STATISTICS FOR CONCRETE AND STEEL QUALITY IN SAUDI ARABIA

Abdulrahim M. Arafah[1]

ABSTRACT

This paper presents results of an experimental program designed to develop the probabilistic models of compressive strength of concrete and yield strength of reinforcing steel produced in Saudi Arabia as a first step towards the development of a national design code for reinforced concrete buildings. A total of 955 concrete samples and 434 samples of steel bars were randomly collected from the construction sites for strength testing. Results indicate that ready-mixed concrete types are well modeled by the normal distributions whereas site-mixed concrete is well represented by log-normal distribution with low mean-to-nominal ratio and high coefficient of variation. Variation in the yield strength of reinforcing steel is modeled by normal distribution with high mean-to-nominal ratio. The models are verified by the Chi-square and Kolmogorov-Smirnov goodness-of-fit tests at 5 percent significance level. The models developed in this paper is useful for prediction of the performance of structural elements and assessment of their reliability levels. They are also essential for calculating resistance factors for limit state design code.

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1 Assistant Professor, Civil Engineering Department, College of Engineering, KSU, P.O.BOX 800, Riyadh 11421, Saudi Arabia.

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INTRODUCTION

Structural reliability theory has several important applications in the field of civil engineering. Reliability approaches can be employed for calculating structural reliability under various sources of uncertainties and determination of safety factors for limit state design codes. Reliability-based methods have been employed successfully in the development of design codes of reinforced concrete and steel structures [1,2,3,4].

Load and resistance factors in limit state design for reinforced concrete structures account for the variabilities in load and resistance parameters. The philosophy behind the probabilistic approach is the acknowledgment that absolute reliability is an unattainable goal in the presence of various kinds of uncertainties. The reliability based design provides a formal framework for developing design criteria which ensure that the probability of unfavorable performance is acceptably small.

The main problem in applying reliability concepts in structural engineering fields is the lack of knowledge about the statistical characteristics of load and strength parameters and their probabilistic models.

This paper presents probabilistic models of compressive strength of concrete and yield strength of reinforcing steel produced in Saudi Arabia as a first step towards the development of a national design code for reinforced concrete buildings. The models are verified by the Chi-square and Kolmogorov-Smirnov goodness-of-fit tests at 5 percent significance level. The developed models developed are essential for prediction of the variations in the performance of structural elements and assessment of their reliability levels. They are also essential for calculating the resistance factors for various limit states.

BACKGROUND

In the reliability-based limit state design, probabilistic methods are used to guide the selection of load and resistance factors which account for the variabilities in the individual load and resistance parameters. The advantages of probabilistic limit states design include the possible achievement of a more consistent reliability for different design situations as the variabilities of the related strength and load parameters are considered explicitly, and the possible selection of suitable reliability level for a structure which reflects the consequences of failure [5,6].

This section presents three basic relationships: (1) the relationship between the statistical characteristics of strength parameters with that for the strength of structural members, (2) the relationship between statistical characteristics of structural members with its reliability index, and (3) the relationship between the target structural reliability index and the resistance factors for design.

The variability in the strength of reinforced concrete members is attributed to (a) the variations in material properties, the compressive and tensile strengths of the concrete and the yield strength of the reinforcing steel, and the member sectional dimensions, and (b) errors in the statistica and analytical prediction models. The statistical characteristics of the limit state of structural members can be estimated for a wide range of design from basic strength parameters employing simulation techniques such as Monte-Carlo simulation [6]. The sensitivity of related factors can be investigated such as: type of concrete, percentage of reinforcement ratio, sectional dimensions of the member and the level of concrete confinement.

Structural reliability is usually measured by the reliability index, β , which is defined as,

$$\beta = - \Phi^{-1} (P_f) \tag{1}$$

where Φ^{-1} is the inverse of standard normal distribution function and P_f is the probability of failure. The exact value of β can be calculated for linear limit state function with two mutually independent random variables which have normal distributions, as follows,

β = خطأ! **(2)**

in which μ_R , σ_R are the mean and standard deviation for R, and μ_Q , σ_Q are the mean and standard deviation for Q. The relation between P_f and β is shown in Fig. 1 . Several expressions for β are available depending on the nature of limit state and distributions of related parameters [5,6,7,8].

The target reliability index in design codes are usually selected to reflect the type of failure (ductile or brittle) and the consequences of failure (serious or not serious). For example, the Nordic Committee on Building Regulations [9] has recommended a wide range of target reliability indices starting from 3.09 ($P_f=10^{-3}$), for ductile and not serious types of failures, to 5.30 ($P_f = 10^{-7}$) for brittle and serious types of failures [8]. The recently developed ASCE 7-93 [3] criteria are based on target reliability index of 3.0 for ductile failures such as would occur in under reinforce beams and in spiral columns, and of 3.5 for brittle failures expected in shear and tied columns [6].

There are several formulae for incorporating the safety factors in the limit state design. The European formula is based on the partial safety factors for loads, γ_{ai} , and resiatance parameters, γ_{mi} [10]. The ACI formula is based on the resistance factors for the member limit states, ϕ [11]. The European formula provides more consistent reliability levels for wide range of designs specially for beam-column members.

The design safety factors account for several types of uncertainties such as the variations of load parameters and the sources and properties of materials. To explain the relationship among the statistical characteristics of load and resistance parameters, target reliability index and the safety factors, consider a safety checking format such as,

$$\phi \; R_n \; \varepsilon \; \gamma \; Q_n \qquad (3)$$

where ϕ and γ are the resistance and load factors which, for log normal distributions of load and resistance parameters, can be approximately expressed as,

$$\gamma = \lambda_Q \text{ Exp } (0.75 \; \beta_T \; V_Q) \qquad (4)$$

$$\phi = \lambda_R \text{ Exp } (- 0.75 \; \beta_T \; V_R) \qquad (5)$$

where λ_Q and λ_R are the mean-to-nominal ratios of load and resistance parameters, respectively, whereas V_Q and V_R are the coefficient of variations of load and resistance parameters and β_T is the target reliability index.

EXPERIMENTAL PROGRAM

The main objective of the experimental program is to develop probabilistic models of compressive strength of concrete and yield strength of reinforcing steel produced in Saudi Arabia.

Concrete Sampling and Testing

Concrete samples were randomly collected from twenty cities overall Saudi Arabia. A total of 955 concrete samples were collected from the construction sites during the casting process. At each site, two concrete samples were taken from two different trucks. Each sample comprised two standard cubes (150x150x150 mm) which were transferred after 24 hours to the laboratory where they water-cured for 28 days before testing for compressive strength. The concrete samples were classified according to the type of mixing as ready-mixed, and on site mixing in a rotary drum or manually. These types are designated as C, ME, and MA, respectively.

Reinforcing Steel

Variability of the yield strength of Grade 60 steel with nominal yield strength of 413 MPa, produced the Saudi Iron and Steel Company is investigated. A total of 335 samples of steel bars were randomly collected from market and job sites and designated as H1. Recently, the company employed quenching process for production of bars. An additional 98 samples of steel bars were designated as H2 and tested for strength to investigate the effect of new production procedure on the statistical characteristics of yield strength.

PROBABILISTIC ANALYSIS

The probabilistic models for the strengths of the five types of concrete and two types of reinforcing steel are developed through the following four main steps: (1) check the reliability and homogeneity of data, (2) plot the data on normal probability forms, (3) perform the linear regression analysis, and (4) check the validation of the developed models.

Reliability and Homogeneity of Data

In order for the test results to provide representative probabilistic models for the distribution functions they must be reliable and form a homogeneous set. Reliability of strength results is considered by random sampling process. Concrete samples were collected from construction site during the casting process. Samples are taken from different trucks and from only two trucks per construction site. Two cubes were tested in each sample to check the reliability of testing process. Theoretically, the strength results of the two cubes shall be identical. When the difference between the two cubes exceeds 10 percent of the strength, human errors in cubes preparation and testing is more likely to be occurred and the result is rejected. If the difference is within 10 percent, the average strength of the cubes is computed. Several precautions were taken to ensure the homogeneity of test results and minimize the possibility of errors in sampling, testing, recording, and processing of strength results.

Normal Probability Papers

Normal probability paper (NPP) is a special form on which the normal cumulative distribution function is represented by a straight line. This type of forms is usually employed to model normal and log-normal distribution functions. The relation between the two models can be states as "If a parameter has log-normal distribution its natural logarithm has normal distribution" [12].

Regression Analysis

The third step in the modeling process is to perform the regression analysis between strength parameter (x) and the inverse of the standard normal distribution function (z). An equation for the best fitted line can be developed using the well-known Least Square Method as follows,

$$\mathbf{x = a\ z + b} \tag{6}$$

where a and b are constants which are calculated using linear regression analysis on NPP.

Model Validation

The final step in the modeling process is to check the validity of the obtained models. The models were checked by the Chi-square and Kolmogorov-Smirnov (K-S) goodness-of-fit tests at 5 percent significance level [12]. The statistics D_1 and D_2 for the Chi-Square and Kolmogrov-Smirnov goodness-of-fit tests, respectively, are calculated as follows,

$$D_1 = \text{خطأ!} \qquad (7)$$

where Oi and Ei are the observed and expected number of occurrences in i^{th} interval, respectively, and k is the number of intervals

and

$$D_2 = \max^{n}{}_{,i=1} \left[\left| \text{خطأ!} - F_X(X^{(i)}) \right| \right] \qquad (8)$$

i.e, D_2 is the largest of the absolute values of the n differences between the hypothesized distribution function, $F_X(X^{(i)})$, and the observed cumulative histogram, i/n.

The statistics D_1 and D_2 are then compared with corresponding critical values, D_{1c} and D_{2c} , at 5 percent significance level [12].

RESULTS, ANALYSIS AND DISCUSSION

Strength of Concrete

The modeling process was performed for the five types of concrete as shown in Figs. 2 through 7. The probabilistic models of variations in concrete strength are listed in Table 1. The models were checked by the Chi-square and K-S goodness-of-fit tests at 5 percent significance level. Table 2 shows Chi-square test for concrete type C35. Results for concrete grades are listed in Table 3. Graphical presentation of K-S test is shown in Figs. 2 to 7. The following observations can be drawn,

1. The plots indicate that concrete types C35, C30, C20, C25, C20 and MA have normal distribution whereas ME has log-normal distribution. The results of Chi-

square and K-S tests indicate that the D_1 and D_2 are less than critical values D_{1c} and D_{2c} , respectively, which indicate that models of strength variations listed in Table 1 are acceptable at 95 percent confidence level.

2. The mean-to-nominal ratios, λ_c, are 1.05, 1.02, 0.95, 0.85, and 0.85 for concrete grades C35, C30, C20, ME and MA. Results indicate that λ_c decreases with decreasing the concrete nominal strength.

3. The COVs for grades C35, C30, C20, ME and MA are about 16, 20, 21, 43, and 31 percent, respectively. Results indicate that V_c increases with decreasing the nominal strength.

4. Concrete produced by ME or MA is of poor quality which adversely affects the capacity and durability of structures.

5. The 5th percentiles of the distribution functions are much less than the nominal strengths. This can be attributed to poor quality control on the production process and/or change in the concrete proportions resulting from extra water traditionally added at the site prior to discharge from the truck.

Reinforcing Steel

The modeling process was performed for the steel types H1 and H2 as shown in Figs. 8 and 9. The probabilistic models of variations in steel yield strength are listed in Table 4. The models were checked by the Chi-square and K-S goodness-of-fit tests at 5 percent significance level. Results are listed in Table

5. Graphical presentation of K-S test is shown in Figs. 8 and 9. The following observations can be drawn,

1. The plots indicate that steel types H1 and H2 are normally distributed. The results of Chi-square and K-S tests indicate that the D_1 and D_2 are less than the critical values D_{1C} and D_{2C} , respectively, which indicate that probabilistic models shown in Table 4 are acceptable at 95 percent confidence level.

2. The mean-to-nominal ratio of H2 is 1.34 which is higher than that for H1 and those reported in the literature. This adversely affects the ductility of structural members specially those with poor quality of concrete and subjected to high rate of loading.

3. The COVs of H1 and H2 are 6.0 and 4.3, respectively, which are close to those reported in literature.

4. The 5th percentiles of H1 and H2 are 435 and 515 MPa which are higher than the nominal strength (413 MPa) which shows the effect of the new production process on the probabilistic model of strength distribution.

For comparison with data from other countries, The coefficient of variation ,V_C , of the in-situ compressive strength for concrete grades 35 and 20 MPa are estimated to be 15% and 18%, respectively [13]. Concrete strength is considered to follow a normal distribution [13]. Ellingwood [14] estimated the V_C to be 20.7% under average control. Freudenthal et al. [15] reported that the distribution of f_C conformed to a logarithmic normal distribution under poor quality control. The results obtained in this study are consistant with data available in literature. The COV of strength for the site-mixed concrete was observed to be very high indicating the poor quality of this type of cocnrete and the below standard production method of this type of concrete.

Mirza and MacGregor [16] indicated that the mean and coefficient of variation of yield strength for grade 60 steel were 465 MPa and 9.8%. The Beta function was used to model the yield strength. Ito and Sumikama [17] studied typical statistics of yiled strength of grade 60 steel from several countries as listed in Table 6. Results indicated that λ for yield strength is between 1.08 and 1.19 whereas the COV is between 4.8 and 10.6 percents. Results from steel type H1 are close to those available in the literature whereas λ for H2 is higher than those available in literature indicating inconsistencies in the steel production employing the quenching process in Saudi Arabia.

CONCLUSIONS

The paper presented the outcomes of an experimental program prepared to estimate the statistical characteristics and probabilistic models for concrete and reinforcing steel strengths in Saudi Arabia for reliability and risk analysis. Results indicated that concrete and steel Strength parameters have normal distribution except concrete mixed at site in a rotary drum which is closer to the log-normal distribution. Results can be used for prediction of the statistical characteristics of structural elements and calculations of the partial safety factors as a first step towards the development of a national limit state design code for reinforced concrete buildings. The results presented in this paper is describing the current practice in Saudi Arabia however the approach can be employed in other countries.

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Table 1 Concrete strength distribution models and their statistics.

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				16.14	2	
				0		
				21.60	6	
	l			42.96	97	
				30.64	6	

Table 2 Chi-Square Test for Concrete Type C35

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able 3 Results of Chi-Square and K-S tests for concrete

	Test		est	
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Table 4 Reinforcing steel strength distribution models and their statistics.

	f	ution nction				

Table 5 Results of Chi-Square and K-S tests for reinforceing steel

	e Test		est	

Table 6 Summary of selected studies on yield strength of reinforcing bars [15]

Manufacturing country	λ	V_S %
Canada	1.19	7.7
England	1.11	4.8
England	1.10	6.9
U.S.A.	1.18	10.5
U.S.A.	1.13	7.3
U.S.A.	1.11	7.3
U.S.A.	1.08	7.6
U.S.A.	1.17	7.1
U.S.A.	1.14	8.2