Chemical and Mechanical Properties of Steel Rebars Manufactured in Pakistan and Their Design Implications

Muhammad Masood Rafi[1]; Sarosh H. Lodi[2]; and Amir Nizam[3]

Abstract: The use of steel to reinforce concrete has been a long tradition, which is made possible by its strength and compatibility with concrete. Steel is also considered an ideal material for carrying out seismic-resistant construction due to its ductility and high-energy-absorption capacity. Raw materials from different sources are used in the manufacturing of steel bars in Pakistan. As a result, chemical composition, crystalline structure, and mechanical properties of these bars can vary from each other. This paper presents the results of chemical and mechanical tests on the cold-twisted ribbed and hot-rolled deformed steel reinforcing bars. Both types of bars are used in the construction industry in Pakistan. Although chemical composition of the bars met the requirements of the respective standards, a significant number of bars were unable to meet the specified strength and elongation requirements. The bar-failure percentage to meet the specified minimum yield strength was as high as 60%. A large variation in the data of rebar strength was noted. Design implications of using these bars were studied and it was found that the failure mode of flexural members may change from ductile to brittle. Therefore, suggestions are made for safe structural design. A reliability analysis is also carried out and modified values of strength-reduction factors in flexure and shear are suggested. **DOI: 10.1061/(ASCE)MT.1943-5533.0000812.** *© 2014 American Society of Civil Engineers.*

Author keywords: Reinforcing steel; Chemical properties; Tensile strength; Concrete beams; Flexural strength; Shear strength; Rotation.

Introduction

Concrete is a weak material in tension and cracks when local tensile stresses exceed its tensile strength. Consequently, concrete is reinforced to enable it to resist these stresses. Although the possibility of using reinforcing bars made of nonmetallic materials has been explored by different researchers (Nawy and Neuwerth 1977; Saadatmanesh and Ehsani 1991; Faza and GangaRao 1992; Nanni 1993; Ghavami 1995; Al-Salloum et al. 1996; Benmokrane et al. 1996; Kawai et al. 2000; Amada and Untao 2001; Toutanji and Deng 2003; Al-Negheimish and Al-Zaid 2004; Galati et al. 2006; Rafi et al. 2008; Rafi and Nadjai 2011; Noel et al. 2011), steel is still a material of choice to reinforce concrete (Goto 1971; Kankam 2004; Szota 2008). Various properties of steel, such as strength and compatibility with concrete, have made steel very popular in reinforced concrete (RC) and prestressed concrete applications. The use of reinforcing steel is increasing in the construction industry owing to the technological improvements, which have been made over the years (Baydogan et al. 2005). The high-energy-absorption capacity of steel makes it an ideal material to carry out construction in the seismically active regions. The current seismic design philosophy of RC structures relies heavily on the ductility and energy-absorption capacity of steel. These properties enable a structure to deform beyond elastic range and permit a redistribution of forces in the structure without collapse. Therefore, it is necessary that the physical properties of steel meet the demand of fundamental assumptions, which are made in the design of structural members (Kankam and Adom-Asamoah 2002).

The strength and ductility of steel are the foremost important mechanical properties for structural engineers. These are greatly affected by chemical composition, heat treatment, and the method of steel manufacturing (Kankam and Adom-Asamoah 2002). At the same time, the cost of steel is also dependent on these factors. In an attempt to control the cost, steel manufacturers in developing countries are forced to use recycled steel in order to produce economical steel reinforcing bars. For example, 80,000 t of steel scrap is recycled annually in Ghana (Kankam 2004). Pakistan is no exception and the steel manufacturers in Pakistan use different sources of raw material in the production of reinforcing bars. These bars drastically vary from each other in chemical composition, crystalline structure, and mechanical properties (Lodi and Masroor 1994). This study aims to investigate the chemical and mechanical properties of steel bars manufactured in Pakistan. Design implications of using these bars are discussed and recommendations are made.

Background and Scope

The construction industry in Pakistan is not organized scientifically (Rafi and Siddiqui 2010). As a result, unmonitored and uncontrolled construction is very common. This situation is further aggravated by the use of materials of unknown properties. The capacity of this construction to resist lateral seismic loads is uncertain. Because several parts of Pakistan lie in seismically active regions, this seismically deficient construction is a major concern from the viewpoint of life safety. The devastations made by the 2005 Kashmir earthquake can never be forgotten by the Pakistani nation. More than 73,000 people were killed and, at least, 69,000 people were injured (Earthquake Reconstruction and Rehabilitation

[1]Professor, Dept. of Earthquake Engineering, NED Univ. of Engineering and Technology, Karachi 75270, Pakistan (corresponding author). E-mail: rafi-m@neduet.edu.pk

[2]Dean, Faculty of Civil Engineering and Architecture, NED Univ. of Engineering and Technology, Karachi 75270, Pakistan. E-mail: sarosh.lodi@neduet.edu.pk

[3]Senior Laboratory Engineer, Dept. of Civil Engineering, NED Univ. of Engineering and Technology, Karachi 75270, Pakistan. E-mail: amirsahrai@hotmail.com

Note. This manuscript was submitted on December 6, 2012; approved on March 5, 2013; published online on March 7, 2013. Discussion period open until July 1, 2014; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Materials in Civil Engineering***, Vol. 26, No. 2, February 1, 2014. © ASCE, ISSN 0899-1561/2014/2-338-348/$25.00.

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Authority 2006) due to the collapse of structures during this earthquake. In addition, approximately 2.8 million people were made homeless owing to the damage of 450,000 buildings (Rossetto and Peiris 2009). Although the design of structures is carried out using the American Concrete Institute (ACI) 318R-02 (ACI 2002) code in Pakistan, the designers often do not have an effective control on the materials used during construction due to various factors such as unreliable test reports, inconsistent material properties, etc. As a result, the capacity of these structures to resist forces assumed in the design is uncertain.

There are three sources of raw material used for the manufacturing of steel rebars, which are used for concrete reinforcement in Pakistan. In brief, the first of these sources is steel billet, which is produced by Pakistan Steel Mills Corporation Ltd. Two types of bars are produced from this billet. These include cold-twisted ribbed and hot-rolled deformed bars. The former type has oblique indentations and is manufactured by cold twisting of mild-steel reinforcing bars. These are usually available with the trade name of TOR bars. Although the cold-twisted ribbed bars are not allowed by the ASTM standards [such as A615/A615M-05a (ASTM 2005)], as the ductility of these bars is unknown, they are used for the construction of structures, which are designed using the ACI code. Other sources of raw material for steel manufacturing are ship plates and ingots, which are made from locally generated and imported scrap (Lodi and Masroor 1994). Only cold-twisted ribbed bars are manufactured from these materials and their use in the manufacturing of rebars can be attributed to the absence of technolegal regime and inadequate implementation of bar manufacturing standards. The use of nonstandard bars is an area of gross violation of approved design and is a major concern for structural engineers in Pakistan (Lodi and Masroor 1994). The hot-rolled deformed bars are manufactured by rerolling of high-strength billet. As mentioned earlier, this billet is produced by Pakistan Steel Mills Corporation Ltd. These bars have both the longitudinal and circumferential indentations on them and are generally considered to be of superior quality compared with the cold-twisted ribbed bars.

A typical stress-strain curve for steel consists of a linear elastic portion, which is followed by a yield plateau. The length of yield plateau generally depends on the strength of steel. A strain-hardening zone follows the yielding of bar until its failure. The strain-hardening zone provides ample rotation capacity of a plastic hinge in normally designed RC structures and prevent their sudden failure. Fig. 1 illustrates typical stress-strain curves for both hot-rolled deformed and cold-twisted ribbed bars available in Pakistan. It is noted in Fig. 1 that although the former bar curve exhibits the aforementioned characteristics expected of steel reinforcing rebar, it is hard to identify yield point on the latter bar curve. Similarly, the

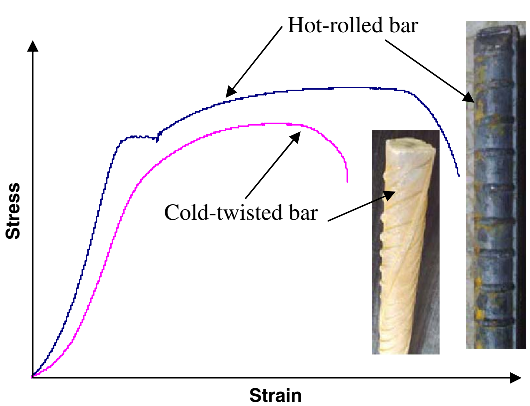

Fig. 1. Typical stress-strain curves of bars

cold-twisted ribbed bar has a small strain-hardening zone compared with a hot-rolled deformed bar. Nonetheless, the cold-twisted ribbed bars are also demanded by the local construction industry owing to their lower prices compared with the hot-rolled deformed bars.

Pakistan Standards and Quality Control Authority (PSQCA) is the national standardization body in Pakistan. Its functions are advising the government on standardization policies, programs, and activities to promote industrial efficiency and development (PSQCA 2011). The PSQCA has recommended the use of BS 4449 [British Standards Institution (BSI) 1997] as a standard for the manufacturing of cold-twisted ribbed bars, whereas the hot-rolled deformed bars are manufactured to comply with the requirements of A615/A615M-05a (ASTM 2005). This paper presents the results of tests, which were conducted on both types of bars. The data of chemical composition, yield strength (f_y), tensile strength (f_u), and elongation of the tested bars are compared with the requirements as specified by the aforementioned standards and the results are discussed. In addition, bend tests were also carried out on the bars as suggested by BS 4449 (BSI 1997) and A615/A615M-05a (ASTM 2005). The presented data are essential to develop realistic models of the capacity of existing building stock to resist a maximum considered earthquake. Implications of design using these bars are investigated, which indicated deficiencies in the design based on the ACI code method. Empirical studies are conducted and modifications are suggested to ensure safe structural design. Further, a reliability analysis is carried out to suggest resistance factors for flexural members, which are designed using the bars manufactured in Pakistan.

Experimental Program

The Department of Civil Engineering at NED University of Engineering and Technology offers rebar testing services commercially to the construction industry. The tests include yield strength, tensile strength, elongation, and bend tests. Thousands of bar samples are tested each year and the data of these tested bars are stored in a database. The data in the presented paper are taken from this database for the bars tested in 2009 and 2010. The presented data of rebars consist of approximately 2,400 hot-rolled deformed and 2,200 cold-twisted ribbed bars of different diameters. These bars were tested on either Shimadzu UH-500 KNI or WAW 2000A (Times Group China) type universal testing machines. In addition, some of the tested rebars were selected randomly for chemical analysis tests. These tests consisted of spectrographic analysis according to ASTM A751 (ASTM 2007). Chemical tests were performed to determine the proportion of carbon and other alloying additives such as manganese, sulfur, and phosphorous used in the manufactured bars. The diameter of the bars tested are 10 (Number 3), 12, 13 (Number 4), 16 (Number 5), 19 (Number 6), 20, 22 (Number 7), 25 (Number 8), 32 (Number 10), and 40 mm. The numbers in the brackets show the equivalent A615/A615M-05a (ASTM 2005) designations of reinforcing bars. A summary of sample size of the tested bars is provided in Table 1.

Reliability Analysis

Reliability refers to the probabilistic measure of satisfactory performance with respect to a prescribed condition (such as collapse). Reliability can be viewed as a complementary function of the probability of failure (P_f). It refers to the probability of load (Q) becoming greater than resistance (R). The reliability analysis for the presented study was carried out using the procedures developed

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Table 1. Details of Tested Bars

Bar diameter (mm)	Number of samples	
	Hot-rolled deformed bar	Cold-twisted ribbed bar
10 (Number 3)	437	410
12	88	119
13 (Number 4)	430	199
16 (Number 5)	502	300
19	311	197
20	80	154
22 (Number 7)	23	95
25 (Number 8)	451	347
32 (Number 10)	119	—
40	—	381
Total	2,441	2,202

by Cornell (1969) and Lind (1971). The derivation of equation for calculating ϕ is given previously (MacGregor 1976). For a log-normal distribution, P_f is given as follows:

$$P_f = P[\ln(R/Q) < 0] = \varphi(\beta) \tag{1}$$

where $\varphi(x)$ = standard normal cumulative distribution function and β is defined as safety index. A log-normal distribution is assumed for Eq. (1) as it provides a better representation of R/Q. For a random variable Y, safety index is given as follows:

$$\beta = \frac{\ln \alpha_r - \omega_Y}{V_Y} \tag{2}$$

where V_Y = coefficient of variation of Y; ω = log-normal distribution parameter [Eq. (3)]; $\alpha_r = R_m/R_u$, where R_m and R_u are the mean and design strengths, respectively.

$$\omega_Y = \ln \mu_Y - \frac{1}{2}V_Y^2 \tag{3}$$

where μ_Y = mean of random variable Y.

The reliability (R_t) of the structure is calculated as follows:

$$R_t = 1 - P_f \tag{4}$$

The safety provisions in the current limit state design procedure of the ACI code use strength-reduction and load factors and are expressed in the form of Eq. (5)

$$\phi R \geq \kappa Q \tag{5}$$

where ϕ = strength-reduction factor and k = load factor.

The right-hand side of Eq. (5) represents the design strength of a RC member, whereas the left-hand side refers to the actual strength. Loads in Eq. (5) may include either external loads or their internal effects such as moment, shear, thrust. In specific terms, Eq. (5) may be expressed as Eq. (6) for a flexural member, which is subjected to bending and shear, as follows:

$$\phi M_n \geq M_u \qquad \phi V_n \geq V_u \tag{6}$$

where M_n and V_n = actual strengths in flexure and shear, respectively; M_u and V_u = design strengths in bending and shear, respectively. The value of ϕ depends on both the variance of α_r and the value of β, and is calculated by Eq. (7) as follows:

$$\phi = \alpha_r \exp(-\beta\eta V_R) \tag{7}$$

where η = separation function and is equal to 0.75 (Lind 1971); V_R = coefficient of variation of R_m/R_u.

Table 2. Data of Mean, Standard Deviation, and Coefficient of Variation

Parameter	μ	α	Coefficient of variation
Concrete compressive strength (MPa)	Eq. (8)	—	0.18
Steel yield strength (MPa)	498.83	73.55	0.15
Beam width (mm) (+)	1.30	8.00	—
Beam effective depth (mm) (−)	0.40	10.00	—
Stirrup spacing (mm)	—	—	0.03
Beam tension steel area (mm^2)	—	—	0.06
Beam stirrup steel area (mm^2)	—	—	0.06
Moment	—	—	0.04
Concrete shear strength	—	—	0.15
Steel stirrup shear strength	—	—	0.15

The details of calculation of V_R can be reviewed in MacGregor (1976). The mean concrete strength (f_{cm}) was calculated from Eq. (8), as suggested by Allen (1968), Petersons (1964), and Bloem (1968)

$$f_{\mathrm{cm}} = 0.675 f_c + 1.1 \leq 1.15 f_c \tag{8}$$

where f_c = 28-day concrete compressive strength in ksi. Note: 1 MPa = 145 psi.

Because the data on mean and standard deviation for beam sizes, concrete strength, and loads are not available in Pakistan, these have been assumed based on the studies conducted by Bloem (1968), Fiorata (1973), and Mattock et al. (1961) and are provided in Table 2.

Discussion on Results

In general, all the bars were found to be satisfactory in the bend tests with a failure percentage less than 0.5%, which was considered insignificant. Consequently, the bend tests are not covered in the subsequent discussion. The results of other mechanical and chemical tests are discussed in the following sections for both the hot-rolled deformed and cold-twisted ribbed bars.

Hot-Rolled Deformed Bars

Table 3 shows the results of chemical composition tests on the hot-rolled deformed bars tested. Although A615/A615M-05a (ASTM 2005) recommends the chemical tests to determine the percentages of carbon, manganese, phosphorous, and sulfur, it suggests the maximum contents only for phosphorous, which are also included in Table 3. It is noted in Table 3 that most of the bars satisfy the A615/A615M-05a (ASTM 2005) requirements for the percentage of phosphorous. Only the 12-mm diameter bars contained more phosphorous contents than those recommended by A615/A615M-05a (ASTM 2005). However, this could be the result of normal scattering in the observed data. Further, the carbon contents in most of the bars are less than 0.2% and are within the allowable limits for reinforcing steel.

The A615/A615M-05a (ASTM 2005) standard recommends minimum f_y as 420 MPa for Grade 60 bars, whereas f_u has been

Table 3. Data of Chemical Analysis of Hot-Rolled Deformed Bars

Bar diameter (mm)	Carbon (%)	Manganese (%)	Sulfur (%)	Phosphorous (%)	
				Observed	ASTM A615
10	0.14	0.70	0.035	0.021	0.06
12	0.16	0.74	0.018	0.09	0.06
16	0.12	0.82	0.012	0.010	0.06
20	0.21	0.90	0.025	0.013	0.06
25	0.11	0.48	0.012	0.016	0.06

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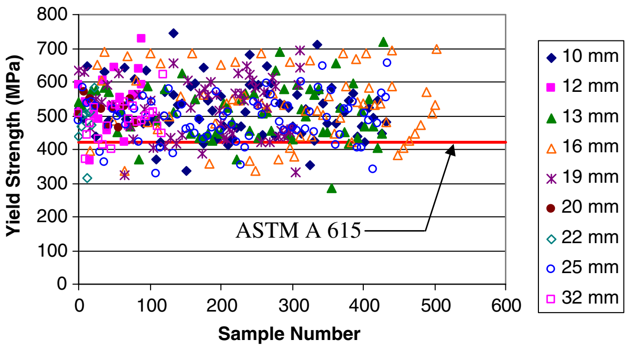

Fig. 2. Data of yield strength of hot-rolled deformed bars

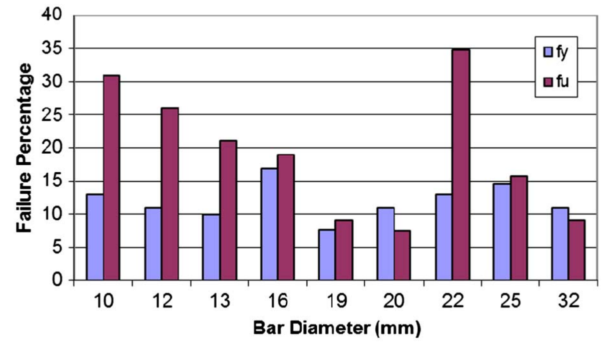

Fig. 4. Failure percentage of hot-rolled deformed bars

suggested as 620 MPa. Figs. 2 and 3 illustrate the data of, respectively, f_y and f_u of the tested bars of different diameters. The results of only a few bars have been included in Figs. 2 and 3 for clarity. The aforementioned A615/A615M-05a (ASTM 2005) requirements for yield and ultimate strengths have also been indicated in Figs. 2 and 3. It is noted in Fig. 2 that yield strength of approximately 13% samples fall below the required strength. Similarly, 19% samples do not meet the requirement of ultimate strength as given by A615/A615M-05a (ASTM 2005; Fig. 3). Because both these failure percentages are greater than 10%, these can be considered as significant failure percentages. Fig. 4 presents the individual percentages of bars of different diameters failing the requirements for yield and ultimate strengths as given by A615/A615M-05a (ASTM 2005). It is noted in Fig. 4 that the failure percentage for most of the bars is more than 10% for yield strength and is in the range of 10–15%. Further, failure percentage of bars to meet the ultimate strength requirements is significantly higher, which is more than 30% for 10- and 22-mm diameter bars. Only 19-, 20-, and 32-mm diameter bars have failure percentages less than 10% for ultimate strength. In addition, a large scatter in the strength data is observed in Figs. 2 and 3 beyond that recommended by the A615/A615M-05a (ASTM 2005). The bar strengths varied up to more than 70% above the required strength.

The results of elongation tests carried out for different diameter bars are plotted in Fig. 5 along with the requirement as given by A615/A615M-05a (ASTM 2005). It is noted in Fig. 5 that only a small proportion of the tested bars (less than 1%) is unable to meet the elongation requirements and majority of bars satisfy these requirements. Further, of all the failing bars, mostly 10-mm diameter bars do not meet the elongation requirements of A615/A615M-05a (ASTM 2005).

Fig. 6 presents the data of standard deviation for both f_y and f_u of the tested bars. It is seen in Fig. 6 that, except for 10-mm diameter bars, standard deviation for the rest of bars appears to be consistent and is close to 70 MPa for both f_y and f_u, which indicates consistent mechanical properties of these bars. A high standard deviation for 10-mm diameter bars indicates large variation in their material properties. As mentioned earlier, 10-mm diameter bars also have a comparatively higher failure proportions (Fig. 4) to meet the strength requirements of A615/A615M-05a (ASTM 2005). Because these bars are mainly used in stirrups, these results indicate that the ability of a structure to resist seismic loads could be greatly compromised as stirrups not only assist concrete to resist design shear force, but are also considered essential for ductile detailing and construction, such as beam-column joints.

Cold-Twisted Ribbed Bars

The results of chemical analysis of cold-twisted ribbed bars are provided in Table 4. As mentioned earlier, the PSQCA recommends the use of BS 4449 (BSI 1997) as a manufacturing standard for

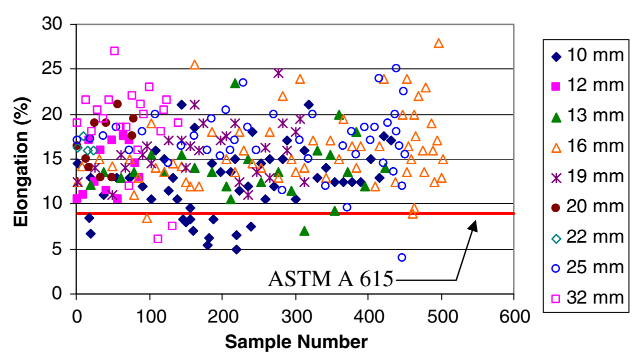

Fig. 5. Data of elongation test results for hot-rolled deformed bars

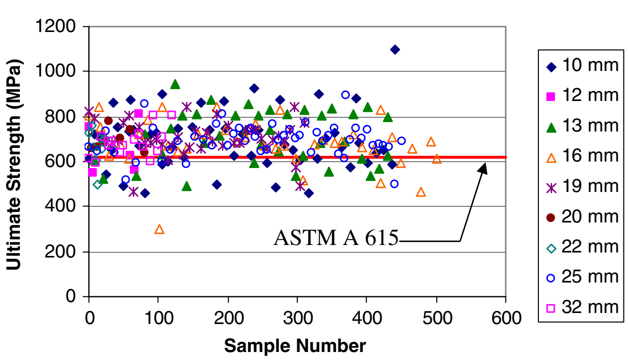

Fig. 3. Data of tensile strength of hot-rolled deformed bars

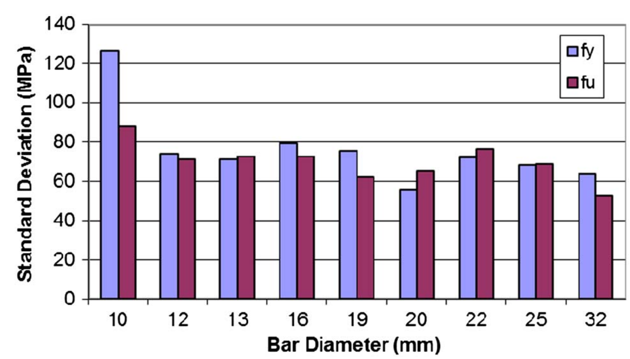

Fig. 6. Standard deviation in the hot-rolled deformed bar strength

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Table 4. Data of Chemical Analysis of Cold-Twisted Ribbed Bars

Bar diameter (mm)	Carbon (%)		Manganese (%)	Sulfur (%)		Phosphorous (%)	
	Observed	BS 4449	Observed	Observed	BS 4449	Observed	BS 4449
10	0.22	0.25 ± 0.02	0.48	0.031	0.05 ± 0.005	0.050	0.05 ± 0.005
12	0.18	0.25 ± 0.02	0.54	0.032	0.05 ± 0.005	0.020	0.05 ± 0.005
16	0.14	0.25 ± 0.02	0.71	0.026	0.05 ± 0.005	0.047	0.05 ± 0.005
20	0.17	0.25 ± 0.02	0.55	0.016	0.05 ± 0.005	0.037	0.05 ± 0.005
25	0.14	0.25 ± 0.02	0.51	0.021	0.05 ± 0.005	0.070	0.05 ± 0.005

cold-twisted ribbed bars. The recommended percentages of different elements as suggested by BS 4449 (BSI 1997) are also given in Table 4. It is noted in Table 4 that nearly all the bars meet the chemical composition requirements as given by BS 4449 (BSI 1997). Only the 25-mm diameter bars showed higher phosphorous contents. However, this could be attributed to the normal scattering in the observed data.

The yield and ultimate strengths for Grade 460B bars are specified, respectively, as 460 and 497 MPa by BS 4449 (BSI 1997). The f_y value specified in BS 4449 (BSI 1997) is 40 MPa higher than the yield strength of 420 MPa as specified by A615/A615M-05a (ASTM 2005), whereas tensile strength of 497 MPa given by BS 4449 (BSI 1997) is significantly less than the specified strength in A615/A615M-05a (ASTM 2005). Figs. 7 and 8 present the data of rebar strength of different diameter bars tested. The results of a few bars have been shown in Figs. 7 and 8 for clarity. The minimum specified values of strengths, as given by BS 4449 (BSI 1997), are also shown in Figs. 7 and 8. It is seen in Fig. 7 that a significantly large proportion of the tested bars is unable to meet the criterion of yield strength of bars suitable for concrete reinforcement, as specified by BS 4449 (BSI 1997): this proportion comes out to be approximately 33%. Similarly, 13% of rebars were found to be falling short of the tensile strength requirements as suggested by BS 4449 (BSI 1997; Fig. 8). This is despite the fact that ultimate tensile strength is only 8% greater than the yield strength in BS 4449 (BSI 1997). Further, it is clear that the failure percentage of bars to meet the yield strength requirement in this case is considerably higher than what was noted in the case of hot-rolled deformed bars (Fig. 2). Fig. 9 illustrates individual failure percentages of the bars according to their sizes. It is noted in Fig. 9 that mostly small diameter bars (10, 12, and 13 mm) have higher failure percentages. In particular, nearly 60% of 10-mm diameter bars tested failed to meet the requirements of minimum yield strength as specified by BS 4449 (BSI 1997). Failure percentage of 10-mm diameter bars to satisfy the ultimate tensile strength requirements is also the highest of all the tested bars of different diameters (Fig. 9). As mentioned earlier, 10-mm diameter bars are usually preferred for stirrups in beams and columns in Pakistan. A higher failure percentage of these bars could result in shear-deficient construction. It is further noted in Figs. 8 and 9 that scattering in the strength data for cold-twisted ribbed bars is similar to the hot-rolled deformed bars, as mentioned earlier.

Fig. 10 illustrates the results of rebar-elongation tests. In contrast to what was noted for hot-rolled deformed bars (Fig. 5),

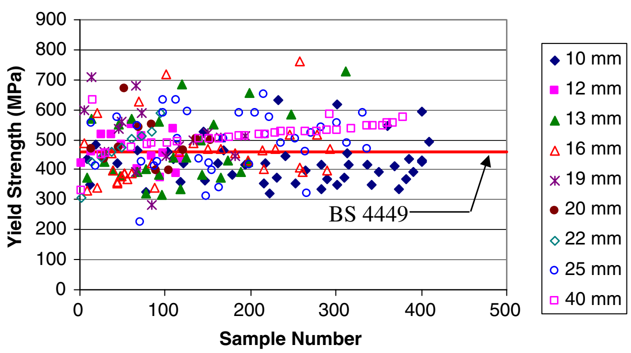

Fig. 7. Data of yield strength of cold-twisted ribbed bars

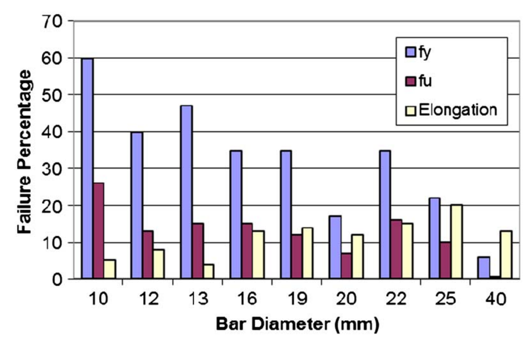

Fig. 9. Individual failure percentage of cold-twisted ribbed bars

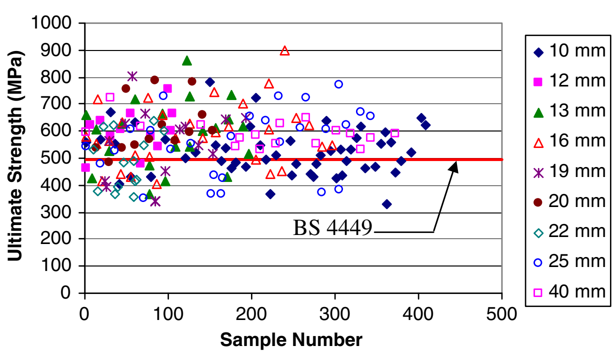

Fig. 8. Data of tensile strength of cold-twisted ribbed bars

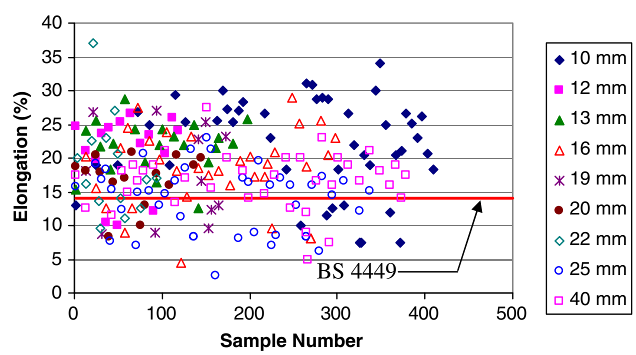

Fig. 10. Data of elongation test for cold-twisted ribbed bars

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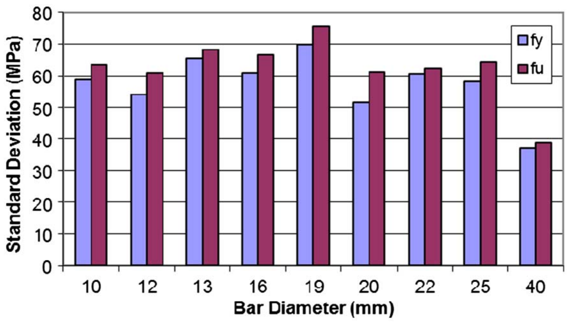

Fig. 11. Standard deviation in the cold-twisted ribbed bar strength

considerably large numbers of cold-twisted ribbed bars (12%) are unable to meet the elongation requirements as given by BS 4449 (BSI 1997). Individual percentages of different diameter bars failing to meet the elongation requirements are presented in Fig. 9. It is seen in Fig. 9 that 25-mm diameter bars have the largest failure record (20%) in the elongation test. Most of the other diameter bars also have more than 10% failure. Because the elongation is an indirect measure of ductility of bars, these results indicate that a significant proportion of the tested bars lack ductility, which is an essential requirement for seismic-resistant construction.

The values of standard deviation in the data of f_y and f_u are illustrated in Fig. 11. It is seen in Fig. 11 that the values of standard deviation vary from each other for different diameter bars, which is an indication of inconsistent mechanical properties of these bars. These results further strengthen the earlier observation that these bars cannot be relied upon to carry out construction in seismically prone areas.

Design Implications

Flexural Design

As mentioned earlier, the ACI 318R-02 (ACI 2002) is used for the design of RC structures in Pakistan. The ACI code recommends an underreinforced section design to avoid sudden failure. This is achieved by providing a tension steel ratio (ρ) less than the maximum ratio (ρ_{max}). This latter ratio is based on steel strain, balanced reinforcement ratio (ρ_{bal}), and f_y of steel. The balanced reinforcement ratio implies a condition when tension steel yields at a strain of f_y/E_s (where E_s is the modulus of elasticity of steel) at the same time as the concrete fails after reaching a maximum strain of 0.003. The maximum reinforcement ratio is calculated by Eq. (9). The ACI code also provides a lower limit on the steel percentage (ρ_{min}) used in a cross section, which is given by Eq. (10). For a concrete strength (f_c) of 30 MPa, $f_y = 420$ MPa and $E_s = 200$ GPa, ρ_{max} and ρ_{min} values come out to be 0.0191 and 0.0033, respectively, from Eqs. (9) and (10). The aforementioned concrete strength is arbitrarily selected and a range of strength has been considered later in the discussion.

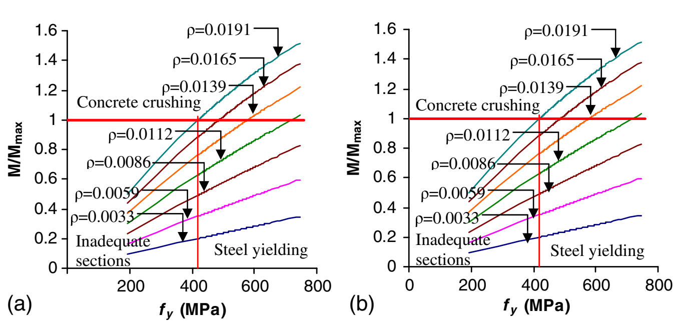

Fig. 12. Capacity of beam cross section: (a) hot-rolled deformed bars; (b) cold-twisted ribbed bars

$$\rho_{max} = \left(\frac{0.003 + \frac{f_y}{E_s}}{0.008}\right)\left(0.85\beta_1 \frac{f_c}{f_y} \times \frac{600}{600 + f_y}\right) \tag{9}$$

where $\beta_1 = 0.85 - 0.05[(f_c - 28)/7] \geq 0.65$.

$$\rho_{min} = \frac{1.4}{f_y} \tag{10}$$

The nominal moment capacity of an underreinforced RC section can be calculated by multiplying the tension force in steel by its arm up to the center of compression force in concrete [Eq. (11)] as follows:

$$M = \rho b d f_y \left(d - \frac{a}{2}\right) \tag{11}$$

where $a = \rho d f_y / 0.85 f_c$; b = width; and d = effective depth of cross section.

Fig. 12 illustrates variation in an underreinforced beam cross-section capacity [Eq. (11)] with different f_y values for both the hot-rolled deformed and cold-twisted ribbed rebars (Figs. 2 and 7). The ρ values vary between ρ_{min} and ρ_{max} in Fig. 12 and the moment capacity has been normalized to the maximum moment (M_{max}), which is calculated from Eq. (11) by substituting $f_y = 420$ MPa and ρ_{max}. The effective beam depth has been assumed as $0.8h$, where h is the beam height. It is noted in Fig. 12 that a linear relationship between normalized moment and f_y exists at lower ρ values, whereas the curves become nonlinear at higher reinforcement ratio. The horizontal line drawn at an ordinate value of unity in Fig. 12 represents the condition in which section capacity becomes equal to the maximum capacity allowed by ACI code for an underreinforced section for the previously assumed cross-section properties. Fig. 12 can be divided into three regions by drawing a line perpendicular to the aforementioned horizontal line, which intersects the abscissa at $f_y = 420$ MPa. The region on the left of the vertical line and below the horizontal line represents the sections with inadequate capacity compared with the capacity assumed in the design. The region on the right of the vertical line and below the horizontal line represents underreinforced sections, which fail by steel yielding. The region above the horizontal line consists of cross sections failing by concrete crushing. All these sections were designed as underreinforced and their capacities were determined using Eq. (11). However, the concrete strains are higher in these sections as the cracked neutral axis (NA) moves downward to satisfy force equilibrium owing to higher f_y values in the steel. As a result, maximum concrete strain of 0.003 can be reached at a steel strain less than f_y/E_s. It is noted in Fig. 12 that the results for both the hot-rolled deformed and cold-twisted ribbed bars are similar. This is owing to the fact that the range of variation in the data of yield strength of both types of bars is also similar (Figs. 2 and 7).

The previous discussion indicated problems related to both understrength and overstrength steel bars. The results of very high rebar strength compared with the assumed design strength further strengthened the earlier observations of low level of control over the use of nonstandard steel bars. The ACI 318R-02 (ACI 2002) code requires that the bar yield strength as determined by the mill test should not be greater than 545 MPa for a Grade 60 bar in seismic regions. Keeping in view the existing factors of ineffective quality control on the bar manufacturing and reliability

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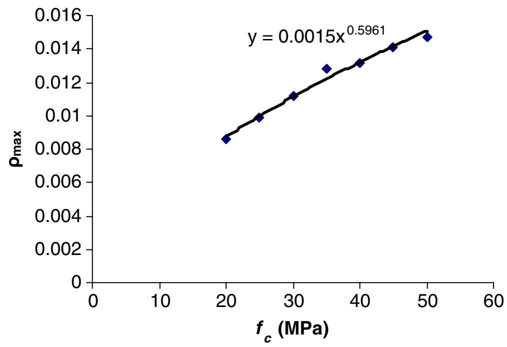

Fig. 13. Relationship of $\rho_{\max}$ with concrete strength

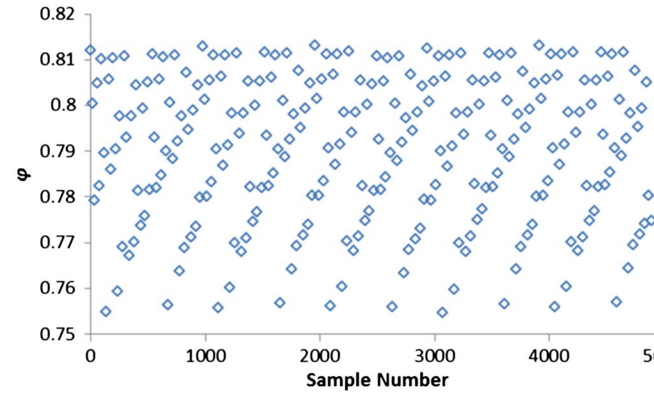

Fig. 14. Data of ϕ in flexure for the underreinforced beams

of material test results, the authors have used a two-pronged approach in this study. These are aimed at suggesting changes in the design procedure to increase the level of safety in the design. The forthcoming discussion is based on the assumption that bars failing to meet the strength requirements of respective standards will not be permitted for use in construction.

It is noted in Fig. 12 that a section remains underreinforced for all f_y values up to a ρ close to 0.0112. This applies to both the hot-rolled deformed and cold-twisted ribbed bars. An attempt has been made here to modify Eq. (9) empirically to develop a more accurate estimation of $\rho_{\max}$ to prevent the cross section from becoming overreinforced. Efforts have been made to introduce such changes that will allow the basic design process to remain close to the original ACI code design as the designers are familiar with this code.

The aforementioned procedure (Fig. 12) was repeated for the sections made of concrete strengths varying between 20 and 50 MPa. The effective depth was varied as $0.8h$ and $0.9h$. The strength is increased in increments of 5 MPa and a unique value of $\rho_{\max}$ was obtained at the corresponding f_c value. The $\rho_{\max}$ value here represents ρ which keeps the section underreinforced, as in Fig. 12. The results are graphically represented in Fig. 13 and a simplified relationship of $\rho_{\max}$ was obtained by the regression analysis, which is given in Eq. (12). The correlation coefficient for Eq. (12) comes out to be 0.98. Unless a better quality control is achieved in the rebar production, the maximum steel reinforcement ratio can be limited as follows:

$$\rho_{\max} = 0.0015(f_c)^{0.6} \tag{12}$$

Further, as mentioned earlier, a large variation in the rebar yield strength was noted in the aforementioned discussion (Figs. 2 and 7). The cumulative standard deviation in the bar f_y comes out to be 60 and 75 MPa, respectively, for the cold-twisted ribbed and hot-rolled deformed bars. It is suggested that the steel bars in addition to satisfying the strength requirements as given in their respective standards should also provide guaranteed yield strength of 420 MPa, calculated as the mean f_y minus the aforementioned standard deviation [Eq. (13)]. This provides 95% confidence interval of unity for both rebar types.

$$f_y = (f_y)_{\text{avg}} - \sigma \tag{13}$$

In addition to the aforementioned analysis, a reliability analysis is carried out using the procedure mentioned earlier in this paper. A total of 4,900 beam cross sections were analyzed using the aforementioned beam sizes, reinforcement ratios, and concrete strengths. The value of β was taken as 3.5 (MacGregor 1976) and a unique value of ϕ was calculated for each beam with the help of Eq. (7). These data are plotted in Fig. 14. It is noted in Fig. 14 that ϕ varies between 0.75 and 0.81 for the analyzed beams. These values are in agreement with the findings by Kent and Park (1972). For the data plotted in Fig. 14, the mean value of ϕ comes out to be 0.79 with a standard deviation of 0.016. It is suggested that the value of ϕ can be approximated to 0.80 for flexure design using the bars manufactured in Pakistan. This value is less than the recommended ϕ of 0.9 by ACI code.

Although overreinforced design ($\rho > \rho_{\text{bal}}$) (controlled by concrete crushing) is not allowed by ACI code, reliability analysis was also carried out for overreinforced beams to provide comprehensive guidelines for flexure design using the locally manufactured bars. For overreinforced sections, nominal moment capacity can also be calculated with the help of Eq. (11) by replacing f_y with steel stress (f_s); this can be determined from Eq. (14), which is obtained from strain compatibility and equilibrium conditions as follows:

$$f_s = \sqrt{\frac{(E_s\varepsilon_c)^2}{4} + \frac{0.85\beta_1 f_c}{\rho} E_s\varepsilon_c} - \frac{E_s\varepsilon_c}{2} \leq f_u \tag{14}$$

The reliability analysis for overreinforced beams was carried out using the same beam cross sections as for underreinforced beams. The reinforcement ratio was increased incrementally from 1% to 10% above ρ_{bal} in these beams. The value of β was taken as 4.0 for brittle beam design (MacGregor 1976). The data of calculated values ϕ for the employed beams are plotted in Fig. 15. It is noted in Fig. 15 that the values of ϕ are in the range of 0.5–0.65 for overreinforced beams. The average value of ϕ comes out to be 0.61 with a standard deviation of 0.02; this can be rounded off to 0.6.

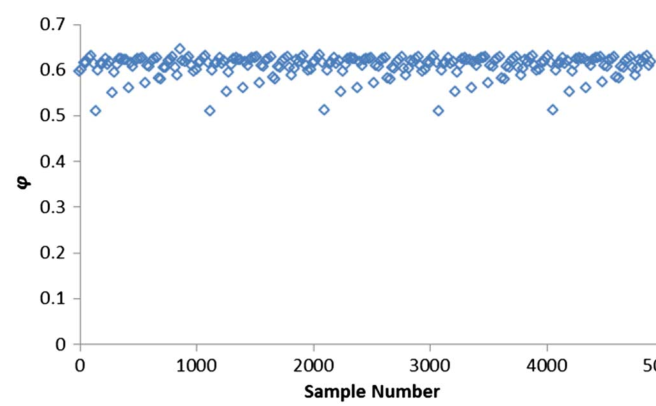

Fig. 15. Data of ϕ in flexure for the overreinforced beams

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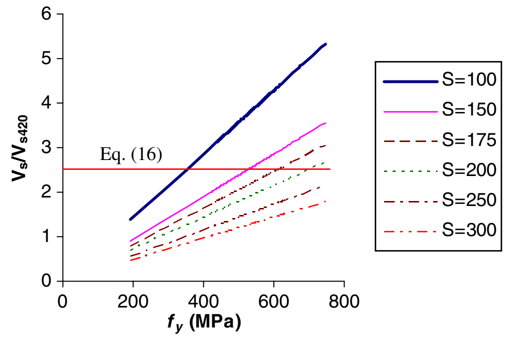

Fig. 16. Normalized V_s for the beam cross sections

Shear Design

The nominal shear strength of a beam is taken as a sum of resistance provided by the concrete [V_c; Eq. (15)] and shear reinforcement (V_s). The shear reinforcement is provided in the form of vertical rings (stirrups) and their shear capacity is calculated by Eq. (16). The maximum spacing (S) of stirrups is taken as 600 mm if $V_s \le 0.33\sqrt{f_c}bd$ and 300 mm if $V_s > 0.33\sqrt{f_c}bd$. A section is considered inadequate in shear for the conditions given in Eq. (17)

$$V_c = 0.17\sqrt{f_c}bd \tag{15}$$

$$V_s = \frac{A_v f_y d}{S} \tag{16}$$

where A_v = area of shear reinforcement.

$$V_s > 0.67\sqrt{f_c}bd \tag{17}$$

To study the effects of rebar strength on shear design, a beam of width 150 mm and depth 450 mm was analyzed. The beam effective depth is taken as $0.8h$ and 10-mm diameter bar was assumed for stirrups. The spacing of stirrups was varied from 100 to 300 mm in 50-mm intervals and f_c was taken as 30 MPa. Fig. 16 illustrates changes in V_s at a few S values for the beam. The V_s values in Fig. 16 have been normalized to maximum $V_s(V_{s420})$ which is calculated using $f_y = 420$ MPa and $S = 300$ mm keeping the rest of the beam properties the same. The limiting value of normalized V_s as determined by Eq. (16) is also plotted in Fig. 16 as a solid horizontal line. It is noted in Fig. 16 that, for each ring spacing used in the analysis, normalized V_s increases with f_y, as can be expected from Eq. (16). By contrast, normalized V_s reduces as S is increased.

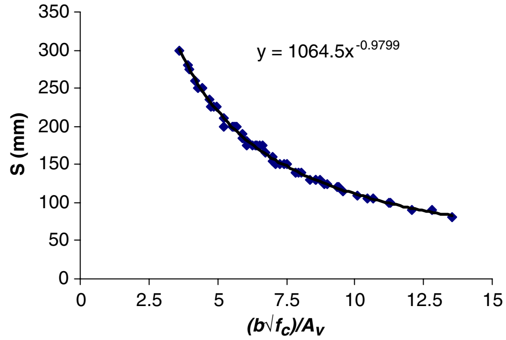

Fig. 17. Results of regression analysis for stirrup spacing

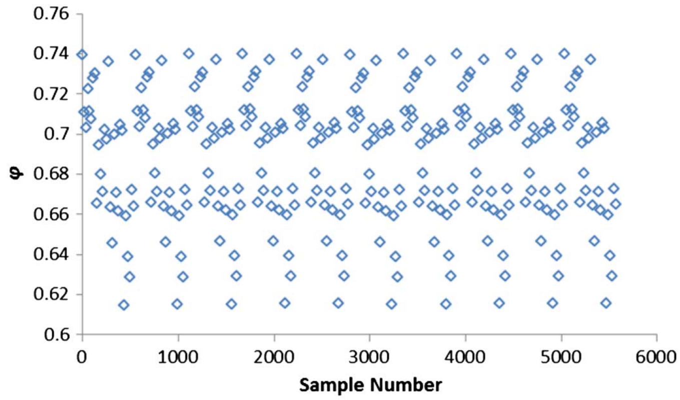

Fig. 18. Data of ϕ for beams in shear

This reduction in the normalized V_s becomes small at higher ring spacing. Further, the normalized V_s becomes greater than calculated from Eq. (17) at ring spacing less than 200 mm. As a result, the concrete section becomes deficient in shear and requires revised dimensions to avoid brittle failure. This indicates that the ring spacing in these sections cannot be taken less than 200 mm. The aforementioned analysis was repeated using different combinations of beam depths of 600, 750, and 900 mm, beam widths of 200, 250, and 300 mm, 12-mm diameter bar for ring, and with d taken as $0.9h$. The results of minimum ring spacing from this analysis are plotted in Fig. 17 and a relation of minimum spacing ($S_{\min}$) was determined by the regression analysis [Eq. (18)]. The correlation coefficient for Eq. (18) comes out to be 0.99. It is suggested that the spacing of shear reinforcement in the beams must not be kept less than that determined from Eq. (17), keeping the rest of the design procedure the same as stated by ACI code.

$$S_{\min} = 1{,}065\left(\frac{A_v}{b\sqrt{f_c}}\right) \tag{18}$$

Similar to the aforementioned flexure design, the reliability analysis is carried out using the aforementioned data of beam dimensions, size of stirrups, and concrete strengths. A total of 5,600 beams were analyzed and ϕ values were calculated separately for each beam using the procedure stated earlier in this paper. The value of β was taken as 4.0 for shear, as suggested by MacGregor (1976). The resulting data are illustrated in Fig. 18. It is noted in Fig. 18 that ϕ varies between 0.60 and 0.75. The mean value of ϕ in this case is calculated as 0.69 with a standard deviation of 0.033.

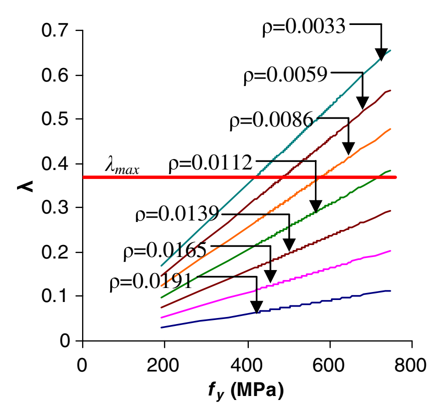

Fig. 19. Variation in λ versus f_y

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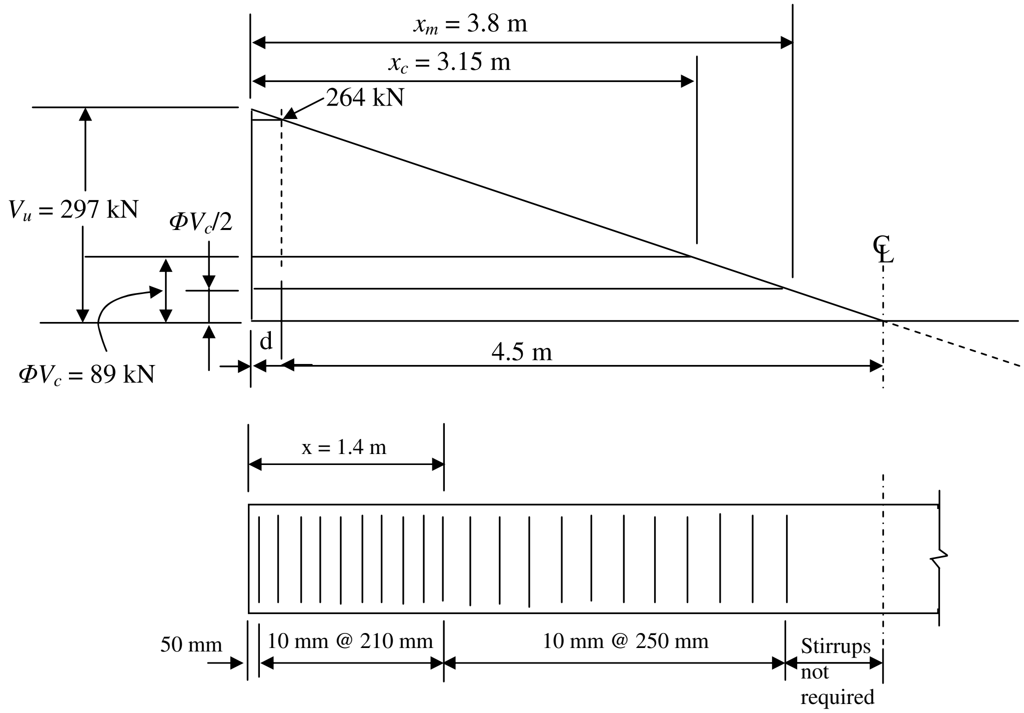

Fig. 20. Shear design of beam

The ACI code recommends ϕ as 0.75 for shear design and a value of 0.70 is suggested here based on the aforementioned analysis for safety-reduction factor in shear.

Rotation Capacity

The rotation capacity (θ) of a tensile plastic hinge is calculated with the help of Eq. (19) (Hassoun and Al-Manaseer 2008) as follows:

$$\theta = \frac{0.0035}{\lambda} - \frac{f_y}{E_s(1-\lambda)} \tag{19}$$

where λ = ratio of depth of NA (c) to effective depth and is given by Eq. (19) (Hassoun and Al-Manaseer 2008).

$$\lambda = \frac{c}{d} = \frac{\rho f_y}{0.72 f_c} \leq 0.5 \tag{20}$$

Fig. 19 illustrates variations in λ versus f_y at different ρ for a concrete strength of 30 MPa. The maximum value of $\lambda(\lambda_{\max})$ as determined using $f_y = 420$ MPa and $\rho_{\max}$ [Eq. (9)] is shown by a solid horizontal line in Fig. 19. It is noted in Fig. 19 that λ increases with both f_y and ρ, as can be expected from Eq. (20). This is an indication of decreased ductility of beam. At ρ values higher than 0.0112, the curves cross over $\lambda_{\max}$ line with increasing f_y. This is owing to the fact that at higher f_y values, NA depth increases and the section behaves as an overreinforced section. This reduces rotation capacity of the beam, compared with an underreinforced beam. Consequently, concrete strains are higher, which can result in brittle concrete failure (Fig. 12). It is evident in the aforementioned discussion that control over NA depth is vital to avoid this condition. As a result, a factor γ is suggested for calculating λ from Eq. (20), which can be written as Eq. (21), where γ is greater than 1

$$\lambda = \frac{\gamma c}{d} = \frac{\gamma \rho f_y}{0.72 f_c} \leq 0.5 \tag{21}$$

To determine the value of γ, the maximum percentage increase in λ beyond $f_y = 420$ MPa is determined for each ρ in Fig. 19. This procedure was repeated for $f_c = 20$–50 MPa. The calculated percentage was found to be independent of ρ and f_c which yielded a value of γ as 1.78. However, because such a high value of this factor will cause considerable constraint on the design, a value of 1.5 is suggested here. This factor γ will not cause any change in the moment capacity of the section as this is used only for the estimation of beam rotation capacity, which will be preceded by the flexure design.

Design Examples

To illustrate the design procedure based on recommendations given in the aforementioned sections, design examples have been taken from the existing literature. Two examples are included in the "Appendix" section to illustrate flexure and shear design along with beam-rotation capacity check. It is demonstrated in these examples that flow of design process is unaffected by the suggested changes as compared with the original ACI code design, while the reliability of design is enhanced considering the local bar properties.

Conclusions

The results of chemical and mechanical tests on the rebars used in RC construction in Pakistan are presented in this paper. Design implications for using these bars are investigated. Based on the results of the presented studies the following conclusions can be drawn.

- Both the hot-rolled deformed and cold-twisted ribbed bars satisfied the chemical composition requirements as given by the respective ASTM and British Standards.

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- A total of 13% of the tested hot-rolled deformed bars did not meet the yield strength requirements as given by ASTM standards whereas failure proportion of the bars to meet the tensile strength requirements was 19%. The standard deviation in the strength for most of the hot-rolled deformed bars was close to 70 MPa. Only 10-mm diameter bars showed large variations in their mechanical properties.
- A large number of cold-twisted ribbed bars (33%) had yield strength less than that recommended by BS 4449 (BSI 1997) for concrete reinforcement. The failure proportion of 10-mm diameter cold-twisted ribbed bars to satisfy the strength requirements was the highest and 60% of the bars did not meet the yield strength requirements. A total of 12% of the cold-twisted ribbed bars failed to meet the elongation requirements as given by BS 4449 (BSI 1997). The failure proportion for most of the bars was more than 10% on the individual bar level. The data of standard deviation in the strength of cold-twisted ribbed bars showed large variations, which indicate inconsistent mechanical properties of these bars.
- The effects on flexure and shear design and tensile hinge rotation capacity using the manufactured bar properties were investigated based on the provisions of ACI code. It was found that a section designed as underreinforced behaves as overreinforced section at higher f_y values. Relations are suggested for ρ_{max} [Eq. (12)] and S_{min} [Eq. (18)], respectively, for flexure and shear design of beams to avoid this condition. A factor of 1.5 is recommended to control the depth of NA in the determination of rotation at a plastic hinge. Further, Eq. (13) is suggested as an additional criterion for the bar yield strength requirements.
- A significant number of beam sections were analyzed to determine the mean and design strengths. These data were used to carry out a reliability analysis that assumes log-normal distribution of R/Q. The values of ϕ were calculated from the reliability analysis. These come out to be 0.80 and 0.60 for flexure design of underreinforced and overreinforced beams, respectively. For beam shear design, 0.70 can be used as capacity-reduction factor.

Appendix.

Example 1

This example is based on Example 16.7 in Hassoun and Al-Manaseer (2008), which demonstrates flexure design and rotation-capacity calculation procedures.

A beam of 7.5-m span length (L) is fixed at both ends and carries a uniform ultimate load (w_u) of 80 kN/m, and a concentrated factored load (P_u) of 215 kN. Design the beam using the limit state design procedure. Use $b = 350$ mm, $f_c = 21$ MPa, and $f_y = 420$ MPa.

Flexure Design

The factored moment (M_u) on the beam is equal to

$$M_u = \frac{w_u L^2}{16} + \frac{P_u L}{8} = 483 \text{ kN m}$$

ρ_{max} from Eq. (12) is 0.0093, which gives $R_u = 2.78$ MPa with ϕ taken as 0.8.

In this scenario, $R_u = \phi\rho f_y[1 - (\rho f_y/1.7 f_c)]$, $M_u = R_u bd^2$, $483 \times 10^6 = 2.78 \times 350 \times d^2$, $d = 704$ mm; $h = 704 + 63 = 767$ mm ≈ 770 mm; and area of steel $A_s = \rho bd = 0.0093 \times 350 \times 707 = 2{,}301 \text{mm}^2$.

Use five Number 25 bars in one row; A_s provided = 2,550 mm^2.

Rotation Capacity

$$a = \frac{A_s f_y}{0.85 f_c b} = 171 \text{ mm}$$

$$c = \frac{a}{0.85} = 201 \text{ mm} \quad \lambda = \frac{\gamma c}{d} = \frac{1.5 \times 201}{707} = 0.43 < 0.5 \quad \text{OK}$$

$$E_c = 4{,}730\sqrt{f_c} = 22.5 \text{ kN/mm}^2$$
$$E_s = 200 \text{ GPa}$$
$$n = \frac{E_s}{E_c} = 8.9$$
$$I_{cr} = 56.71 \times 10^8 \text{ mm}^4$$

The required rotation (θ_{required}) of plastic hinges at the fixed ends (A and B) of the beam is calculated using Eq. (22) (Hassoun and Al-Manaseer 2008)

$$\theta_{\text{required}} = \frac{L}{6E_c I}[2(M_A - M_{FA}) + (M_B - M_{FB})] \tag{22}$$

$$\text{Fixed-end moments } M_{FA} = M_{FB} = \frac{w_u L^2}{12} + \frac{P_u L}{8} = 576.6 \text{ kN m}$$

Therefore, θ_{required} comes out to be 0.0028 rad from Eq. (22).

The rotation capacity provided [Eqs. (19) and (21)] is equal to 0.0040 rad > 0.0028 rad OK.

Example 2

For shear design, Example 12.1 of PCA Notes on ACI 318-05 [Portland Cement Association 2005] is used here.

Determine the required size and spacing of vertical U stirrups for a 9-m span simply supported beam. Use $b = 325$ mm, $d = 500$ mm, $f_c = 21$ MPa, $f_y = 420$ MPa, and $w_u = 66$ kN/m.

Shear Design

Shear (V_u) at support = $66 \times 4.5 = 297$ kN (Fig. 20).

V_u at distance d from support = $297 - 66 \times 0.5 = 264$ kN

Concrete shear strength $\phi V_c = \phi 0.17\sqrt{f_c}bd = 89$ kN($\phi = 0.7$)

$$V_u = 264 \text{ kN} > \phi \qquad V_c = 89 \text{ kN}$$

Therefore, shear reinforcement is required

$$V_u - \phi V_c = 175 \text{ kN} < 0.67\sqrt{f_c}bd = 374 \text{ kN} \quad \text{OK}$$

Distance (x_c) from support beyond which minimum shear reinforcement is required = 3.15 m.

Distance (x_m) from support beyond which concrete can carry total shear force = 3.8 m.

Check minimum permissible spacing [Eq. (18)] assuming 10-mm bar ($A_v = 157$ mm^2).

$$S_{\min} = 1{,}065\left(\frac{A_v}{b\sqrt{f_c}}\right)$$

$$S_{\min} = 1{,}065 \times \frac{157}{325\sqrt{21}} = 112 \text{ mm}$$

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Spacing of 10-mm U stirrups at critical section ($V_u = 264$ kN)

$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c} = 131 \text{ mm} > 112 \text{ mm} \quad \text{OK}$$

Check maximum permissible spacing

$$S_{\max} \leq \frac{d}{2} = 250 \text{ mm} \quad \text{Governs}$$

$$S_{\max} \leq 600 \text{ mm}$$

Maximum stirrup spacing for minimum shear reinforcement

$$S \leq \frac{A_v f_y}{0.062\sqrt{f_c}b} = 910 \text{ mm} \quad S \leq \frac{A_v f_y}{0.35b} = 740 \text{ mm}$$

Distance (x) for providing stirrups at $S_{\max}$ (250 mm) = 1.4 m.

Acknowledgments

The authors wish to acknowledge the support provided by all the laboratory technical staff members and M/s Razaque Steels (Pvt) Ltd for conducting chemical analysis test of the bars.

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