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SUITABILITY ASSESSMENT OF THE PARTIAL SAFETY FACTORS OF CP110 USED IN THE DESIGN OF ULTIMATE STRENGTH OF REINFORCED CONCRETE COLUMNS

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ABSTRACT

Empirical data were used to assess the suitability of the partial safety factors in the Code of Practice CP110 which are often used in the design of the ultimate strength of reinforced concrete section. To do this, the effects of short term variables that affect the ultimate strength of a rectangular column section was studied using Monte Carlo technique by the input of statistical parameters of these variables into a computer program designed for the computation. Simulation of 1000 columns at selected eccentricities was done and the distribution of strength was determined. The interaction curve obtained from this simulation was compared with the theoretical curve obtained when the nominal values of the parameters were used. It was found that the safety factors varied at different eccentricity from those obtained when the nominal values were used for the computation of strength. This in turn calls for additional strength reserve in the use of the curve provided for in the code.

INTRODUCTION

The ultimate goal of the client and that of the structural designer is to achieve an absolutely safe structure, but in reality, no structure can be said to be absolutely safe though all efforts are made towards achieving this goal. One of such efforts is the use of safety factors both for the materials and loadings in the computation of the ultimate strength. Mosley and Bungey (1999) observed that theoretically it could be possible to derive mathematically the probability of reaching the ultimate limit state but paucity of adequate data makes this unrealistic and the values often adopted are based on experience and simplified calculation. According to Heyman (1973) "...the empirical assignment of partial safety factors in this way may seem sensible and acceptable; in the absence of precise information it is right to make use of experience. But it is wrong to forget that the numerical work has been arranged empirically and to come to the belief that the values of partial load factors found to give good practical results actually correspond to a real state of loading." Also in CP110, it is left to the designer to in critical cases specify appropriate tolerances or use a higher value for the partial safety factor for load. It is difficult to determine how realistic the use of higher safety factor are and to what extent their nonobservance affect safety.

Due to the uncertainty in the derivation of the partial safety factors, it is important to check if the recommended values given in the CP110 are of general application. Most designers adopt the values for the partial safety factors in the British code of practice without due consideration to local constraint such as the degree of inaccuracy in construction sites. In this paper the partial safety factors for loads is not considered, emphasis are on the materials, hence the partial safety factors provided in CP110 for the materials are excluded, that is, the stress-strain curve for concrete was assumed as

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parabolic and rectangular up to strain of 0.0012 and 0.0035 respectively due to the exclusion of $y_m = 1.5$ and the stress-strain curve for high yield steel was assumed as in CP110 but without partial safety factor for steel $y_m = 1.15$. For mild steel, it can be assumed as elastoplastic relationship.

A rectangular column with cross – sectional dimension of 300 x 600mm was considered for the study. The structural design of this column was based on the provisions of the British code of practice CP110 where several nominal parameters were used for the calculation of the ultimate strength. The nominal parameters considered were cross sectional dimension of concrete, cover to reinforcement, eccentricity due to non verticality of column and the strength of materials (steel and concrete) to compute the ultimate strength based on what is practically attainable on the construction sites. The probability models for these parameters which are used in the assumptions for the simulation were as a result of published work by Owoyale et al (2003) hence these models should be considered preliminary and could be modified when more data are available.

All the nominal parameters used by the designers in the calculation of ultimate strength are not always equal to the values adopted from the statistical point of view, the behavior of geometric properties and the strength of materials are all random variables and no single value could be used to represent these parameters. Since the probability models of these parameters are known and a deterministic relationship exists between the ultimate strength of a section and these parameters as contained in the British Code of Practice, it is possible to use randomly selected values for the parameters to compute the variability of the ultimate strength of the section. The purpose of this paper is twofold (1) to describe the method for evaluating the relationship which exists between axial load, moment for a given concrete cross – section thus providing an analytical tool and (2) to compare the interaction curve obtained from the simulated strength with the theoretical interaction curve where nominal values were used.

METHODOLOGY

The axial load – moment interaction curve may be visualized as the locus of pairs of load and moment values that according to ultimate strength theory will cause failure. The interaction diagram has two distinct regions. At small load eccentricities, failure occurs when the concrete crushes before the steel has yielded termed a compression failure. At large eccentricities, the reinforcement yields first, followed by a secondary compression failure in the concrete, termed a tension failure. The balanced point is that point where the tension steel yields and the concrete crushes simultaneously. The interaction diagram is calculated from considerations of equilibrium and compatibility.

$$e = m/n \qquad (1)$$

where: e is the eccentricity; m is the moment and n the axial force.

The expressions in equation 2 and 3 below for the moment \`M\` and axial force \`N\` for the parabolic- rectangular stress distribution in CP 110 were taken from Reynolds and Steadman (1980), this is because of better accuracy and economy over the rectangular

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stress block. This preference was also observed by Beeby (1978) and the adoption of this stress distribution in the current edition of BS8110.

$$N = k_1xb + A_{s1}^1 f_{yd1} - A_{s2}f_{yd2} \quad (2)$$
$$M = k_1xb\ (0.5h - k_2x) + A_{s1}^1 f_{yd1}(0.5h - d) + A_{s2}f_{yd2}(d - 0.5h) \quad (3)$$

Where
$$k_1 = 0.445f_{cu} - 0.00838(f_{cu})^{½} \quad (4)$$
$$k_2 = (1876 - 70.73(f_{cu})^{½} + f_{cu}) /(3752 - 70.73(f_{cu})^{½} \quad (5)$$

f_{yd1} and f_{yd2} are the appropriate values of f_y for compression reinforcement A_{s1} and tension reinforcement A_{s2} respectively.

An asymmetrical column section that was chosen for analysis is shown in figure 1. The column dimensions were b = 300mm, h = 600mm. the compressive strength of concrete was 25N/mm^2 the characteristic strength of steel was 425N/mm^2 and the area of bars (A_s) in both the compression and tension zone was 1963.5 N/mm^2.

Substituting the above nominal values in equation (2) and (3) and the value of f_{cu} in equation (4) and (5) and by assuming different strain distribution and hence the position of the neutral axis, the values for the moment m and axial load n were obtained in Table 1 which enables the construction of non dimensional drawing of interaction curve in figure 2. Pfang et al (1964) also gave a procedural approach of obtaining this curve. The plateau on the drawing is the minimum eccentricity h/20 allowed by the code.

Assumption for Simulation

The under listed assumptions were based on the report by Owoyale et al (2003)

1. Dimensions have normal distribution with mean = 0 and standard deviation = 12.5mm
2. Unintentional eccentricities have normal distribution with mean = 0 and standard deviation = 6.7mm
3. Cover to reinforcement has lognormal distribution with mean = 0.44mm and standard deviation = 15.6mm
4. Strength of concrete has normal distribution with mean = 19.7N/mm^2 and standard deviation = 4.5 N/mm^2.

Generation of Random Values of Parameter

The distributions used for generation of random variables are normal distribution, log normal distributions and cumulative distribution function. Normal distribution was used for generation of random variables for concrete strength and all geometric imperfections except concrete cover to reinforcement where log normal distribution was used. Curve of cumulative probability was adopted for the generation of random variables for the variability of high yield reinforcing steel because there is difficulty in expressing analytically the distribution obtained by Owoyale, et al (2003).

To generate normal distribution random variables equation 6 was used.

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$$x = \sigma_x(\sum r_i - 6) + \mu_x \quad (6)$$

Where x is normal distribution random variable to be simulated with known expected value μ_x and standard deviation σ_x and r_i is define over the interval $0 \leq r \leq 1$. For the generation of log normal distribution equation 7 was used.

$$x = \exp\{\mu_y + (\sum r_i - 6)\sigma_y\} \quad (7)$$

where log x = y and only positive values of x are considered. μ_y and σ_y are the mean and standard deviation of y respectively.

To generate random variables using the cumulative distribution function, random number r_i are generated from the uniform density (0, 1) and the corresponding value x_i is obtained by transformation. Further readings on this could be found in Jansson (1966) and Naylor et al (1968)

Calculation of Ultimate Strength

A special computer program was developed (the flow chart for this program is shown in figure 3) for IBM personal computers. The final version of the program was written in Quick Basic and when it was run on IBM PS/2 computer model 50z with mathematical co-processor, one case of computation took about a minute of the computer's time.

The moment and axial force were calculated by substituting the values of parameters obtained from the simulation of 1000 columns at different eccentricities to equations 1 to 4. The representative short term stress strain curve adopted for this report are 50% and 15% more than those provided in CP110 due to the exclusion of partial safety factors for materials. The three parts that made up the compression and tension zones for the steel stress-strain curse were separately considered. Simulation for 1000 columns was done and the frequencies of falling results in the various intervals were counted and printed out.

Figure 4 shows the histogram of strength, the ordinate represents frequencies in per mille (1/1000), the abscissa represents the strength (moment).

The eccentricity assumed to determine ultimate moment for the columns are 0.1, 0.2, 0.5, and infinity. At e/h = 0 a case of pure compression the expression in equation (2) was used. When e/h > 0 was considered, a case of eccentric compression, the expression in equation (2) and (3) were used. Table 2 shows the summary of the results at different eccentricities which are represented graphically in figure 5

Discussion of Results and Conclusion

In figure 4 the histogram of strengths was plotted and approximate normal curve constructed. These curves approaches the moment axis asymptotically, nevertheless the 5% quantile could be calculated to show the position of the characteristic strength. These distributions were represented at different assumed eccentricities in figure 5 in the same plane with the theoretical interaction curve obtained using the nominal values provided by the designer in accordance with the provision of the British Code of Practice CP110. The shaded area between the points of intersection of the theoretical curve of CP110

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and the characteristic strength on the histogram of simulated strength at the assumed eccentricity gives the cumulative effects of inaccuracies (and hence the safety factors) due to the variation in cross sectional dimensions, variations in concrete cover, non verticality of column, variation in the strength of in situ concrete and that of reinforcing bars. All these inaccuracies could be attributed to the quality of workmanship on construction sites. Since the normal population mean and standard deviation are known, the area can be calculated by using the standard normal distribution. The area under the curve (which are expressed in percentage) between the transformed ordinate shows the difference in the safety factors between the designed interaction curve of CP110 and the empirical interaction curve attainable on the construction sites. These areas vary for the different eccentricities considered as shown in Table 3. Therefore in the calculation of the ultimate strength based on the provision of this Code the percentage strength should be added at the various eccentricities before the safety factors envisaged by the Code could be achieved.

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Table 1: Computed moment, axial load and eccentricities using the column nominal values of Parameters.

n	m	e/h
0.505	0.025	0.050
0.446	0.047	0.100
0.388	0.061	0.156
0.350	0.070	0.200
0.291	0.082	0.282
0.258	0.088	0.282
0.217	0.090	0.416
0.181	0.090	0.500
0.152	0.088	0.580
0.133	0.085	0.640
0.113	0.081	0.717
0.083	0.073	0.883
0.069	0.070	1.001
0.055	0.065	1.200
0.029	0.058	1.970
0.009	0.050	5.500
0.000	0.049	2405

Table 2: Statistical properties for 1000 simulated columns at different eccentricities

Mean strength (N/mm^2)	Standard deviation (N/mm^2) $x10^{-2}$	Coefficient of Variation (%)	Characteristic strength (N/mm^2)	e/h
0.4736	9.414	19.88	0.3192	0
0.0394	7.979	20.24	0.0263	0.1
0.0618	1.228	19.88	0.0416	0.2
0.0832	1.600	18.00	0.0586	0.5
0.0520	5.319	10.23	0.0433	INFINITY

Table 3: Percentage of additional strength at different eccentricities

e/h	Additional factors of safety (%)
0	62
0.1	43
0.2	70
0.5	62
Infinity	13

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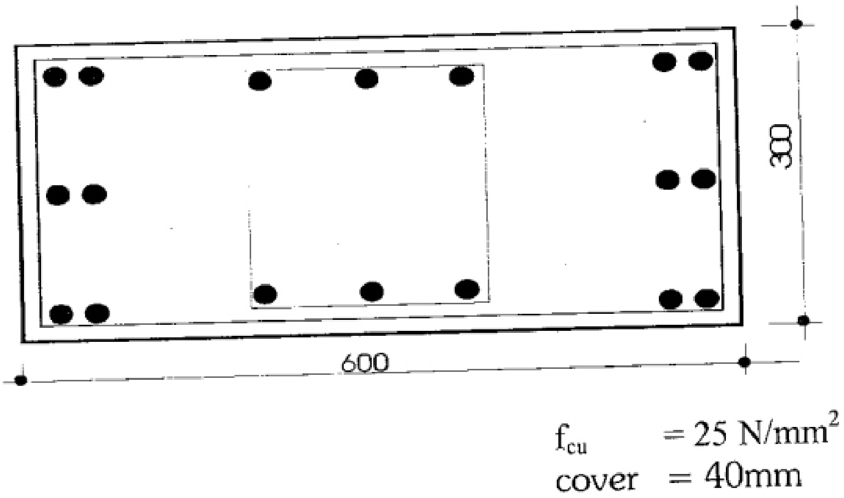

Fig. 1: Cross Section of Column Studied

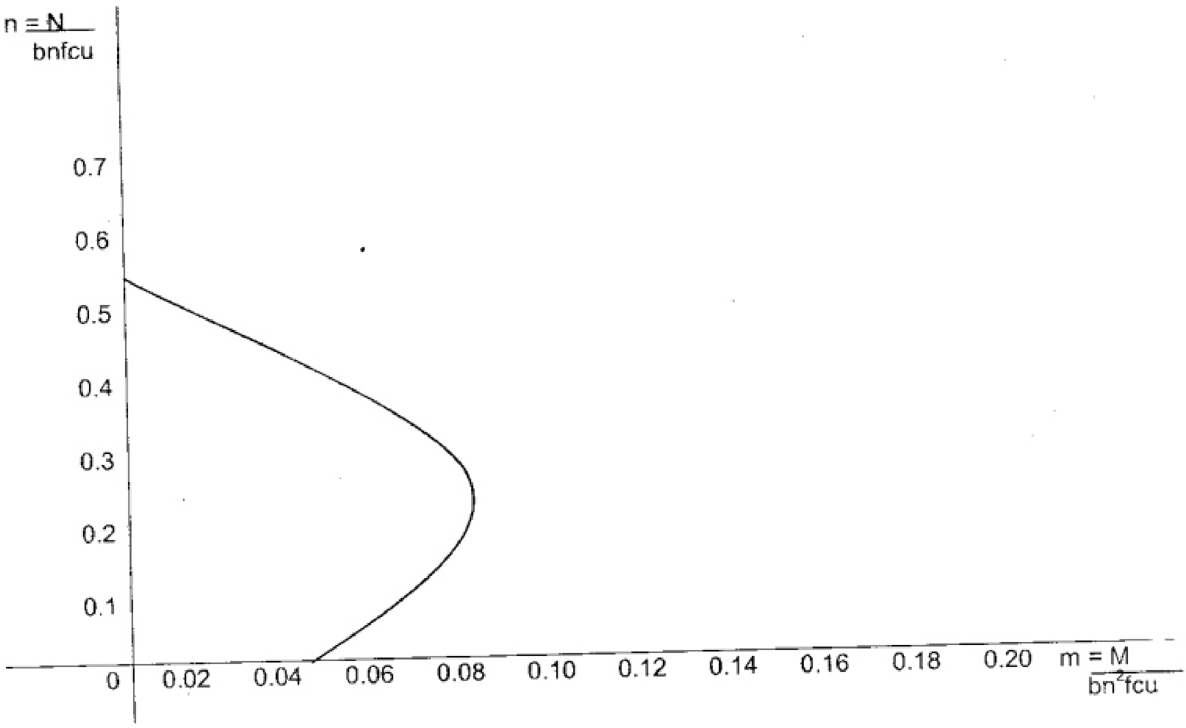

Fig. 2: Interaction Diagram for the nominal values of parameters for 300 x 600mm Columns

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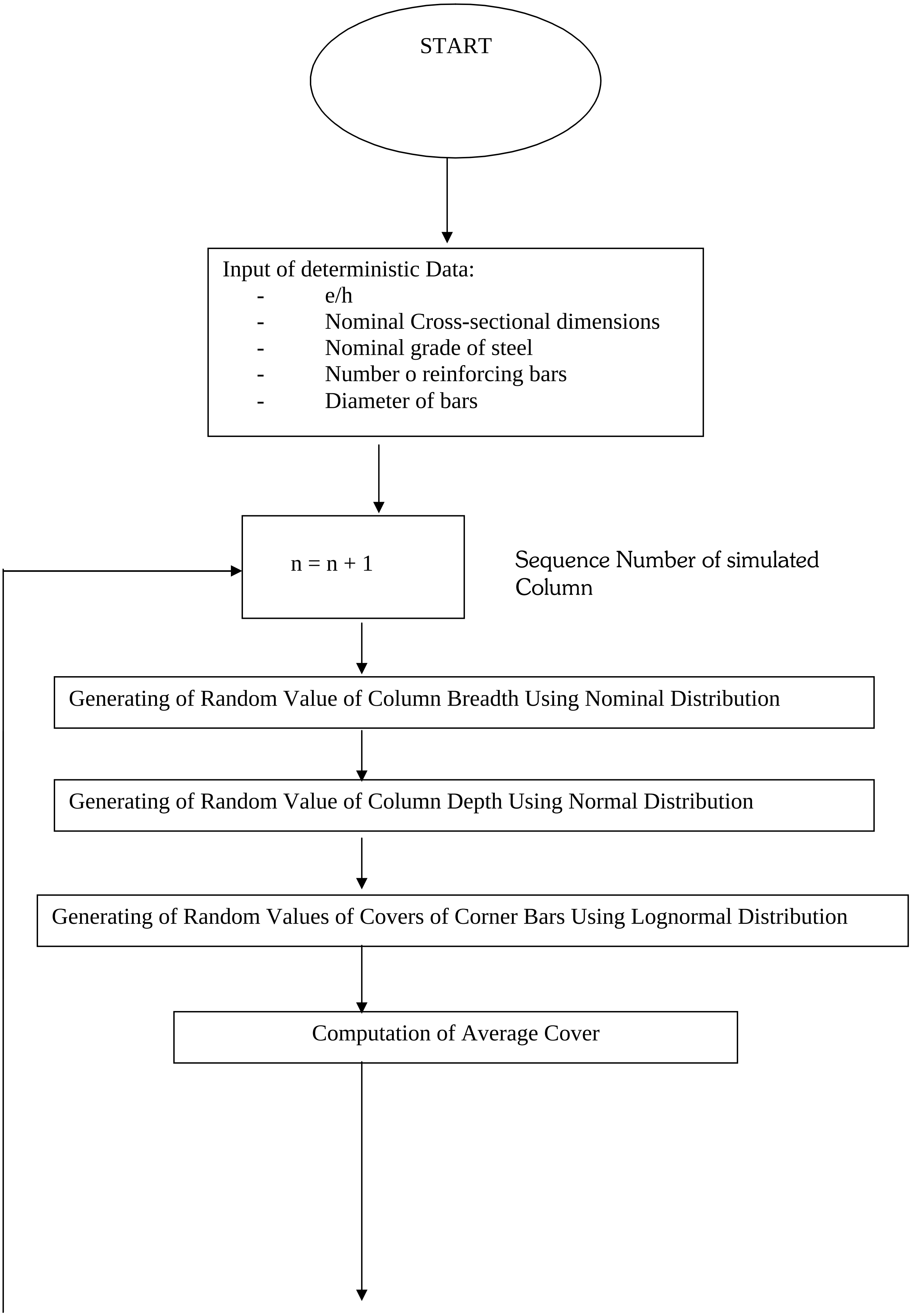

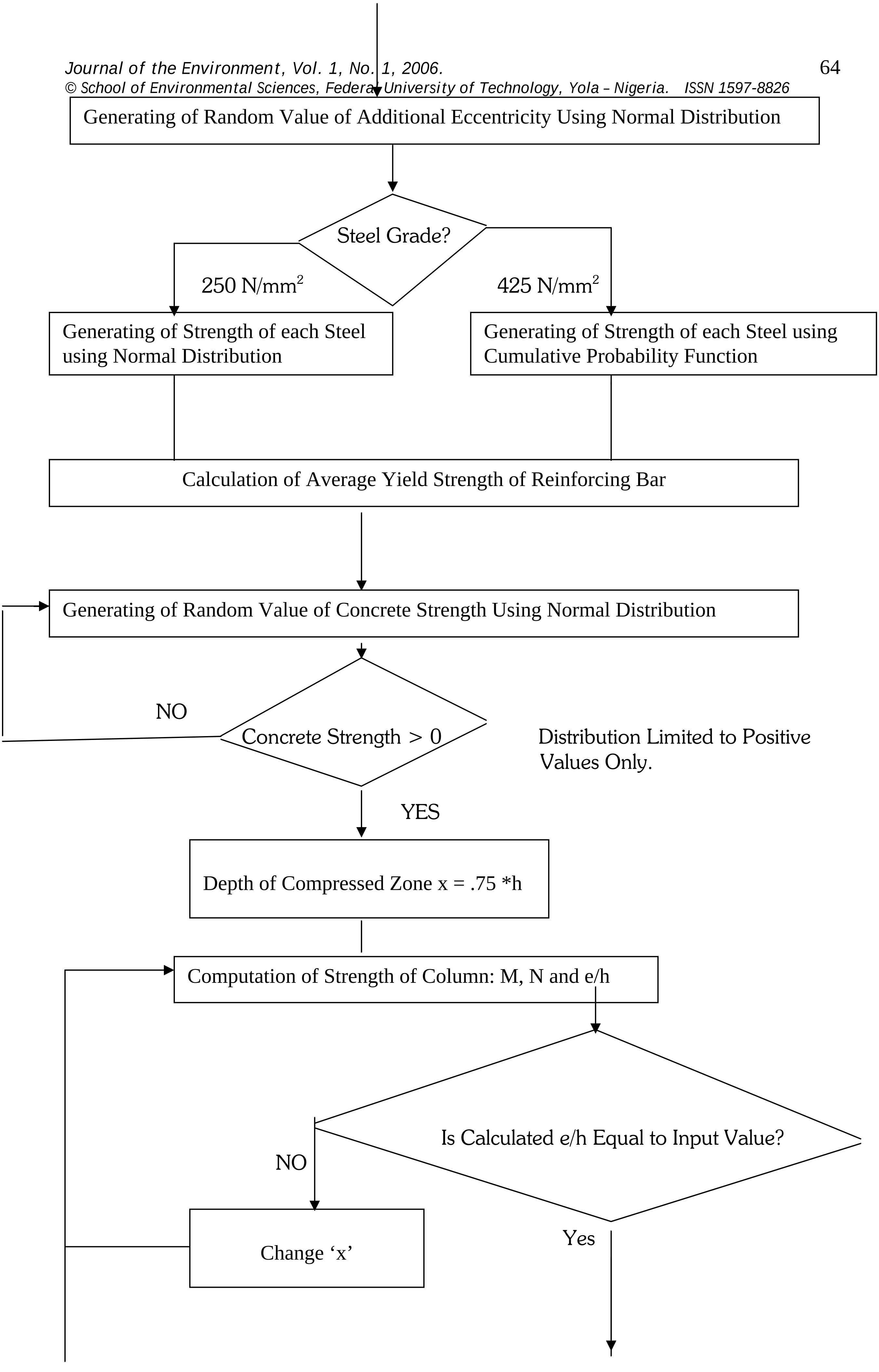
Generating of Random Value of Additional Eccentricity Using Normal Distribution
Steel Grade?
250 N/mm^2
425 N/mm^2
Generating of Strength of each Steel using Normal Distribution
Generating of Strength of each Steel using Cumulative Probability Function
Calculation of Average Yield Strength of Reinforcing Bar
Generating of Random Value of Concrete Strength Using Normal Distribution
NO
Concrete Strength > 0
Distribution Limited to Positive Values Only.
YES
Depth of Compressed Zone x = .75 *h
Computation of Strength of Column: M, N and e/h
Is Calculated e/h Equal to Input Value?
NO
Change 'x'
Yes

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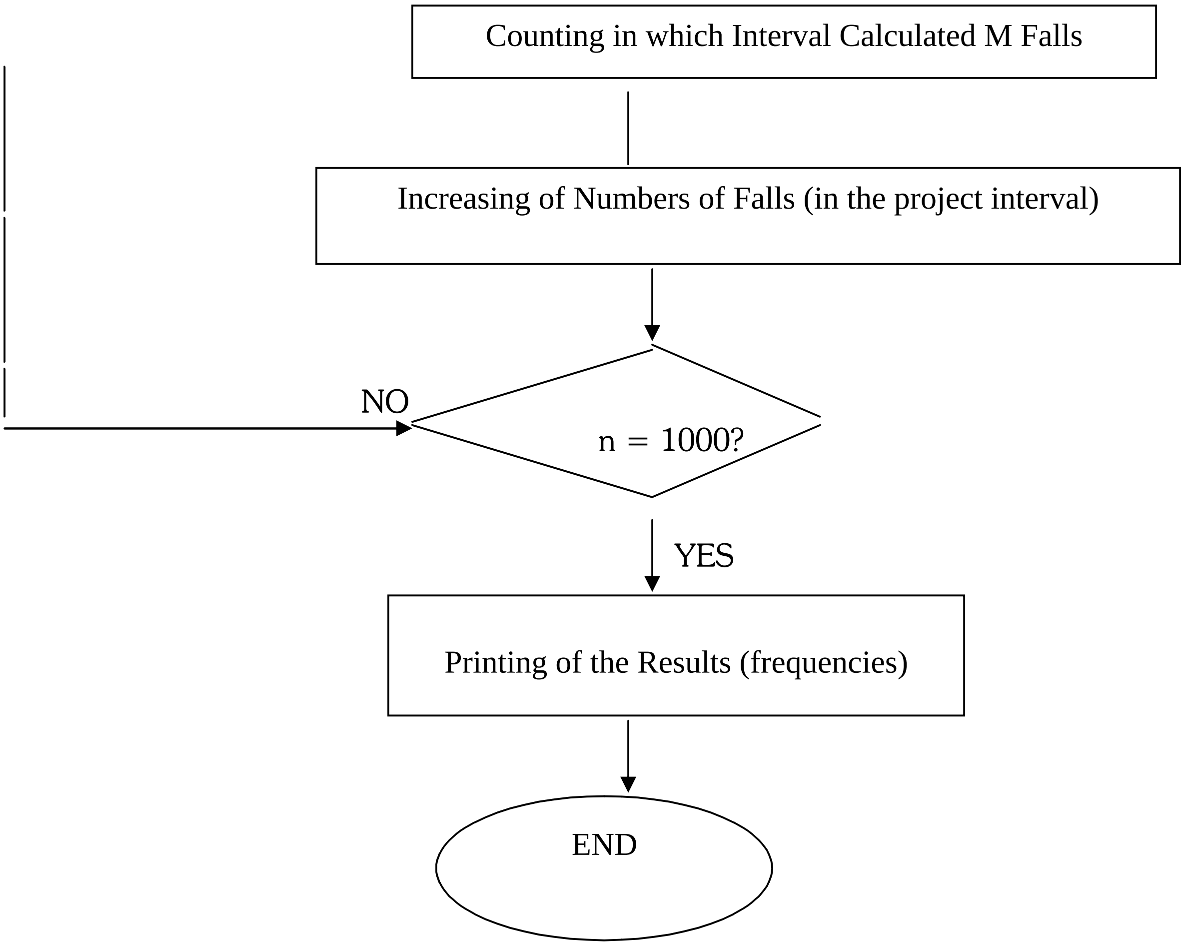

Figure 3: Flow Chart for Simulating Column Strength.

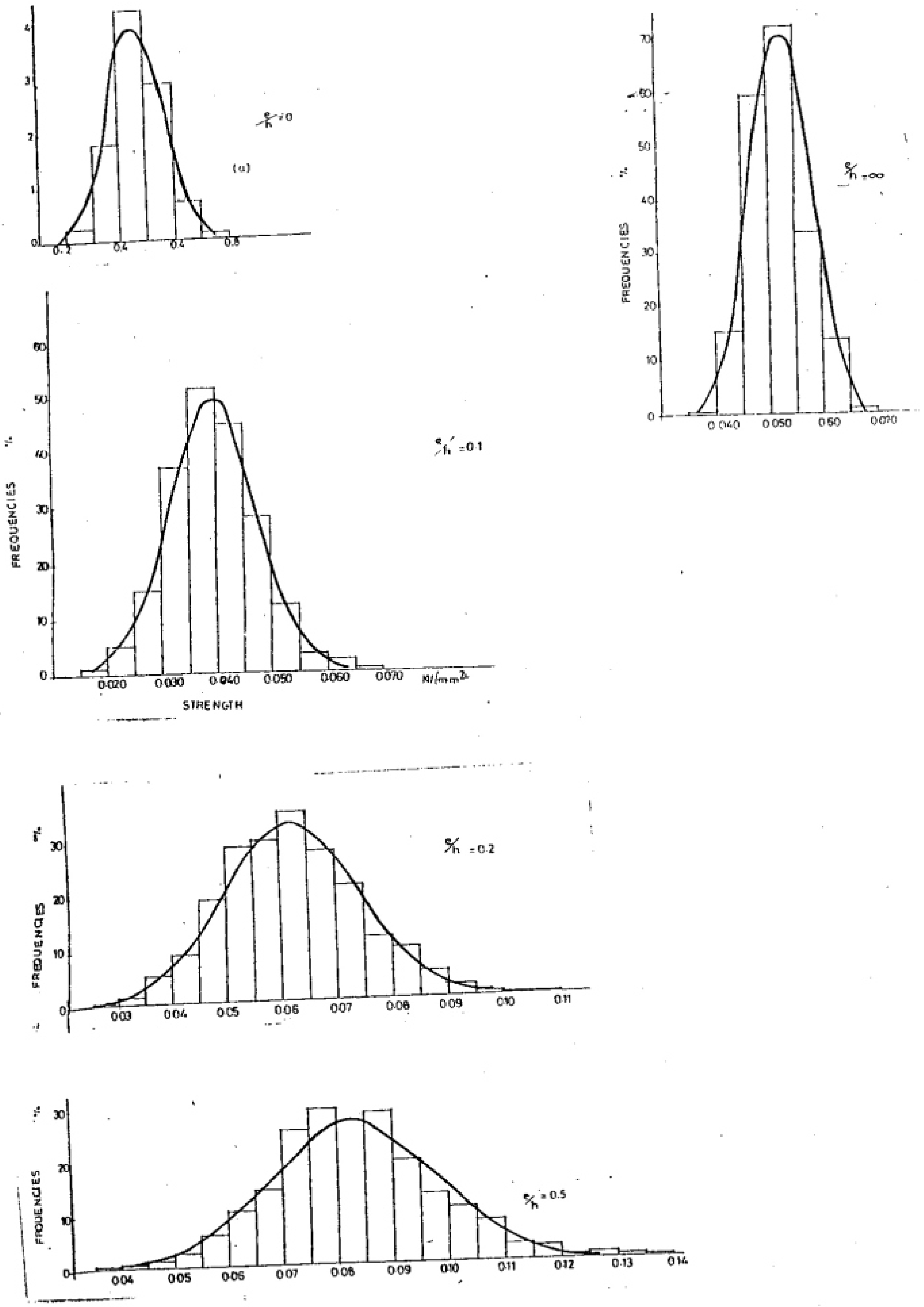

FIG 4 HISTOGRAM FOR STRENGTH AT ASSUMED ECCETRICITIES

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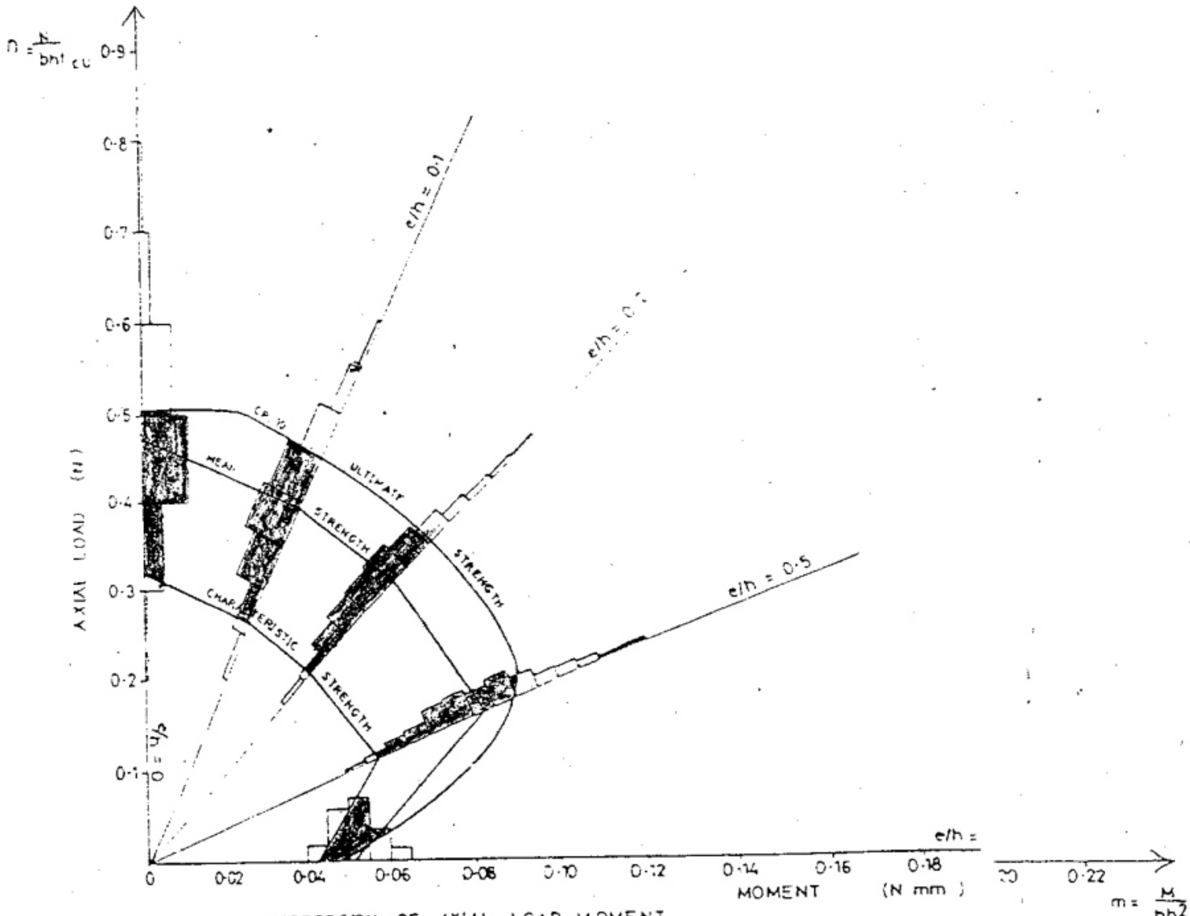

FIG. 5 DISPERSION OF AXIAL LOAD MOMENT

INTERACTION CURVE OF A RAMDOMLY GENERATED SAMPLE OF 1000 COLUMNS