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INVESTIGATING THE FACTOR OF SAFETY FOR THE DESIGN OF COLUMNS USING PROPERTIES OF STEEL REBAR'S MANUFACTURED IN PAKISTAN

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Abstract: This paper includes the behaviour of RC column, using the steel strength data employed by Rafi et al. (2014). Eccentric short columns are studied for this purpose, both tension and compression controlled sections, are analysed considering the current design practice of Pakistan. Three cross sections were analysed using different steel percentages against load-moment interaction and the strength analyses. Concrete strength is also varied in this analysis. The load moment interaction diagrams were observed in major and minor axes and strength analysis is made for compression controlled and tension controlled sections. In this analysis it is observed that a section designed as a tension controlled is giving brittle failure at certain limit which should be avoided. Considering this scenario, several random cross sections are analysed, strength reduction factors for eccentric and pure axial columns are computed. Conclusions are made on behalf of this analysis for different types of column design.

Keywords: Strength reduction factor, Strength analysis, Load moment interaction

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1. INTRODUCTION

Steel is considered as the best reinforcing material for concrete structures. The steel properties, such as, strength and compatibility with concrete has made it very popular in reinforced concrete and pre-stressed concrete applications. The high energy absorption capacity of steel makes it an ideal material to carry out construction in the seismically active regions. Such properties enable a structure to deform beyond the elastic limit without collapse. Hence it is necessary that physical and mechanical properties of steel must meet the basic design assumptions.

In Pakistan scrap material is one of the sources for steel manufacturing [1]. These steel bars are also used in structural applications in RC structure. On other hand these bars have great variation in properties like chemical composition, strength and ductility etc. [2] conducted a study on steel with rust and conclude that manufacturing process and material has a greater impact on the properties of steel bars. The capacity of such construction to resist seismic loads is highly uncertain. Since several zones of Pakistan lies in seismically active regions this uncontrolled and deficient construction is a major point of concern from the view point of life safety.

[3] conducted a study on flexural member design using the properties of steel rebar's manufactured in Pakistan. The author provided the flexure design in Pakistan. Since compression members play a crucial role in providing seismic resistance to RC buildings, it is necessary that the effects of using these bars on the behavior of columns are also studied.

To overcome the uncertain properties of these steel bars to be incorporated in structural applications of compression members (columns), it is of a great importance if a study has been conducted for the safe design of columns using properties of steel which is usually produced in Pakistan by scrap material and is used heavily in construction industry. The objective of this research is to study the behavior of columns designed using the steel bars manufactured in Pakistan. Columns with different designs have been analyzed to study the differences in the behavior assumed in their design. Scope of this research includes the study and determination of strength reduction factor for the design of columns. ACI 318-11 is utilized for the analytical purpose and all the analyses and calculations are done using Ms-Office tools. This report is purely related to the structural design practice of Pakistan. The recommendations given in this research can be used for the building code of Pakistan. However, economical aspects of the recommendations are beyond the scope of this report

2. METHODOLOGY

Two types of analyses have been carried out for short columns. These include load-moment interaction diagram and strength analysis. Three cross sections have been considered which include

14x22, 10x24 and 8x18 in. These cross sections were arbitrarily chosen based on the experience of the commonly used column sizes in Pakistan. The steel strength for the steel manufactured in Pakistan as reported by [3] has been used in this report. The statistics of steel strength data has been shown in Figure 1. Steel ratio varied from 1 to 3 percent. Concrete compressive strengths (f_c') of 3, 4 and 5 ksi have been selected. The parameters of the analysis include the steel ratio, steel yield strength and concrete compressive strength.

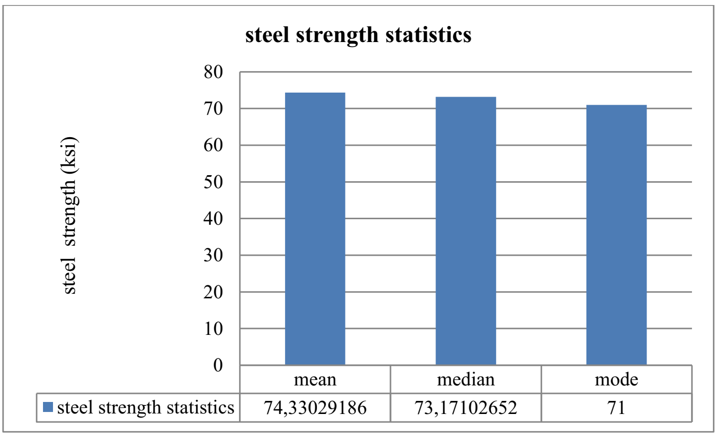


Figure 1: Statistics of steel strength data

2.1 LOAD MOMENT INTERACTION DIAGRAM

To plot the load moment interaction diagram, the eccentricity ranges from zero (state of pure axial compression) to infinity (pure flexure state). The steps of plotting load moment interaction diagram have been suggested by [4]. The balanced condition is a boundary between compression controlled and tension controlled failure. The balanced load, moment and eccentricities have been determined using the procedure of ACI code. Eccentricities smaller than balanced eccentricity describes the compression controlled sections to be design for compression failure while the eccentricities greater than balanced eccentricity are categorized as tension controlled sections and designed for tensile failure mode.

Load moment interaction diagram are plotted as described by Nadeem Hassoun to observe the behavior of same cross section with respect to change of steel strength. From steel strength statistical data, it can be observed that mean strength is the highest among all that's why average steel strength rounded to 75 ksi is taken for analysis. Interaction diagrams of previously mentioned cross sections are plotted for the steel yield strength of 60 and 75 ksi. These interaction diagrams are plotted for both the orientation of steel placement i.e. steel in weaker direction and steel in stronger dimension.

2.2 STRENGTH ANALYSIS

As mentioned earlier the data of steel bars employed by [3] is employed and steel ratio is considered from 1% to 3%. The strength analysis has been done on both type of section design i.e. Tension controlled sections and Compression controlled sections.

2.2.1 TENSION CONTROLLED SECTIONS

The steps of the analysis have been taken from [5]. For a cross section, balanced eccentricity is calculated for the chosen fc', fy and steel percentage. The same section is analyzed for eccentricities greater than eb. The design eccentricity is taken as a fraction of balanced one. The above steps are repeated for the bar strength data until the failure mode is changed from tension to compression failure. For tension controlled section, it is assumed initially that compression steel is yielded which is verified after the determination of the neutral axis position. If it is not to case, then the stress in compression steel should be used in calculation obtained from strain diagram.

If the stress from strain diagram is found equal or greater than yield strength, the initial assumption of compression steel yielding is verified or if it is not the case then value of stress is compression steel must be revised and a new value of neutral axis depth c is to be determined resulting a new stress value. This cycle continues until the assumed stress and calculated stress matches.

The same procedure is being repeated for steel strength data employed in the analysis. Different samples were modelled using different steel percentages and cross section sizes to obtain a data set of depth of neutral axes.

2.2.2 COMPRESSION CONTROLLED SECTIONS

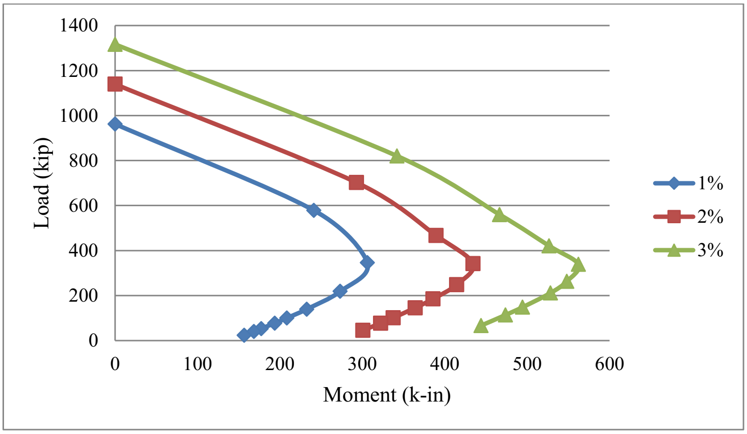
In addition to the strength analysis of tension controlled sections, the same approach is used to analyze the same sections for compression controlled behavior. For this purpose, design eccentricity is taken as a fraction of balanced eccentricity and percentage of steel varied from 1% to 3%. The steps for compression failure analysis are as follow [5].

For a cross section, balanced eccentricity is determined for the selected fc', fy and steel percentage. The same section is analyzed for eccentricities less than eb. The bar strength data is used for analysis of cross- section at the selected eccentricity. Apply the design procedure of compression controlled sections as described in ACI code and determine the depth of neutral axis such that c > cb. Now change the steel percentage in cross-section and repeat the procedure

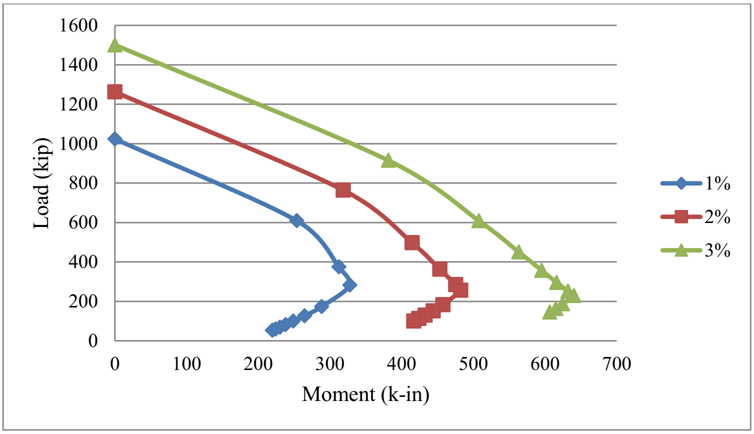
3. ANALYTICAL RESULTS

3.1 LOAD MOMENT INTERACTION DIAGRAM

Figure 2 shows the load-moment interaction diagram for 8x18 in. column using 60 ksi and 75 ksi steel bars. The concrete strength of 3 ksi has been used and three reinforcing bar percentages (1, 2 and 3%) have been considered. The vertical and horizontal lines drawn on the curves indicate balanced load (P_b) and balanced moment (M_b) respectively. As mentioned earlier the region of load-moment interaction diagram above balanced condition represent compression control region. It is seen in Figure 2 that the compression control region increases with both the amount of steel reinforcing percentage and f_y. This is a typical trend observed for 14x22 in. cross section and this trend is similar for all the other cross-sections and concrete strengths considered in this analysis.



(a)



(b)

Figure 2: Load-moment interaction diagram of 14x22 in. cross section with f_c' 3 ksi: (a) f_y 60 ksi; (b) f_y 75 ksi

3.2 TENSION FAILURE ANALYSIS

Figures 3 to 6 present variations in neutral axis (c) versus f_y for the employed column cross section at four different eccentricities using concrete strength of 3 ksi. The eccentricities were taken as a fraction of balanced eccentricity. The curve for balanced condition is also plotted in Figures 3 to 6. The result using f_c' of 4 ksi and 5 ksi are found similar.

The region above the intersection of the curve with the balanced condition in Figures 3 to 6 indicates compression controlled condition. It can be seen in Figures 3 to 6 that the behavior of the section changes from tension controlled to compression controlled as f_y is increased. This is indicated by the segment of the curves which cross over e_b curve.

At small eccentricity, variation in c with f_y are small for variation in ρ. At higher eccentricity above e_b the behavior of cross-section changes from tension controlled to compression controlled with the increase in ρ.

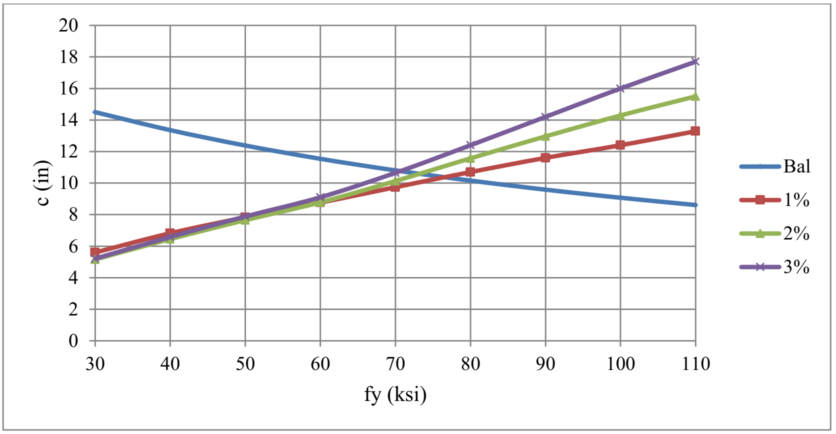


Figure 3: Results of tension failure analysis for 14x22 in column at 1.25 e_b for f_c' = 3 ksi

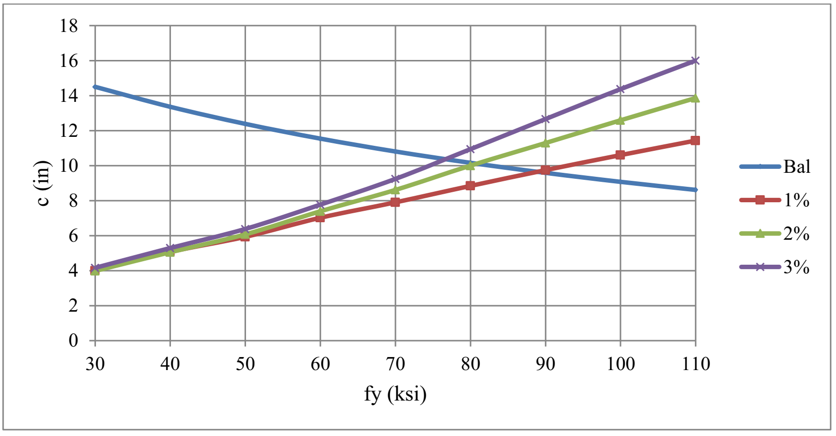


Figure 4: Results of tension failure analysis for 14x22 in column at 1.5 e_b for f_c' = 3 ksi

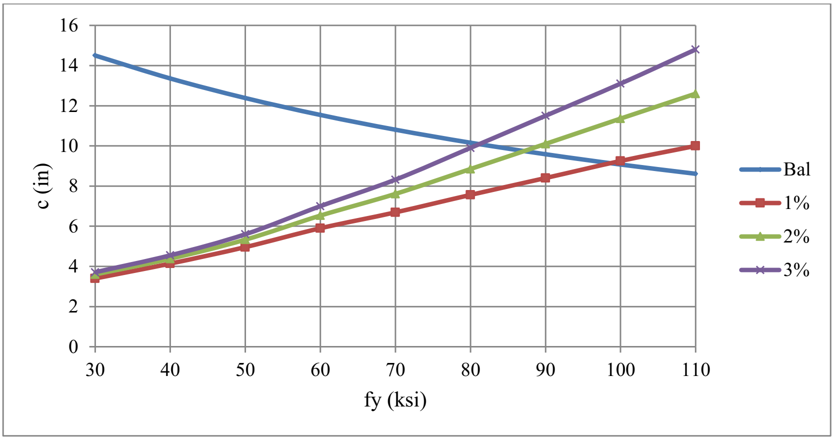


Figure 5: Results of tension failure analysis for 14x22 in column at 1.75 e_b for f_c' = 3 ksi

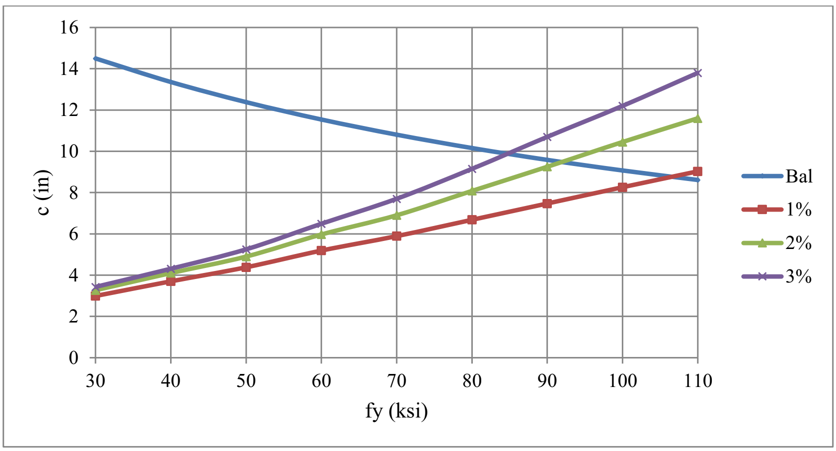


Figure 6: Results of tension failure analysis for 14x22 in column at 2 e_b for f_c' = 3 ksi

3.3 COMPRESSION FAILURE ANALYSIS

This analysis shows that compression controlled sections are not much significantly affected by the variation of steel bar strength. And almost all the region of data fall above the balanced condition which is represented in Figures 7 to 9. Higher percentage of steel in a compression controlled section yields in a lesser value of *c* as compared with the lesser value of steel ratio

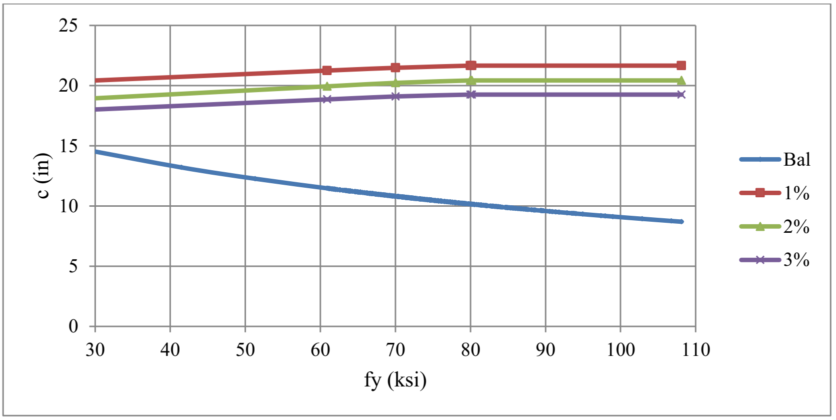


Figure 7: Results of compression failure analysis for 14x22 in column at $0.25e_b$ for $f_c' = 3$ ksi

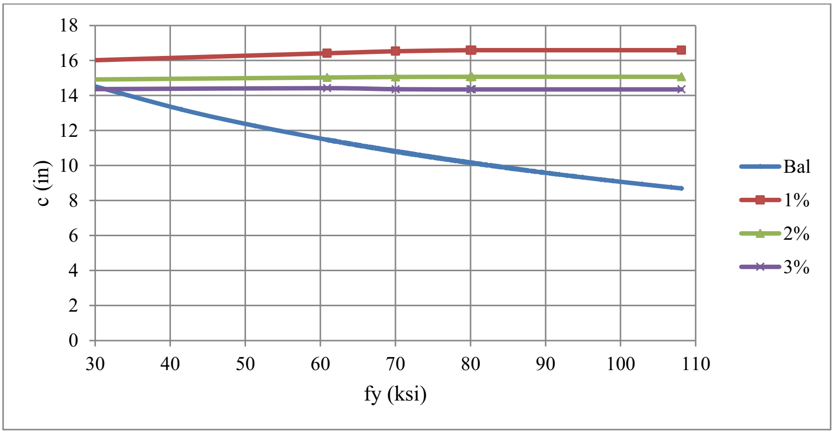


Figure 8: Results of compression failure analysis for 14x22 in column at $0.5e_b$ for $f_c' = 3$ ksi

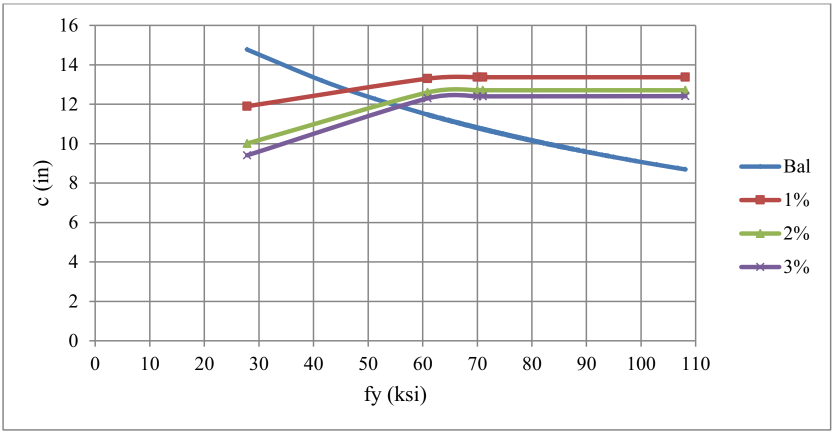


Figure 9: Results of compression failure analysis for 14x22 in column at $0.75e_b$ for f_c' = 3 ksi

4. STRENGTH REDUCTION FACTOR

4.1 STATISTICAL PROPERTIES OF VARIABLES

The construction industry in Pakistan is still lacking with strict regulations and building control clauses. The general construction including domestic and residential units are not even designed scientically [13]. The properties assumed in the calculations of strength reduction factors are listed in table 1.

4.1.1 CONCRETE

The specified concrete strength is assumed to be 3000 to 5000 psi. The mean strength of concrete in the structure itself is taken as 0.9 times that of cylinder desired strength. The coefficient of variation is taken as 0.18, calculated by combining V=0.15 for the control cylinder test [6] and the coefficient of variation of the ratio between the strength in structure and that in control cylinders, assumed to be 0.10 [7]. The tensile strength of concrete has been neglected.

4.1.2 STEEL REBARS

Grade 60 reinforcement steel bars that were employed by [3] is been considered in this study with a specified yield strength of 60 ksi and a mean strength of 74.33 ksi with the standard deviation of 9 ksi. The coefficient of variation for steel is calculated as 0.12. It is to be noted that steel bars made of scrap metal are considered as low ductile reinforcement [12] so it must be sensitive over the calculation of factor of safety. Its compatibility with concrete has made steel the best material for reinforcement even in this modern age [14] but a proper inspection must be made prior to design.

4.1.3 DIMENSIONS

In the absence of data for the variation in dimensions, the average width and depth of cross section will be assumed to be 0.05 inch greater than the design values with a standard deviation of 0.3 inch [8]. The column cross sections are assumed to have a longitudinal steel percentage of 1.5% to 3% (which is according to the design history of Pakistan). Finally the rolling tolerances are such that the mean area of provided steel will be assumed to be 0.98 times the nominal area chosen by the designer and the coefficient of variation is 0.03 [9], it is done due to the fact that designer cannot accurately provided the calculated steel area rather steel provided has a slightly greater area which is assumed to be 1.02 times the required with the coefficient of variation of 0.05. Thus the mean area of steel is assumed to be 1.02x0.98= 1 and the coefficient of variation of the steel area will be assumed to be $\sqrt{0.05^2 + 0.03^2}$ or simply it is 0.06.

4.1.4 ACCURACY OF ACI DESIGN EQUATIONS

Due to the use of Whitney's stress block, the limiting strains and the neglect of strain hardening zone, the strengths calculated by using ACI code may differ from the actual strengths even if the measured strengths of steel and concrete in the member from control specimens are used in calculation. For Tied columns, [10] found mean ratios of test to calculated strengths ranging from 0.97 to 1.00 with the coefficient of variation ranging from 0.046 to 0.074 including possible in-test variations. In this study the mean ratio of actual strength to design strength will be taken as P' =0.98 and its coefficient of variation V_p=0.05 [10] (Same features are adopted for moment equation)

4.2 *Φ* FACTOR FOR COLUMN

Strength reduction factor (Φ) for columns can be calculated as below [6]

$\Phi = \gamma_R e^{-\beta\alpha V_R}$

Where,

γ_R=R'/P_o

R' and P_o are the mean strength and Design strength respectively

β is safety index, taken as 3.5 for ductile failure and for brittle failure it can be assumed as 4 [6]. Cornell 1969 has developed this technique of First Order Second Method (FOSM). Similar approach has been utilized here considering the probabilities of variables as normally distributed over mean value with linear limit state [15]. [16] has described some other techniques of Monte-Carlo simulation and Bayseian updating for determination of the same safety index β but Cornell's approach is the simplest among all. [17] has worked on different scenario based reliability determination for stress with variation in their probability distribution functions and also discussed their respective factor of safety.

α is a separation function and assumed as 0.75 [11]
V_R is the coefficient of variation of random variable

The Axial capacity of column can be calculated as follow

$$P_o = 0.85\, f'_c\,(A_g - A_{st}) + A_s f_y \tag{1}$$

Design strength should be calculated using design yield strength of steel and concrete compression strength of required grade with given cross section.The mean strength (P_o') is calculated using mean value of steel yield strength and mean dimensions. The mean strength of steel in this study is found to be 74.33 ksi with the co-efficient of variation of 0.12 and due to the absence of data the average dimension sizes are assumed to be 0.05 in greater than the given dimensions with the standard deviation of 0.3 as recommended by [6]. Similarly the mean compressive strength of concrete is assumed to be 0.9 times of the desired strength with the co-efficient of variation of 0.18 [6].

Due to the use of Whitney's stress block, the limiting strains and the neglect of strain hardening zone, the strengths calculated by using ACI code may differ from the actual strengths even if the measured strengths of steel and concrete in the member from control specimens are used in calculation. For Tied columns, [10] found mean ratios of test to calculated strengths ranging from 0.97 to 1.00 with the coefficient of variation ranging from 0.046 to 0.074 including possible in-test variations. In this study the mean ratio of actual strength to design strength will be taken as P' =0.98 and its coefficient of variation V_p=0.05.

Therefore the value of design strength (P_o') must be corrected to allow for errors in Eq. (1) itself by multiplying the mean strength with P'=0.98 (i.e. ratio of actual strength to design strength)

$$R' = Po' * P'$$

Where, $P' = 0.98$

$$\gamma_R = R'/Po$$

4.2.1 COEFFICIENT OF VARIATION (V_R)

The strength of axially loaded column is the sum of the load carried by concrete (P_c) and the steel (P_s). The coefficients of variation of concrete and steel can be calculated separately. The load carried by the concrete is the product of variables like f'_c, b, and h, thus

$$V_{pc} = \sqrt{{V_{fc}}^2 + {V_b}^2 + {V_h}^2}$$

The standard deviation of the load carried by concrete is,

$$\sigma_{pc} = V_{pc} * P_c$$

P_c is the load carried by concrete. Similarly, the load carried by the steel is the function of its yield strength and area of steel, therefore

$$V_{ps} = \sqrt{{V_{fs}}^2 + {V_{As}}^2}$$

And its standard deviation is,

$$\sigma_{ps} = V_{ps} * P_s$$

P_s is the load carried by steel. Since the total load capacity is the sum of both the steel and concrete, hence

$$\sigma_{po} = \sqrt{{\sigma_{ps}}^2 + {\sigma_{pc}}^2}$$

And the cumulative coefficient of variation is,

$$V_{ps} = \frac{\sigma_{po}}{\mathrm{Po}}$$

This must be adjusted for the errors in equation used to compute load capacity

$$V_R = \sqrt{{V_{po}}^2 + {V_{pu}}^2}$$

V_{pu} is the variation in code equation used for the calculation of axial capacity

Table 1 show the parameters which are used in the calculation of Φ factor for axially loaded column. These parameters include the concrete compressive strength, steel yield strength and cross sectional dimensions. The standard deviations and coefficient of variation of all these variables are listed in table 1. The shaded region in table 1 are cross sectional variables and changes for each cross sectional size with respect to its steel area, dimensions etc. [18] conclude in his research that an equivalency relationship between reliability and factor of safety can easily be determined using simple monte carlo simulation and for small probability of failure subset simulation also works well. Thus it may be done in future easily for the factor of safeties found in this research to link with their corresponding reliabilities.

A number of different cross sections have been solved for computing strength reduction factor for axially loaded columns with variation of steel ratio from 2% to 3%. For eccentrically loaded column eccentricities are taken as fractions of balanced eccentricity. The results of these calculations are shown in figure 10 to 12.

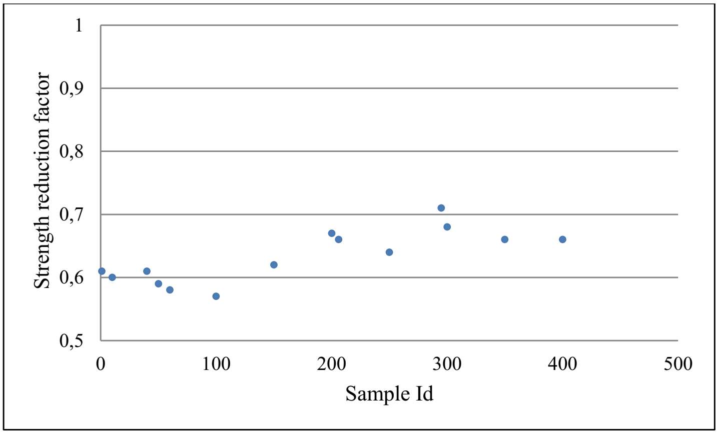


Figure 10: Variation in strength reduction factor for pure axial columns

A number of cross sections (almost 700) have been solved to get an average acceptable value of Φ, and it is average out to be 0.8 for the design purpose of eccentrically loaded columns for tension controlled sections

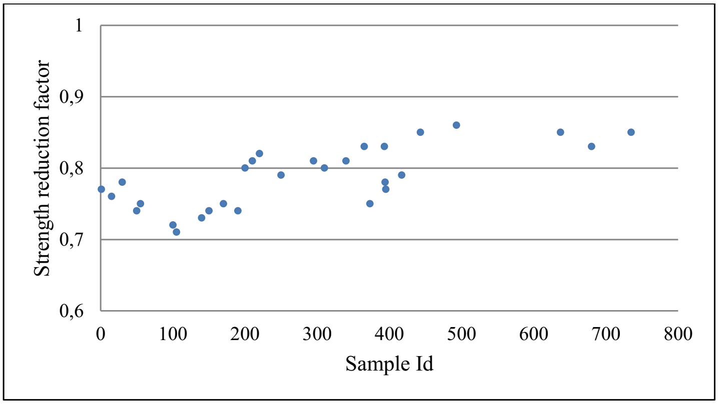


Figure 11: Variation in strength reduction factor for tension controlled sections

Different cross sections have been solved for computing strength reduction factor for compression controlled sections with variation of steel ratio from 1% to 3% at a minimum design eccentricity of $0.25e_b$ and an average value of strength reduction factor is approximated to 0.61 which can be rounded off to 0.6.

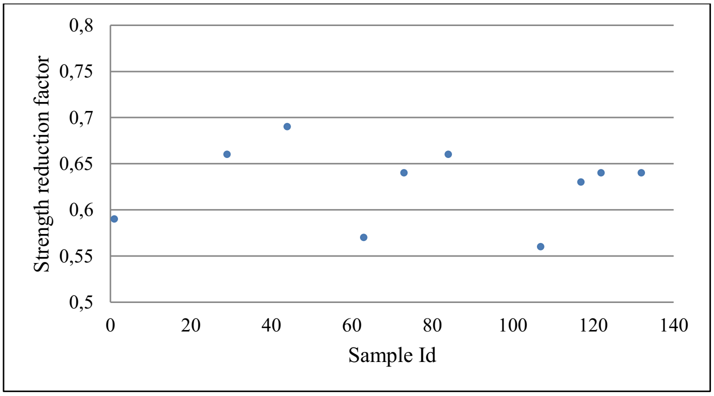


Figure 12: Variation in strength reduction factor for compression controlled sections

Table 1: List of variables used in calculation of Φ factor [6]

material strength	**specified**		**mean in situ**		**mean/specified**	**s.d**	**v**
concrete strength, f'_c	3000	Psi	2700	Psi	-	-	0.18
yield strength, f_s	60	Ksi	74.33	Ksi	-	-	0.12
Dimensions							
width, b (in)	14		14.05		1.003571429	0.3	0.021352
depth, h (in)	22		22.05		1.002272727	0.3	0.013605
As, Col (in2)	9.24		----		1	-	0.06
Av, Stirrups	0.22		0.22		1	-	0.06
Accuracy of code equation							
Pu, Axially loaded column	-		-		0.98	-	0.05
Mu, Eccentrically loaded column	-		-		0.98	-	0.05
Loadings							
dead load	-		-		1	-	0.07
max floor load in 30 yrs life	-		-		0.7	-	0.3
Structural analysis							
dead load effects	-		-		1	-	0.08
live load effects	-		-		1	-	0.2

5. CONCLUSION

It is concluded that as the design eccentricity increases, the cross section tends to fail in tensile way for both compression and tension controlled design. Moreover, load moment interaction shows that greater steel strength results in maximum compressive strength but it gives lower ductility. The study shows that greater percentage of steel in cross section may fail in compressive manner although it is designed for tension failure zone and it is due to the variation of steel strength data manufactured in Pakistan.

Based on the analysis and calculations made in this research, strength reduction factors for design of columns are modified for the design industry of Pakistan using the properties of locally manufactured steel rebar's. The modified values are as under:

- 0.63 for pure axial columns but existing value of 0.65 is acceptable
- 0.8 for tension controlled sections
- 0.6 for compression controlled sections

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LIST OF FIGURES AND TABLES

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