccentricity of short columns resisting moments and axial forces under varying loading conditions, load combination of both live and dead loads at adverse effect as specified by BS:8110 and (EN:2) were considered. The load combinations are as given in eqns. 7 and 8 respectively. $G_{k,}$ is the dead load which constitute the self weight of the column and Q_k is the imposed load.

$$W_{BS} = 1.4G_k + 1.6Q_k \tag{7}$$

$$W_{EN} = 1.35G_k + 1.5Q_k \tag{8}$$

Generally, the number of limit-state functions and the number of independent groups of basic random variables need to be defined based on the assigned values. Some of the statistical data generated by the distribution formulae (eqns. 9 and 10) routed in the CalREL analysis software are presented in Table 1, deterministic data are defined as normal while stochastic data that affect the column due to variations in characteristics data are defined as lognormal.

$$f_x(x) = \frac{1}{\sigma\sqrt{2\pi}} exp\left(\frac{-1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right) \tag{9}$$

$$g_x(x) = \frac{1}{x\xi\sqrt{2\pi}} exp\left(\frac{-1}{2}\left(\frac{1nx-\lambda}{\xi}\right)^2\right) \tag{10}$$

Eqns. 9 and 10 are the normal and lognormal statistical formulae modelled in CalREL, λ is the jump rate which only capture the long-time behaviour of the stochastic variable 'x'. The standard deviation and mean σ, μ are greater than zero. λ is normally assigned a unit value, and ξ=2.

Table 1
Statistics of design variables for short braced reinforced concrete columns.

Variables	Probability Density function	Mean	Standard deviation	Coefficient of variation (%)
Breadth, b (mm)	Normal	400.00	40.00	10
Height, h (mm)	Normal	400.00	40.00	10
Strength of concrete, f_{cu} (N/mm^2)	Lognormal	20.00	6.00	30
Strength of steel, f_y (N/mm^2)	Lognormal	410.00	123.00	30
Reinforcement ratio, ρ	Lognormal	3.00	0.9	20
Ultimate load, N_l (KN)	Lognormal	2083	624.74	25
Load ratio (α)	Lognormal	10	1.1	5

3.3. Nominal eccentricity of short columns resisting moments and axial forces

With the load combination in eqns. 7 and 8, the ultimate loading capacity of a short column with nominal eccentricity according to BS: 8110 and (EN: 2) is presented.

The ultimate loading capacity with a nominal eccentricity as given in (BS: 8110, 1997) is;

$$N_{uz} = \left(\frac{0.6f_{cu}}{\gamma_c}\right)A_c + \left(\frac{0.84f_y}{\gamma_s}\right)A_{sc} \tag{11}$$

The factor of safety for concrete γ_c = 1.5 and for steel γ_s = 1.05 as specified by BS: 8110, Hence, eqn. 11 is rewritten as given in eqn. 12.

$$N_{uz} = 0.4f_{cu}A_c + 0.8f_yA_{sc} \tag{12}$$

with $A_{sc} = \frac{\rho bh}{100}$ (13)

then equation (12) becomes,

$$N_{uz} = 0.4\text{bh}\,(f_{cu} + 2.00\text{x}10^{-2}f_y\rho) \tag{14}$$

the ultimate loading capacity with a nominal eccentricity as given in (EN:2) is;

$$N_{ed} = \left(\frac{0.75f_{cu}}{\gamma_c}\right)A_c + \left(\frac{0.843f_y}{\gamma_s}\right)A_{sc} \tag{15}$$

The factor of safety for concrete γ_c = 1.5 and for steel γ_s = 1.15 as specified by (EN:2), Hence;

$$N_{ed} = 0.5f_{cu}A_c + 0.733A_{sc}f_y \tag{16}$$

Adopting process of eqn. 13 and substituting the values in Equation (16), we have:

$$N_{ed} = 0.5\text{bh}\left(f_{cu} + 1.4\text{x}10^{-2}\rho f_y\right) \tag{17}$$

f_{cu}= Characteristic strength of concrete

A_{sc} = Area of steel reinforcement

f_y = yield strength of steel

A_c = Area of concrete member

ρ = percentage reinforcement ratio

For the examined column, the following data was assumed.

b = 400mm, A_{sc}= 754mm^2, h = 400mm, f_{cu}=20N/mm^2, f_y = 410N/mm^2.

Substituting the values to eqns. 12 and 16 respectively. The magnitude of the ultimate loading can be derived as given in eqns. 18 and 19 respectively.

N_{uz} = 3333.608 kN (18)

and N_{ed} = 1826.6 kN (19)

The value gotten serves as the demand (D) on a column. The capacity, R, of the ultimate load for nominal eccentricity column resisting moments and forces are as presented in eqns. 14 and 17.

Conditions for checking the performance of a column are as stated in eqns. 1 to 6. To ensure safety of the structure, the quantitative values of R must always supersede the values of D otherwise, failure of the structure could be imminent. In order to estimate the implied probability of failure, eqns. 14 and 18, 17 and 19 for both BS: 8110 and EN: 2 are computed as given in eqns. 20 and 21 respectively.

$$p_f = P[0.4\text{bh}(f_{cu} + 2.00\text{x}10^{-2}f_y\rho) - \alpha N_{uz}] < 0 \tag{20}$$

$$P_f = P[0.5bh(f_{cu} + 1.4 \times 10^{-2}\rho f_y) - \alpha N_{ed}] < 0 \quad (21)$$

α is the percentage of ultimate load referred to as load ratio, the percentage ratio used was varied from 10% to 100% by a step of 10 and the percentage reinforcement ratio, ρ, used is as specified by both BS:8110 and EN:2 which ranges between 0.55 and 3.0. Variation of 0.5was adopted.

Eqns. 20 and 21 can also be rewritten as;

$$P_f = P\left[1 - \frac{\alpha N_{uz}}{0.4bh(f_{cu}+2\times10^{-2}\rho f_y)}\right] < 0 \quad (22)$$

$$P_f = P\left[1 - \frac{\alpha N_{ed}}{0.5bh(f_{cu}+1.4\times10^{-2}\rho f_y)}\right] < 0 \quad (23)$$

Statistical data presented in Table 1 were also adopted in modelling and investigating the probability of failure in eqns. 22 and 23. The probability of survival or reliability as stated in eqn. 6 is adopted to evaluate the level of survival of the column

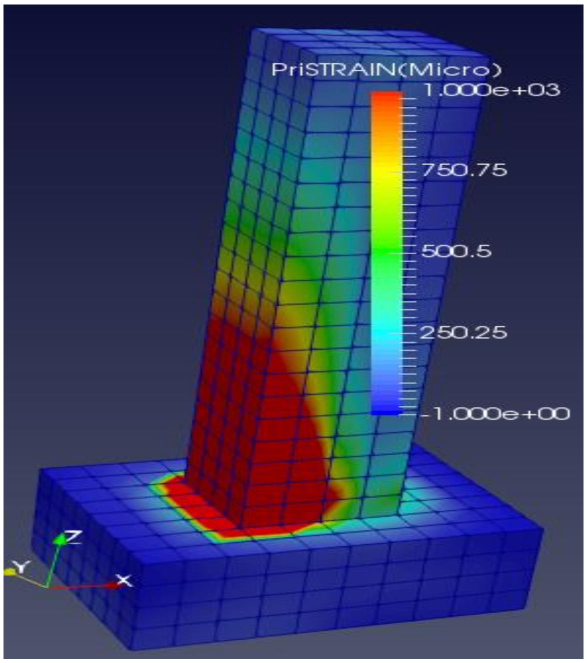
4. Discussion of results

4.1. Failure mode of concrete column

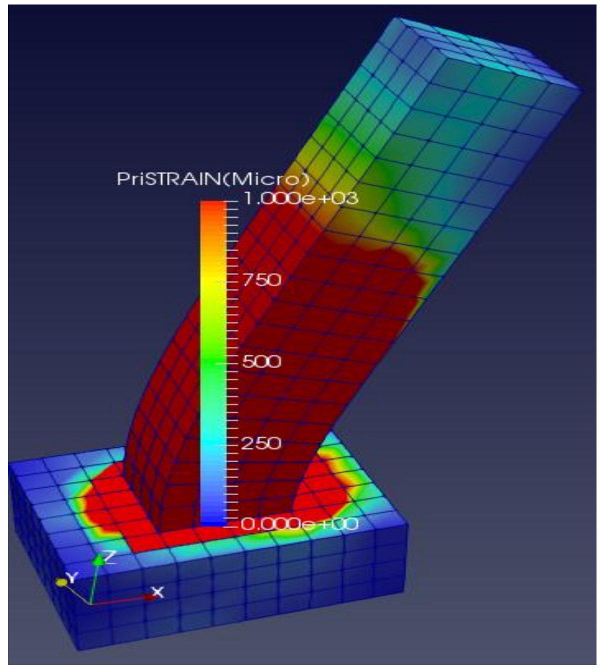
Figure 2 is the total damage observed after applied load of 200kN, this being the final approximate load the modelled column could withstand during analysis. The strain effect of the column shown in Figure 2a, around the joint element (footing and column), it is observed that spalling of concrete cover at the base of the column has set in and gradually transformed towards the middle portion of the column with signs of shear failure. Figure 2b, is the failure mode of the concrete column. Flexural cracks are observed predominant to shear cracks, although the basis of simulation could not adjudge the difference in the rate. It is also likely that pull out of reinforcing bars around the bottom of the column is a severe one, which could be smear cracks of concrete. In addition, total stiffness has been jeopardized owing to gradual loading of column even though the footing was made rigid in the analysis model, Figure 2c, captured some movement of the column as indicated by the red colour. This is an indication of reduced contact density between the two joint elements since column stiffness is gradually reduced because of applied load.

Figure 3 shows the load against the displacement response of the column. Comparing the plots of both BS: 8110 to EN:2, it can be seeing that the displacement increases as the load increases up to 200 kN. As concrete is an elastic material, there exist rebound in the loading, however displacement keep increasing. Elongation of EN:2 plot is higher than BS:8110 which is a pointer to the ability of the former to sustain more deflection before failure. The decrease in the curve indicates the strain softening of the concrete column.

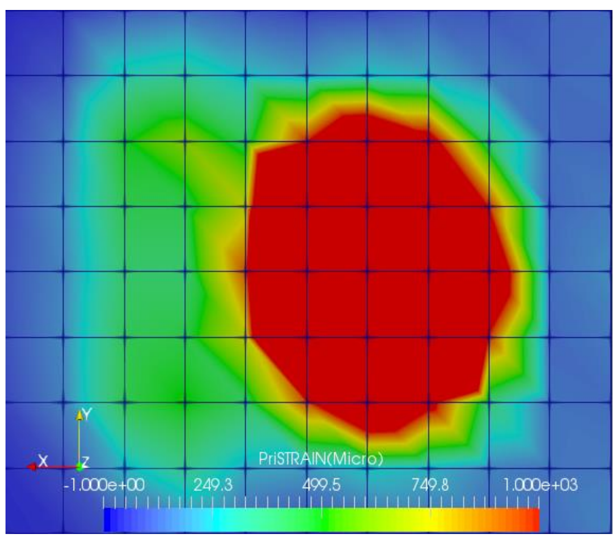
The proportional stress-strain curve, Figure 4 is the comparison between BS: 8110 and EN: 2. At early stage on the plots, there is a linear relationship between the stress and the strain which depict elastic behaviour, immediately after which the strain hardening set in because of the presence of reinforcing bar in the column. Elasto-plastic behaviour of the reinforced concrete column is also observed, (see Figure 2b) i.e. gradual decrease in material stiffness with increase irrecoverable deformation. Many structures in service today exercise the same behavioural pattern due to increase usage demand, and long-term deterioration of structures is associated with fatigue.



a) Strain effect of the column



b) failure mode of the column



c) Residual strain in Longitudinal direction at the bottom of column

Fig. 2. Damage accumulation during loading.

Observing the behaviour of the reinforced concrete column in Figures 3 and 4, EN: 2 could be adjudge more conservative than BS: 8110. The safety margin is quite visible as there is increase in deformation when elastic behaviour of the concrete material disappears.

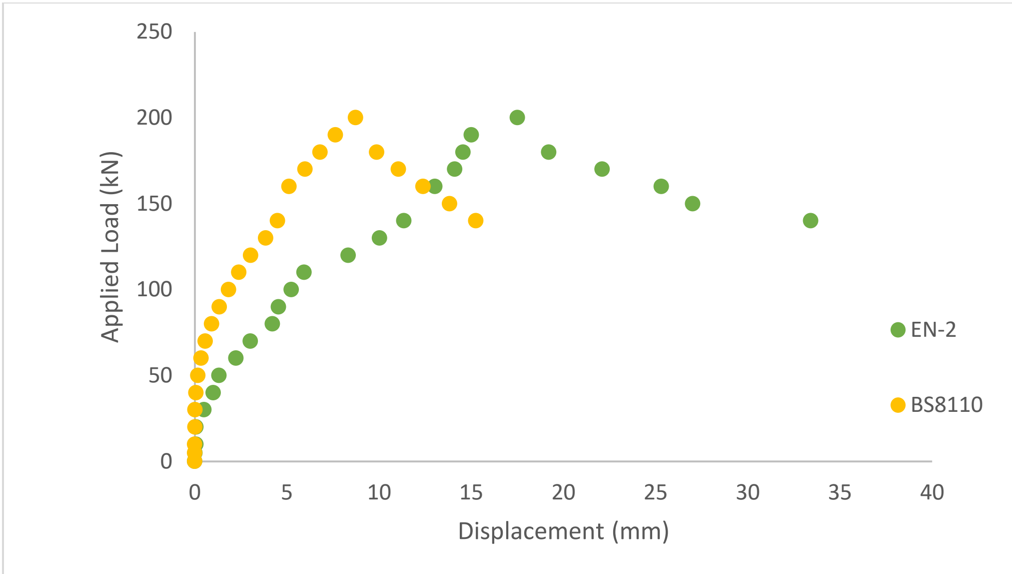


Fig. 3. Load- displacement comparison curve between BS: 8110 and EN:2.

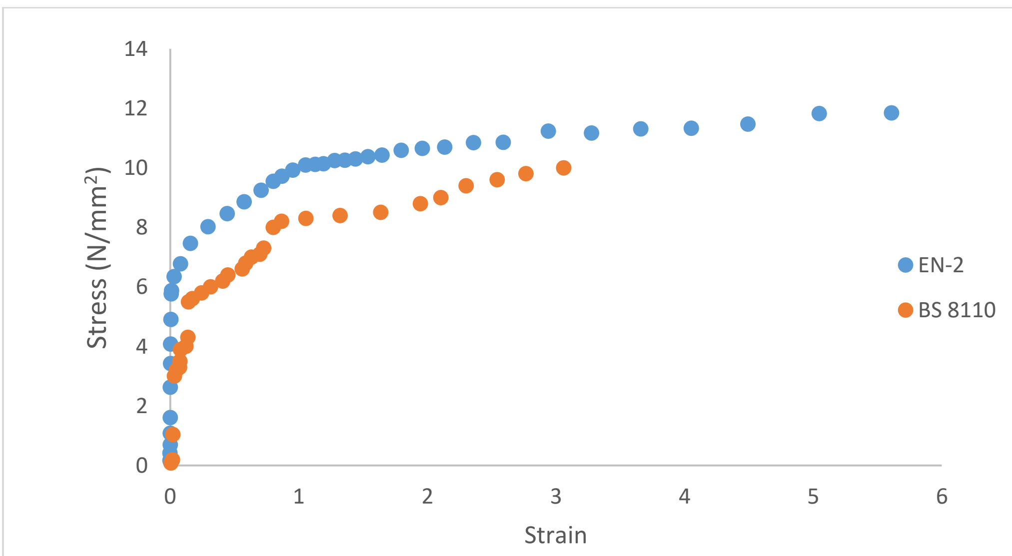


Fig. 4. Stress-Strain comparison curve between BS: 8110 and EN: 2.

4.2. Result of reliability analysis of nominal eccentricity of short columns resisting moments and axial forces

One of the ways to quantify the safety of a structure is to assess its reliability index ‘β’or otherwise, its failure can be assessed by the probability of failure, p_f. Probabilistic assessments were performed on the short column resisting moment and axial forces with nominal eccentricity based on the extrapolated limit state equations. The results derived from the analyses evaluation are discussed.

Figures 5 and 6 (for BS:8110 and EN:2 respectively) have been plotted to show the variation of the safety indices’β’ against the percentage load ratio ‘α’ at varying reinforcement ratio ‘ρ’. The plots exercise similar tendency; there is a general decrease in reliability indices as the percentage load ratio increases for all the considered reinforcement ratios. It is surprising that ρ of 0.55 has the highest reliability index, β, follow by ρ of 1.05, while ρ of 3.00 has the lowest reliability index. This could imply that over estimation of reinforcement ratio in the concrete column can have a catastrophic on the performance of the concrete.

Comparing the level of reliability of the two standards, BS: 8110 has the higher safety indices. Furthermore, based on the assumed target reliability index, β_T, of 3.0 proposed by the joint committee on structural safety (JCSS, 2005), percentage reinforcement ratios from 1.55 to 3.00 fail to meet the target for the two standards.

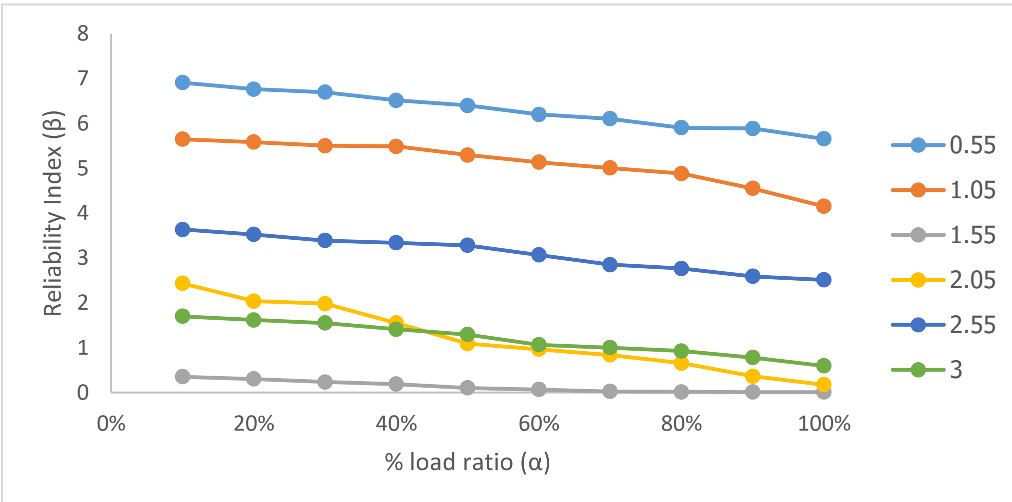


Fig. 5. Reliability index (β) against percentage load ratio (α) at varying reinforcement ratio (ρ) for BS 8110.

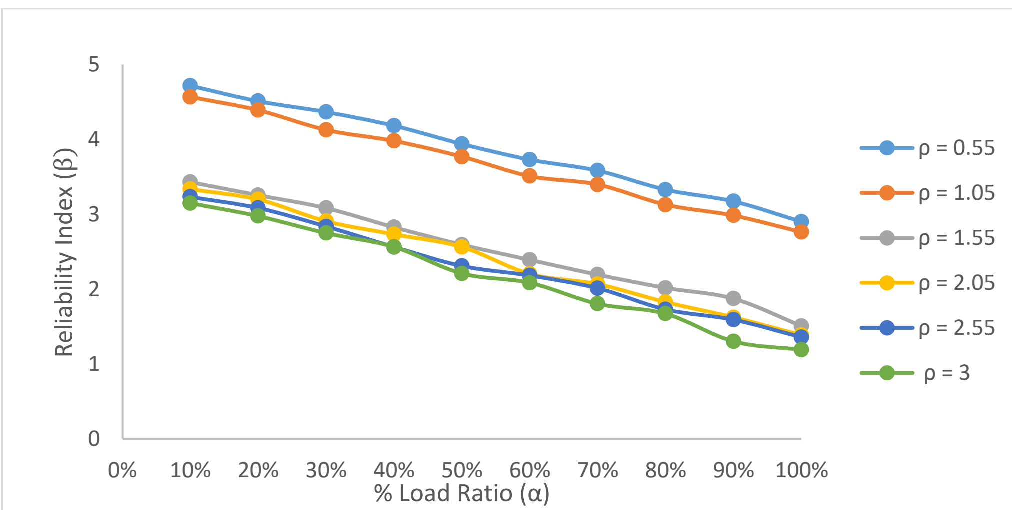


Fig. 6. Reliability index (β) against percentage load ratio (α) at varying reinforcement ratio (ρ) for EN: 2004.

In order to examine the dependency of the column to its resistance capacity, Figures 7 and 8 (for BS: 8110 and EN:2 respectively) are presented. It can be seen that as the percentage load ratio 'α' is increasing at any given reinforcement ratio, reliability index is decreasing for the two surface curves presented. Furthermore, an increase in the reinforcement ratio ‘ρ’ at any percentage load ratio also leads to decrease in reliability index ‘β’. It is obvious that the resistance properties play a significant role in the performance of the concrete column most especially when the concrete column is subjected to fatigue analysis. Column will therefore fail if it carries more than the design capacity. Geometrical properties like the one specified for the design and analysis of the column in this investigation can also contribute to quick deterioration of the resistance capacity. Although these cross sections are acceptable for design, experimental check can be necessary for more robust sections.

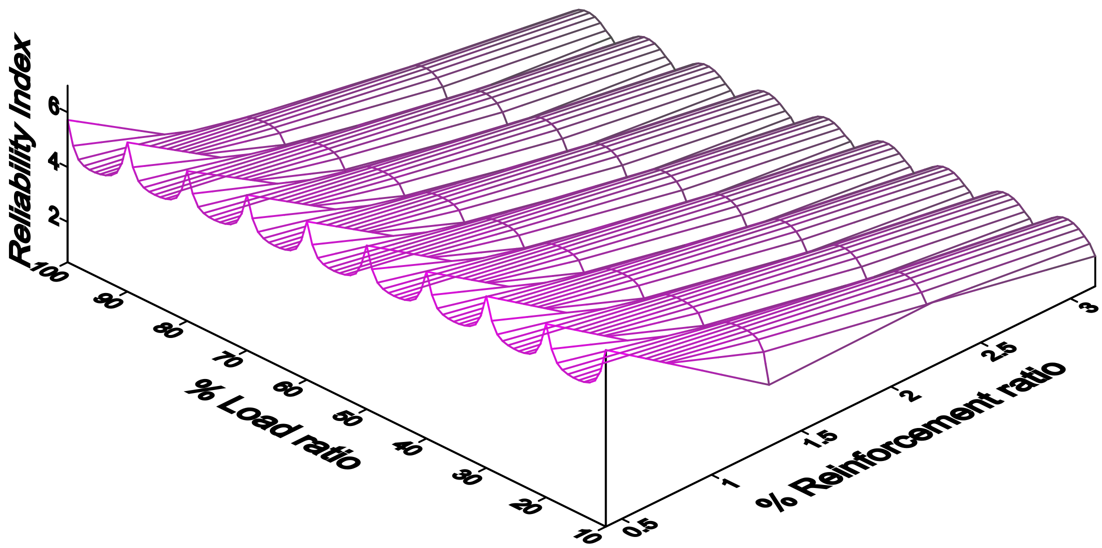


Fig. 7. 3D surface plot of interaction among Variation of β with ∝ and ρ for a nominal eccentricity of short column resisting moment and axial forces. BS 8110.

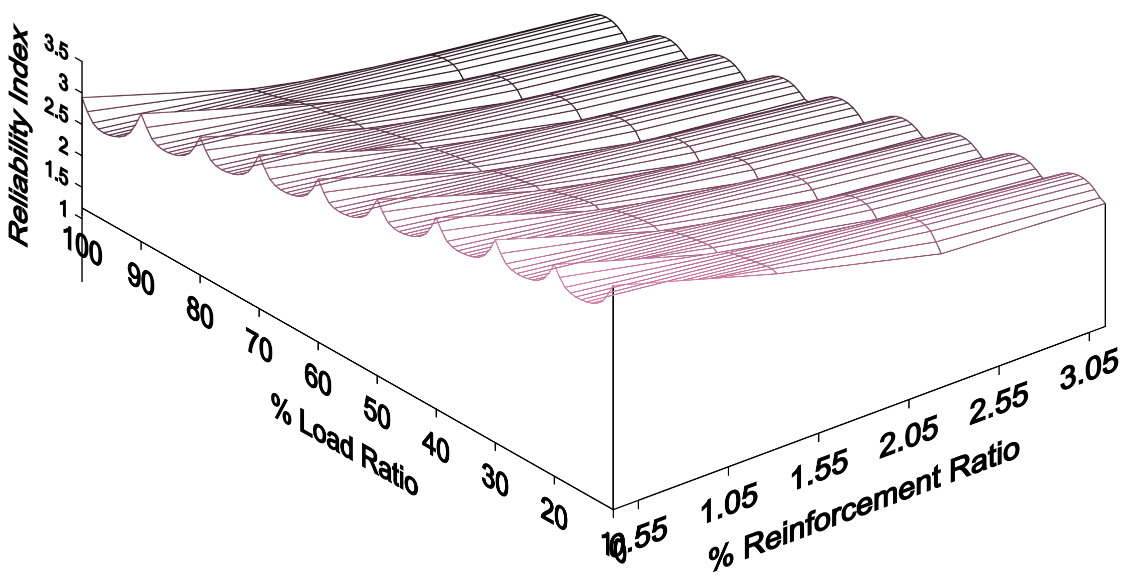


Fig. 8. 3D surface plot of interaction among Variation of β with ∝ and ρ for a nominal eccentricity of short column resisting moment and axial forces. EN: 2004

5. Conclusions

The ultimate limit state design requirement of British Standard (BS: 8110,1997) and Eurocode 2 (EN: 2, 2004) for reinforced concrete columns has been modelled with material properties, geometry and loading as well as assessed under probabilistic/safety conditions, when factors affecting it life expectancy were defined as random. The investigation is limited to reliability and fatigue assessment of nominal eccentricity of short column resisting moments and forces.

The First Order Reliability method (FORM) was employed in determining the measure of safety. For the two standards considered, the results of the analysis have shown that the columns loss their carrying capacity gradually as the load increases. Columns will therefore fail if they carry a lot more than the designed loads. In addition, an increase in reinforcement ratio ρ could have a drastic impart on the performance of the column considered. The fatigue analysis revealed bond deterioration of the RC column and reduction in the total stiffness. This effect is common to many old structures in service nowadays, increase in structural performance has been jeopardised due increase in usage. Fatigue assessment of in-service structures is essential.

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