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Comparative Study on Materials used in Various Codes for Design of RC and Steel Structures

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Abstract: Indian standard code of practice for plain and reinforced concrete IS 456 was last revised in 2000 after 22 years of the third revision. The major revision done in the new code is shift in design philosophy from working stress method to limit state method. The new code has not shown any significant changes in the concepts of structural design, material properties for high strength of concrete and steel, material grade limitation for seismic design, design of deep beam and walls, design of in-filled frames, design of beam column joint and provisions for effective length of frame members. The code does not clearly mention about mode of failures such as shear, flexure in structural members.

In the present paper a comprehensive literature review on the design strength of materials, stress stain curve for concrete, steel and confined concrete, partial safety factors and limitations / recommendations for usage of concrete grade and reinforcement steel grade in design provisions of Indian Standards, American Standards, European Standards, New Zealand Standards, Japanese Standards and from the latest available literature is done.

Indian standard code of practice for plain and reinforced concrete IS 456 was initially published in 1953 and subsequently it has been revised in 1957, 1964, 1978 and the latest revision was done in 2000(IS 456: 1953, 1957, 1964, 1978, 2000). Also the latest revision of IS 1893 came up after 18 years and it has many changes as compared to the earlier versions. In the latest revision of IS 1893, it is mentioned that the necessary adjunct for plain and reinforced concrete is IS 456 (IS 1893: 2002). It recommends the use of IS 4326 for earthquake resistant design and construction of buildings (IS 4326: 1993). Further, for the ductile detailing of reinforced concrete structures subjected to seismic forces, reference is given to IS 13920 (IS 13920: 1993). However, there is no discussion regarding the limitations/recommendations for the usage of concrete and steel grades for earthquake resistant Reinforced Concrete (RC) constructions. Influence of high grade concrete on material properties and stress block parameters is not mentioned in IS codes. Change in material properties with time for estimating the existing strength, effect of curing temperature on strength of concrete and tensile strength of concrete are some major points need be incorporated in design codes. The provisions of the above parameters in design codes such as American, European, New Zealand, Japanese and Indian Standards are studied in the present paper and also the importance of these parameters is discussed in detail.

American Standard Codes (IBC 2000, ACI 318)

For structural concrete, IBC and ACI specify that compressive strength, f'c shall not be less than 17 MPa (IBC, ACI-318). Unless otherwise specified, this strength is based on 28 days cube test. For design purpose maximum yield strength of 550 MPa is allowed for non pre-stressed reinforcement. For shear and torsion reinforcement the maximum fy that may be used in design is 420 MPa, except that f_y up to 550 MPa may be used for shear reinforcement meeting the requirements of ASTM A 497.

ACI provides empirical relation for modulus of elasticity based on the compressive strength and weight of concrete as:

$$E_c = W_c^{1.5} 0.043\sqrt{f_c'}$$

For Wc between 1500 and 2500 kg/m^3 For normal weight concrete, Ec shall be taken as:

$$E_c = 4700\sqrt{f_c'}$$

Ec is defined as the slope of the line drawn from a stress of zero to a compressive stress of 0.45 fc'.

ACI code specifies a nonlinear distribution of strain need to be considered for deep flexural members with overall depth to clear span ratios greater than 2/5 for continuous spans and 4/5 for simple spans. Maximum strain at extreme concrete compression fiber is 0.003. Concrete stress and strain distribution is assumed as rectangular, trapezoidal, parabolic, or shape that results in prediction of strength in agreement with results of comprehensive. Tensile strength of concrete in flexure is neglected in strength design.

Average stress of 0.85 fc' is used in equivalent rectangular stress block, with a rectangle of depth $\alpha = \beta_{1c}$, 1 of 0.85 is used for concrete compressive strength upto 30 MPa and 0.05 less for each 7 MPa increase in compressive strength with least value of 0.65.

For concrete in members resisting earthquake induced forces, compressive strength, f'c shall not be less than 20 MPa. And the maximum design compressive strength of lightweight aggregate concrete to be used in structural design calculations is limited to 35 MPa, primarily because of paucity of experimental and field data on the behavior of members made with lightweight aggregate concrete subjected to displacement reversals in the nonlinear range.

Reinforcement provided to resist earthquake induced flexural and axial forces in frame members and in wall boundary elements should comply with ASTM A 706M. ASTM A 615M Grades 300 and 420. Reinforcement is permitted in above members, if the actual yield strength based on mill tests does not exceed the specified yield strength by more than 120 MPa and their ratio not less than 1.25.

3. European Standard Codes (Eurocode 2, Eurocode 8)

Strength class corresponds to 28 day cylinder (5%) strength. According to EC8, the use of concrete class lower than C 16/20 (16 MPa) for DC "M" and C 20/25 (20 MPa) for DC "H" is not allowed. Here Eurocode discusses the lower grade of concrete; however, it did not mention any cut-off for higher grade.

DC "M" Ductility class "M" corresponds to structures designed, dimensioned and detailed according to specific earthquake resistant provisions, enabling the structure to enter well within the inelastic range under repeated reversed loading, without suffering brittle failures.

DC "H" Ductility class "H" corresponds to structures for which the design dimensioning and detailing provisions are such as to ensure in response to the seismic excitation, the development of chosen stable mechanisms associated with large dissipation of hysteretic energy.

Provisions for reinforcement steel are discussed says that except for closed stirrups or cross-ties, only ribbed bars are allowed as reinforcing steel in critical regions. The application of rules for design and detailing in Eurocode 2 are valid for yield strength range, f_{yk} = 400 to 600 MPa and the maximum actual yield stress f_y,max shall not exceed 1.3f_{yk}

Eurocode2 specifies empirical relations for the compressive strength and tensile strength of concrete based on age t and type of cement, temperature and curing conditions.

Modulus of elasticity E_{cm} is determined from stress strain curve, secant value between σ_c = 0 and 0.4f_{cm} is considered as E_{cm}. Code also specifies the effect of aggregate on the modulus of elasticity, for limestone and sandstone aggregates the value should be reduced by 10% and 30%, respectively. For basalt aggregates the value should be increased by 20%. Effect on E_{cm}, with time is also mentioned empirically. Modulus of elasticity

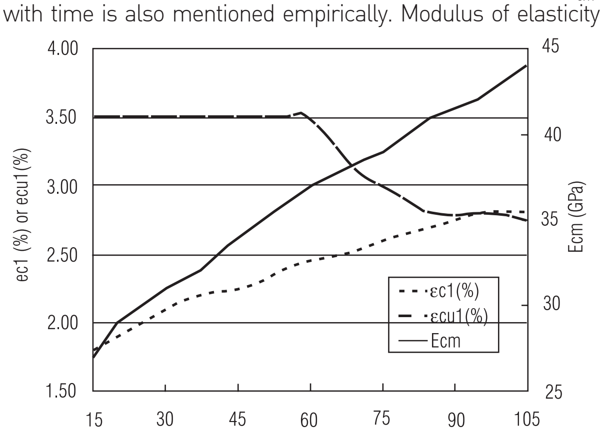


Figure 1: Change in modulus of elasticity and strain with grade of concrete

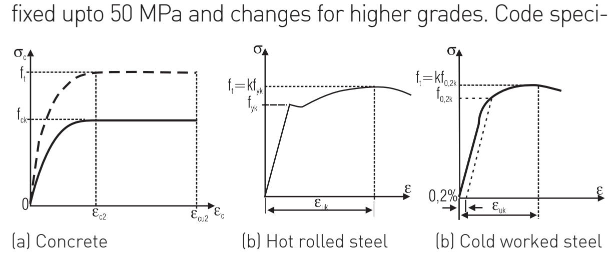
E_{cm}, compressive strain in concrete at peak stress (fc), εc1 and ultimate compressive strain in concrete, ε_{cu} are described in figure 1.

As the strength of the concrete increases, strain at peak stress εc1 increases gradually and modulus of elasticity increases by considerably. However, the ultimate compressive strain in concrete, ε_{cu} drops down from 0.0035 to around 0.0027. It shows that the higher strength concrete exhibits brittle nature after reaching peak stress. It is for this purpose mentioned in Cl 3.1.9 that the confinement of concrete results in a modification of effective stress-strain relationship: higher strength and higher critical strains are achieved. Confinement can be generated by adequately closed links or cross-ties, which reach the plastic condition due to lateral extension of the concrete.

Eurocode 8 applies to the design and construction of buildings and civil engineering works in seismic regions. It states two limit states, (i) Ultimate limit state and (ii) serviceability limit state. In addition to this, partial safety factors for materials for ultimate limit state γc = 1.5 and γs = 1.15 (EC8). It is also mentioned that lower values of γc, γs may be used if justified by measures reducing the uncertainty in the calculated resistance. In the Annex-A of Eurocode 2, it is mentioned that reduced partial safety factors, 1.4 and 1.1 may be considered for concrete and steel respectively.

For obtaining the design compressive and tensile strengths, characteristic compressive and tensile strengths are divided by partial safety factor γc and multiplied by coefficient taking into account of long term effects on compressive and tensile strength and of unfavorable effects resulting from the way the load applied, a_{cc} and a_{ct} respectively. The value of a_{cc} and a_{ct} is 1.0. In addition to this, it is mentioned that if the width of the compression zone decreases in the direction of extreme fibre, the value of design compressive strength f_{cd} should be reduced by factor ranging from 10% for concrete upto C5-/60 to 20% for concrete C90/105. For the design of cross sections, the Parabola-rectangle or Bi-linear stress-strain relationship may be used for concrete. The stress-strain curves for concrete, hot-rolled steel and cold-worked steel are shown in figure 2. Also the stress distribution diagram is shown in figure 3.

The factor λ, defining the effective height of the compression zone and the factor η, defining the effective strength will be fixed upto 50 MPa and changes for higher grades. Code speci-



(a) Concrete (b) Hot rolled steel (b) Cold worked steel

Figure 2: Stress strain curve for concrete and steel

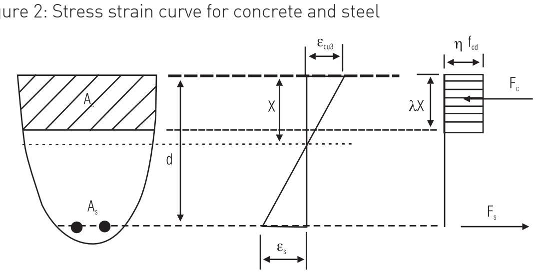


Figure 3: Rectangular stress distribution

fies the effect of confinement on stress strain curve of concrete; higher strength and higher critical strains are achieved (figure 4). The other basic material characteristics may be considered as unaffected for design.

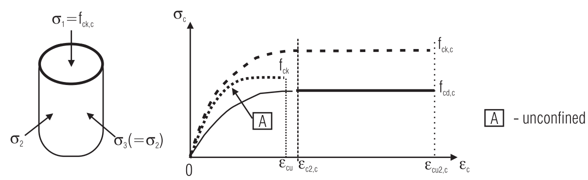


Figure 4: Stress strain relationship for concrete

New Zealand Standard Codes (NZS 4203, NZS 3101)

According to NZS 3101:Part 1:2006, concrete compressive strength fc' shall not be less than 25 MPa, and shall not exceed 100 MPa, without special study. It is also mentioned that for ductile elements and elements of limited ductility it should not exceed 70 MPa, without special study. The reason for limiting fc' to not greater than 70 MPa for ductile structures is that very high strength concrete members require considerable amount of confining reinforcement to ensure ductile behavior. Equations for the amount of confining reinforcement required for very high strength concrete columns are still being developed. However, there is evidence from theoretical moment-curvature analyses recently conducted by LiBing et al using stress-strain models for confined high strength concrete recently derived at the University of Canterbury, that equations for confining reinforcement may also be applicable to high strength concrete columns with fc' upto 100 MPa providing that the stress in the transverse reinforcement is assumed not to exceed 800 MPa (LiBing et al., 1994).

The lower characteristic yield strength of main reinforcement, f_y, used in design should not be greater than 500 MPa and for transverse should not be greater than 500 MPa for shear or 800 MPa for confinement.

NZS 3101 gives empirical relation for modulus of elasticity based on the compressive strength and weight of concrete as:

$$\left[3320\sqrt{f_c'}+6900\right]\left(\frac{\rho}{2300}\right)^{1.5} MPa$$

And for normal weight concrete: $\left[3320\sqrt{f_c'}+6900\right] MPa$

The design direct tensile strength for normal density concrete is specified as $0.36\sqrt{f_c'}$ in the absence of accurate data. Stress-strain curve for concrete can be taken as curvilinear defined by simplified equations or from suitable test data.

Reinforcing bars of Grade 500 shall be manufactured using either the micro alloy process or the in-line quenched and tempered process. Reinforcement bars of ductility Class E shall be used unless the conditions for use of Class N are satisfied. Ductility Class L reinforcement bars shall not be used. Class N ductility can be used where member is not subjected to seismic actions and the strain sustained at the ultimate limit state does not exceed 0.033 or where a member is subjected to seismic actions but the strain in the ultimate limit state does not exceed a value of 0.025.

The maximum strain at the extreme concrete compression is 0.003. Tensile strength of concrete is neglected for flexural design. Strain distribution in reinforcement and concrete vary linearly through the depth of the member, use of strut-and-tie model is specified for deep beams.

Concrete stress of α1fc' is used in equivalent rectangular stress block, with a a straight line located parallel to the neutral axis at a distance a=β1c from the fiber of maximum compressive strain.

Value of α1 is 0.85 used for concrete compressive strength up to 55 MPa and for above values α1 is taken as α_1= 0.85-0.004(f_c' - 55) but not less than 0.75.

Value of β1 is 0.85 used for concrete compressive strength up to 30 MPa and for above values α1 is taken as α_1= 0.85-0.008(f_c' - 30) but not less than 0.65.

It is also mentioned that structural light-weight concrete shall not be used in structures designed for a structural ductility factor greater than 1.25. To date there has been no satisfactory verification of adequate inelastic performance of members incorporating structural light-weight concrete at high ductilities, either in New Zealand or overseas.

For concrete the ultimate strength method is used to calculate member strengths for all actions. Strength reduction factors are: (a) 1.00 for anchorage of reinforcement and for capacity design, (b) 0.85 for flexure with or without axial load, (c) 0.75 for shear and tension, (d) 0.65 for bearing and (e) 0.6 for flexure in plain concrete. And for steel, the strength reduction factors are (a) 0.9 for bending axial load and combinations, (b) 0.8 for bolts or pins in shear or tension, (c) 0.9 for ply in bearing and (d) 0.6, 0.8 or 0.9 for welds depending on type.

Japanese Standard Codes

Basic design policy of JSCE Standards is as follows; Working stress design for gravity loading limits the allowable stress of concrete to be 1/3 of the specified concrete strength, and that of steel to be approximately 2/3 of the specified yield strength. On the other hand, the working stress design for earthquake loading limits the allowable stress of concrete to be 2/3 of the specified concrete strength, and that of steel to be the yield strength. The ultimate strength of members is used in the limit analysis procedure to evaluate the ultimate lateral load resistance of buildings. No strength reduction factors are used in design. Two phases design procedures are used for moderate and severe earthquake motions. The working stress design procedure is used for gravity loading design and for earthquake resistant design against moderate earthquake motions, while the limit analysis design procedure is sued to ensure the minimum resistance against severe earthquake motions. Load factors are not used in working stress design, but the amplitudes of live loads are varied for floor slabs, frame members, and for inertia mass.

AIJ Structural Design guidelines for reinforced concrete buildings (1994)

Types of concrete by aggregates shall be the normal weight concrete with the design nominal strength (Fc) between 210 kgf/cm^2 and 360 kgf/cm^2. The use of light weight aggregate concrete is excluded because the reliability of the strength and ductility evaluation has not been established due to lack of test data.

Indian Standard Codes (IS 13920, IS 456)

For concrete, Table 2 of IS 456 specifies the characteristic compressive strength for different grades of concrete. They are categorized as ordinary concrete (M10 ~ M20), standard concrete (M25 ~ M55) and high strength concrete (M60 ~ M80). It is also mentioned in the foot note that above M55 grade the design parameters given in the code are not applicable. However, it is established that even for concretes of grades below M 55, there could be a drop of upto 17 percent in the ultimate strain of concrete from the code specified value of 0.00355 (Murty, 2001). Here code needs to specify the values of ultimate strains for different grades or it should give recommendations for the grades of concretes for in seismic environment same as it is mentioned that minimum grade of concrete for plain and reinforced concrete for different exposure conditions shall be as per Table 5. According to this, the minimum grade for reinforced concrete shall be M20 (IS 456:2000). The limiting strain of concrete in compression is 0.0035.The modulus of elasticity of concrete can be estimated by $E_c = 5000\sqrt{f_{ck}}$.

Design values for loads are obtained by multiplying the characteristic by load factor, γ_f.

1) 1.5 (DL + IL)
2) 1.2 (DL + IL ± EL)
3) 1.5 (DL ± EL)
4) 0.9 DL ± 1.5 EL

Design strength of the materials is taken as the characteristic values divided by partial safety factors appropriate to the material and the limit state being considered. The clause.36.4.2 says that when assessing the strength of a structure of structural member for the limit state of collapse, the values of partial safety factor, γm should be taken as 1.5 for concrete and 1.15 for steel. For design purpose, the compressive strength of concrete in the structure shall be assumed to 0.67 times characteristic strength. The partial safety factor of 1.5 shall be applied in addition to this. Maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending. In limit state of collapse maximum strain of concrete is taken as 0.002.

Discussion

Performance of the RC structure during earthquake mainly depends on its ability to deform without loosing strength. This criterion is fairly taken into account in all the codes discussed above. Now let us look at comparison of the provisions of different countries for concrete and reinforcement steel.

Concrete

Basis of strength of concrete fck shall be based on 28 day test unless otherwise specified. This is same in all the codes. Except that some codes take cube strength and some use cylinders. Table 2 gives the provisions of different countries for lower and upper bound values for concrete grades. From the table, it can be clearly seen that except New Zealand, other countries have not given any recommendations for both minimum and maximum values of concrete strength in seismic environment. In addition to this, NZS 3101 also gives recommendations for concrete structures in non-seismic areas. Eurocode 8 also mentions the lower strengths, which shall not be less than 16 MPa for lower ductility class and 20 MPa for middle and upper ductility classes. Provisions given by IS 456 are somewhat closely matches with that of New Zealand. However, IS 456 mentioned that design parameters listed in code does not apply for strengths more than M55 concrete.

In general, concrete with compressive strength greater than 50 MPa is generally classified as high strength concrete. Such a concrete possesses highly improved performance characteristics, and it is due to these improved characteristics that high-strength concrete provides the most cost-efficient solution to many structural design problems and it may be used effectively in many places. In order to take full advantage of its improved performance characteristics, the properties and behavior of high-strength concrete need to be defined and quantified. Although research into these areas is progressing at different places, very little information is available on concrete produced in India.

Typical stress-strain curve of concrete is shown in the figure (Ahmad et al., 1985). The peak strength usually is reached around strain 0.002. From the figure 5, it can be noted that as the compression strength f'c increases, the strain at peak stress increases. However, it is clearly seen that the ultimate compressive strain is smaller and crushing strength decreases for higher grade concretes. This apparent brittleness in high-strength concrete is of serious concern and must be considered when ductility requirements result in high concrete compression strains.

S. No	Country Standards	Modulus of Elasticity	Stress block parameter for high grade concrete	Strain Distribution (Deep Beams)	Tensile Strength of Concrete (Flexure Design)	Max Concrete strain	Curing Effect on concrete strength	Confined Concrete Stress Strain curve
1	American Standards	$W_c^{1.5}0.043\sqrt{f_c^1}$	Changes	Non Linear	Not Considered	0.003	Not Mentioned	Not Mentioned
2	European Standards	Eurocode 8	Changes	Non Linear	Not Considered	0.0035	Mentioned	Mentioned
3	New Zealand Standards	$3320\sqrt{f_c'} + 6900\Big]\left(\frac{\rho}{2300}\right)^{1.5}$	Strut and Tie model to be used	Non Linear	Not Considered	0.003	Not Mentioned	Not Mentioned
4	Japanese Standard	Not Available	Not Available	Not Available	Not Available	Not Available	Not Available	Not Available
5	Indian Standards	$5000\sqrt{fck}$	Not Considered	Not Considered	Not Considered	0.0035	Not Mentioned	Not Mentioned

Table 1: Summary of parameters in different code

S. No	Country Standards		Provisions
1	American Standards	IBC 2000	Refers ACI 318
		ACI 318	Min = 17 MPa , Max = No limits
2	European Standards	Eurocode 8	Min = 16 MPa (DC "L"), 20 MPa (DC "M" and DC "H", Max = Not mentioned
		Eurocode 2	Min = 12 MPa, Max = 90 MPa
3	New Zealand Standards	NZS 4203	Min = 20 MPa, Max = 70 MPa
		NZS 3101	Min = 17.5 MPa, Max = 100 MPa
4	Japanese Standard	JSCE	Note mentioned
		AIJ	Min = 21 MPa, Max = 36 MPa ----check this
5	Indian Standards	IS 1893	Refers to IS 456
		IS 456	Min = 20 MPa, Max = 80 MPa

Table 2: Provisions for grade of concrete used in different countries

High strength concrete requires adequate confinement to exhibit ductile behavior. In many cases the ultimate compression strain of unconfined concrete is inadequate to allow the structure to achieve the design level of ductility without extensive spalling of the cover concrete. Unless adequate transverse reinforcement is provided to confine the compressed concrete within the core region and to prevent buckling of the longitudinal compression reinforcement, failure may occur. Particularly susceptible are potential plastic hinge regions in members that support significant axial load, such as column at the base of building frames, where inelastic deformations must occur to develop a full hinging mechanism, even when the design is based on weak beam/strong column philosophy.

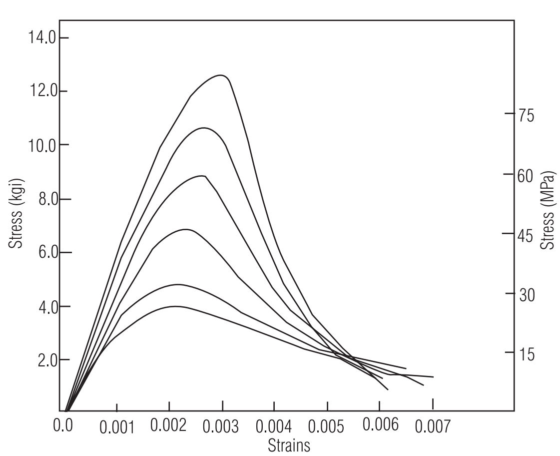


Figure 5: Stress-strain relationship of concrete with different strengths

Steel

Table 3 describes the provisions of reinforcement steel used in different countries. Here also New Zealand gave clear provisions for minimum and maximum steel grades in seismic environment. In addition to this, it also gave recommendations for non-seismic areas. Eurocode and American code puts restrictions that only ribbed bars are permitted. Further, Eurocode gave elaborated rules for seismic areas. Indian standard code mentions the use of three classes of steel i.e., 250 MPa, 415 MPa and 500 MPa. However, it did not give any discussion on the provisions for earthquake resistant structures.

The desirable characteristics of reinforcing steel are (i) adequate minimum yield strength, (ii) a long yield plateau, (iii) gradual strain hardening and (iv) low variability of actual yield strength from the specified nominal value.

i. Adequate minimum yield stress (or 0.2) percent of proof stress) may be ensured by specifying steel to an appropriate standard, such as BS 4449, or BS 4461, ASTM A615 or ASTM A706.
ii. Characteristic strength not greater than 415 N/mm^2 are not recommended in some earthquake areas e.g., California and New Zealand. This is because higher strengths generally imply decreased ductility (a shorter yield plateau) but where adequate ductility is proven by tests, somewhat higher strengths may be used where regulations permit (Dowrick). It is possible without undue effort and desirable from structural viewpoint, for steel manufacturers to produce reinforcing steel with a yield strength of 400 MPa or 500 MPa while retaining the ductility of lower strength reinforcement. Already demonstrated by New Zealand producers.
iii. If the reinforcing steel exhibits early and rapid strain hardening, the steel stress at a section with high ductility may exceed the yield stress by an excessive margin. In California it is recommended that the ultimate tensile stress should not be less than 1.33 times the actual yield stress of the bar.
iv. Similarly, if steel for a specified grade of reinforcement is subjected to considerable variation in yield strength, the actual flexural strength of a plastic hinge may vary greatly exceed the nominal specified value.

In both (ii) and (iv) the result will be a need to adopt the high over strength factors to protect against shear failures or unexpected flexural hinging. ACI tries to exert some control on the variability of steel to ASTM A615 by requiring that actual yield stress should not exceed the minimum specified yield stress (characteristic strength) by more than 124 N/mm^2. This control in the scatter of yield values is essentially a compromise with manufacturing economy.

Most basic measure of ductility of reinforcement is its elongation at failure. Steels with good elongation behavior are more likely to perform will in other tests of ductility. Figure 4 shows the typical stress-strain diagram of commonly available steel. Behavior is characterized by an initial linearly elastic portion of

S. No	Country Standards		Provisions
1	American Standards	IBC 2000	Refers ACI 318
		ACI 318	Min = 420 MPa , Max = 550 MPa
2	European Standards	Eurocode 8	Only ribbed bars (refer Table 1)
		Eurocode 2	Min = 400 MPa, Max = 600 MPa (Refer Table 2)
3	New Zealand Standards	NZS 4203	Min = 250 MPa, Max = 450 MPa
		NZS 3101	Min = 300 MPa, Max = 500 MPa
4	Japanese Standard	JSCE	Not available
		AIJ	Not available
5	Indian Standards	IS 1893	Refers to IS 456
		IS 456	Min = 250 MPa, Max = 500 MPa

Table 3: Provisions for grade of reinforcement steel used in different countries

the stress-strain relationship with modulus Es=200 GPa. Followed by a yield plateau of variable length and a subsequent region of strain hardening. After max stress is reached typically at about fsu=1.5fy strain softening occurs, with deformation concentrating at a localized weak spot. Reinforcements complying with BS 4449, BS 4461, ASTM A706, and ASTM A615 have moderate to good elongation values.

The percentage of elongation is the measure of ductility of the steel material. Fe250 has 20%, Fe415 has 14.5% and Fe500 has only 12% that indicates that, Fe500 is comparatively less ductile material, giving poor ductility (ISSE). Therefore, Fe500 reinforcement will be undesirable during reversal of stresses and plastic hinge formation stage during earthquake. Ductility decreases with increase in steel yield strength. The higher yield strength steel produces high values of shear and bond forces while designing for the capacity design and are difficult to provide for in reasonable size of cross sections of concrete beams and columns.

Conclusions

Basic aim for the design of earthquake resistant structures is to ensure that the structure exhibits inelastic behavior with loosing strength. The main components for ensuring ductile behavior are concrete and reinforcement steel. Recommendations of different codes in this regard are reviewed and found that recommended concrete strengths are between 20 kN/m^2 and the upper bound may be fixed at M50. And regarding steel grades, the most recommended grades are from 420 kN/mm^2 upto 500 kN/mm^2. The use of high strength concrete and reinforcement steel in building constructions has many advantages such as, increased strength of structures, reduced cross-sections, more durable material and therefore substantial savings. However, due to limited amount of experimental research and to the uncertainty inherent in the prediction of failure of the structural elements under cyclic loading, the use of high strength concrete and steel in seismic risk areas needs extra caution in order to ensure adequate ductile behavior.

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