Bending resistance of high performance concrete elements

D. Mestrovic[1] & L. Miculinic[2]
[1]*Faculty of Civil Engineering, University of Zagreb, Croatia*
[2]*Faculty of Civil Engineering, University of Rijeka, Croatia*

Abstract

Our previous research was focused on reactive powder concrete (RPC). In this article high performance concrete (HPC) will be analyzed. With high performance concrete both load bearing capacity and esthetics are combined to achieve superior structure. Due to lesser dimensions of walls and columns, space in high rise buildings is better utilized. With the combination of steel with high yield strength and ductility, difficulties with dense longitudinal and shear reinforcement are avoided. In this article current knowledge about the properties, technology and composition of high strength concrete is presented. Important parameters needed for the analysis of load-bearing (ultimate limit state), ductile and serviceable (serviceability limit state) reinforced concrete elements subjected to bending, with or without longitudinal force, are considered. These parameters are: design of compressive stress diagram, ductility, deflection, and crack width restraint. The impact of tensile, compression and shear reinforcement are analyzed. The results are compared with values for normal strength concrete. The use of high strength concrete is recommended.

Keywords: high performance concrete, bending resistance, ductility.

1 Introduction

Concrete with compression strength of more than 60 N/mm^2 is called high performance concrete (HPC). The first high performance concrete was made in 1960. Today, concrete classes C55/57 to C110/115 (according to EC2 regulations) are called high performance concretes. Concretes with higher compression strength are called ultra high performance concretes. It must be noted that high performance and ultra high performance concretes have

WIT Transactions on The Built Environment, Vol 97, © 2008 WIT Press
www.witpress.com, ISSN 1743-3509 (on-line)
doi:10.2495/HPSM080101

durability enhancements that decrease maintenance costs and lengthen the service life of a structure. This is vital for structures in aggressive environments (sea bridges etc). The choice of ingredients, production, transportation and finally curing for this concrete is very demanding and at the same time this is more important than for normal concretes. Buildings with high performance concretes are becoming more and more popular, especially in seismically active regions. Using high performance concrete results in less material being used, effectively meaning that the structure has smaller mass and as a result of this, seismic forces are lower. These structures can be made ductile; they have the capacity to dissipate energy (plastic deformation of hinges), which also contributes to economical aspects. Ductile structures can be calculated by the linear elastic theory and by the linear plastic theory, which means that distribution of forces is more uniform, material is better utilised and less steel is used. A very important advantage of ductile structures is the avoidance of brittle failure of elements and the whole structure. Very often, structures made of high performance concretes have big spans and are designed for huge loads. A high ratio of live load/selfweight combined with esthetical properties is also the main characteristic of high performance concrete. In combination with high yield strength steel, the ductility problems due to dense longitudinal and shear reinforcement are avoided. High performance concretes are also used for bridges where bearing capacity and durability are very important. Bridge columns made of this material have a smaller cross section, which can be very important for free profile under bridges. Disadvantages of high performance concrete are: loss of ductility, which is manly a problem for higher classes and separation of the protective layer due to reinforcement (the area close to the plastic hinges). Due to the decreased ductility, codes in some countries have adopted their calculations for shear reinforcement (USA), on the other hand in some countries (Canada), where seismic activity is high, use of high performance concrete is restricted. New experimental results suggest that the ductility limit has shifted from 55 N/mm^2 (as suggested earlier) to 75 N/mm^2. Separation of the protective layer can also be avoided using steel fibres (micro reinforced concrete). Steel fibres bond concrete cross sections very effectively and help in the forming of plastic hinges. In this article current knowledge about the properties, technology and composition of high performance (strength) concrete is presented. Important parameters needed for the analysis of load-bearing (ultimate limit state), ductile and serviceable (serviceability limit state) reinforced concrete elements subjected to bending, with or without longitudinal force, are considered.

2 General properties and technology

When making high performance concrete and choosing ingredients, special care must be taken. Concrete ingredients are cement, aggregate, water, chemical and mineral additives. The ratio of ingredients depends upon the desired properties and type of structure. The main problem is often compatibility between cement and additives, which is typical for high performance concretes. The methodology for testing compatibility between ingredients is not standardized, so assessment depends fully upon probation mixtures. High performance concretes have a large amount of cement (>400 kg/m^3) and a lower water/cement ratio. The grain size distribution is characterized by a dominant smaller grain size and a higher proportion of smaller fractions. Superplasticizer is primarily used to decrease the participation ratio of water in the concrete mixture. Additional additives are selected on the basis of the requirements of the particular application (strength, stiffness, freeze-thaw durability, or resistance to chemical attack). One of most efficient mineral additives is silica fume (silica fume is pozzolanic additive). The brittleness of concrete rises with compression strength. This can be partially avoided using fibres (as shown in figure 1). The improvement effect strongly depends upon the amount and type of fibres, friction between particles and the quality of the cement matrix. For improvement of ductility and other properties (tensile strength, crack limitation and dynamic response) steel fibres are usually used. Figure 2 represents dependence upon the water/cement ratio and compression strength that is often used when designing the concrete mixture. It is evident that for the same water/cement ratio a different compression strength can be achieved. This is primarily because of mineral and chemical additives. Production processes, transport, build in, and finally curing are much more complex than with normal concretes, so special care must be taken. One of the most devastating effects when dealing with high performance concrete is autogenic shrinking. Autogenic shrinking is the result of a lesser volume of hydration products than materials that enter process. Autogenic shrinking begins immediately after the hydration finishes. Compared to normal concrete, high performance concrete is a homogeneous material. Most rules and equations regulating normal concretes can't be used for high performance concrete (this is especially important for equations concerning bending strength).

For the relation between compressive (f_c) and bending strength (f_r), the following equations can be used (see eqns. (1) and (2)).

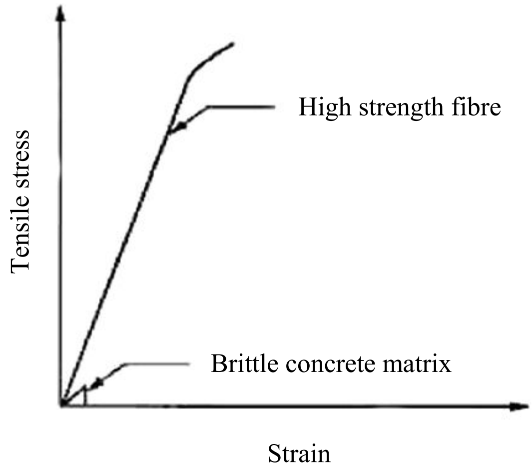

Figure 1: Stress-strain curves for concrete matrix and fibres.

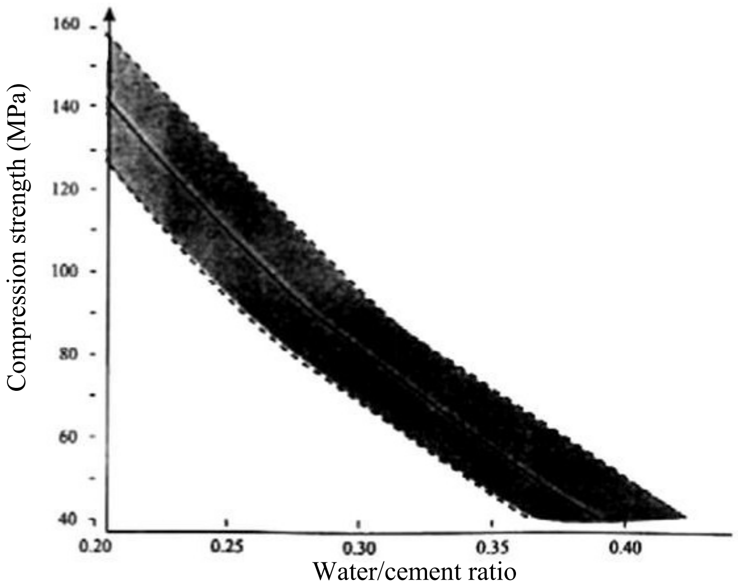

Figure 2: Water/cement ratio and compression strength after 28 days.

$$f_r = 0.94 \cdot f_c^{\frac{1}{2}} \quad \left[N/mm^2\right] \tag{1}$$

$$f_r = 0.23 + 0.12 \cdot f_c - 2.18 \cdot 10^{-4} \cdot f_c^{\ 2} \quad \left[N/mm^2\right] \tag{2}$$

The relation between the modulus of elasticity (E_c) and the compressive strength of high performance concretes is given in eqns. (3) and (4).

$$E_c = 10 \cdot \sqrt[3]{f_c + 8} \quad \left[kN/mm^2\right] \tag{3}$$

$$E_c = 5 \cdot \sqrt{f_c} \quad \left[kN/mm^2\right] \tag{4}$$

3 Parameters for design

The mathematical relationship for strain-stress is given by Thorebfeld, Tomaszewitcz and Jensen [10] in eqn. (5). σ_c is concrete stress, f_{ck} is the characteristic compressive strength of concrete, ε_c is strain caused by stress σ_c and ε_{cu} is the limit strain of concrete (corresponds to f_{ck})

$$\frac{\sigma_c}{f_{ck}} = -\frac{\varepsilon_c}{\varepsilon_{cu}} \cdot \frac{n}{n - 1 + (\varepsilon_c / \varepsilon_{cu})^{n \cdot k}} \tag{5}$$

Values of other marks are given in eqns. (6)–(8).

$$k=1 \text{ for } \varepsilon_c/\varepsilon_{cu}<1 \tag{6}$$

$$k=0.67+f_{ck}/62 \text{ for } \varepsilon_c/\varepsilon_{cu}>1 \tag{7}$$

$$n=0.8+f_{ck}/17 \tag{8}$$

Limit strain is given in eqn. (9). E_c is the modulus of elasticity and is given in eqn. (10).

$$\varepsilon_{cu} = \frac{f_{ck}}{E_c} \cdot \frac{n}{n-1} \tag{9}$$

$$E_c = 3320 \cdot \sqrt{f_{ck}} + 6900 \tag{10}$$

For design according to ENV 1992, designed compressive strength f_{cd} must be calculated. Mansur et al [2] suggest a modified stress-strain diagram, which incorporates the influence of shear reinforcement and steel fibres. Eqn (11) is given for the growing branch and eqn. (12) for the declining branch. Coefficient β is given in eqn. (13). k_1 and k_2 are correction factors, E_{it} is the initial modulus of elasticity, f_0 is peek stress and ε_o is the deformation that corresponds to peek stress.

$$\sigma_c = f_0 \cdot \left\{ \frac{\beta \cdot (\varepsilon_c / \varepsilon_0)}{\beta - 1 + (\varepsilon_c / \varepsilon_0)^{\beta}} \right\} \tag{11}$$

$$\sigma_c = f_0 \frac{k_1 \cdot \beta \cdot (\varepsilon_c / \varepsilon_0)}{k_1 \cdot \beta - 1 + (\varepsilon_c / \varepsilon_0)^{k_2 \cdot \beta}} \tag{12}$$

$$\beta = \frac{1}{1 - f_o /(\varepsilon_0 / E_{it})} \tag{13}$$

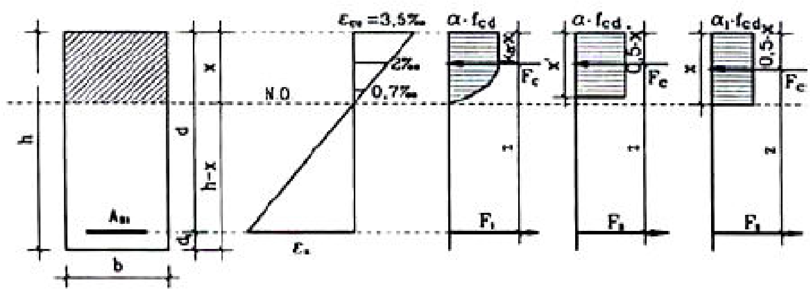

Figure 3: Stress-strain according to EC 2 and CEB/FIB.

According to Eurocode regulations, for the stress-strain diagram the second order parabola is used. This is shown in figure 3. For compressive strength, a linear or parabolic diagram is assumed (this is experimentally proven for concrete of maximal compressive strength of 60 N/mm^2). Coefficient α is assumed to be 0.85 for all cross section shapes, except for the case where the width of the compressive area shrinks from the neutral axis towards the compressive edge (α=0.8 in that case). For higher strength concrete eqn. (14) can be used.

$$\alpha = 0.6 + \frac{10}{f_{ck} \cdot 0.85} \tag{14}$$

4 Ductility

A very important property of reinforced-concrete structures is ductility or the ability to resist deformations very close to the limit load without affecting bearing capacity. High performance concrete is a brittle material, so ductility demand is crucial. Most building codes have analytical equations for classification of ductility. Experimental results show that a higher class of concrete subjected to bending and axial force (or without axial force) has higher ductility. This result must be taken very carefully as this is valid only for concrete with compression strength up to 80 N/mm^2. Higher classes are very sensitive and ductility decreases very quickly. The ductility of elements subjected to bending is usually classified by the coefficient of ductility (local ductility) defined by the ratio of limit curvature (Φ_u) and curvature when tensile steel reaches yield strength (Φ_y). Other ways to measure (assess) ductility are given in eqn. (15). δ_u is maximal deflection in the mid span of the beam under 85% failure load and δ_y is maximal deflection when the steel reaches yield strength.

$$\mu_d = \delta_u / \delta_y \tag{15}$$

Many building codes, such as the American ACI, limit the coefficient of tensile reinforcement. This is done to achieve better ductility. The coefficient of tensile reinforcement is limited as given in eqn. (16) for standard design situations and eqn. (17) for situations where redistribution of the bending moment is expected. ρbs is the balanced coefficient of reinforcement and is defined in eqn. (18).

$$\rho < 0.75 \cdot \rho_{bs} \tag{16}$$

$$\rho < 0.5 \cdot \rho_{bs} \tag{17}$$

$$\rho_{bs} = \frac{0{,}85 \cdot \alpha_v \cdot f_{ck}}{f_{yk}} \cdot \frac{\varepsilon_{cu} \cdot E_s}{-\varepsilon_{cu} \cdot E_s + f_{yk}} \tag{18}$$

European regulations [7] limit the height of the compressive area. For higher ratios the use of double reinforcement is proposed. For elements subjected to bending, compressive reinforcement resists in compressive stresses and thereby contributes to ductility. Shear reinforcement mainly contributes to shear stresses, but it also entangles the compressive area and contributes towards higher compressive strength and ductility. Some authors [8] have concluded that shear reinforcement is crucial for ductility. For this reason, European regulations [9] limit maximal spacing between shear reinforcement.

5 Serviceability

The first crack caused by the bending of concrete element happens when the bending moment is equal to the cracking moment as defined in eqn. (19). f_r is tensile strength, I_g is the moment of inertia, y_t is the distance from the tensile border to the neutral axis.

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} \tag{19}$$

In eqn. (19) the tensile strength is most problematic, for which only approximation is available. Many authors were investigating this problem, but until now no solution is given. If we compare calculated vs. experimental values for the crack moment, variations of almost 40% occur. Theoretical models prove that if shrinkage of concrete is incorporated, the variation is much smaller, but this immensely complicates the calculations. For that purpose, effective modulus of elasticity is ($E_{eff(t,\tau)}$) proposed as in eqn. (20). $\Phi(t,\tau_o)$ is creep coefficient in time t for concrete loaded at time τ_o, $\lambda(t,t_o)$ is the aging coefficient (a value of 0.8 can be assumed).

$$E_{eff}(t,\tau_o) = \frac{E_c(\tau_o)}{1+\chi(t,t_o)\cdot\Phi(t,t_o)} \tag{20}$$

6 Conclusion

In this article an overview of the properties, technology and composition of high strength concrete is given. Important parameters (equations) for design of this concrete are given. It is shown that for this kind of material different equations are applicable. A proposal for a compressive design diagram is given. The ductility of elements is investigated and it is shown that ductility increases for compressive strengths between 40 N/mm^2 to 70 (80) N/mm^2 and than rapidly decreases. The main factors that increase ductility (compressive strength, tensile and shear reinforcement) are given and analyzed. It must be concluded that the future of high performance (strength) concrete is very bright, but additional experiments are needed to successfully incorporate specific properties of these concretes.

References

[1] Skazlic, M., Tomicic, I., Reinforced concrete elements made of high performance concrete subjected to bending, Gradevinar, pp. 631–640, Zagreb, 2006.
[2] Mansur, M.A, Chin, M. S., Wee, T. H., Flexural behavior of high strength concrete beams, ACI Structural Journal, Vol. 94, No.6, 1997.
[3] Radic, J., Concrete structures – manual, Croatian University Press, 2006.
[4] Tomicic, I., Concrete structures, Skolska Knjiga, Zagreb, 1988.

[5] Necevska Svatanovska, G., High strength concrete – Macedonian research, Gradevinar, pp. 531–537, Zagreb, 2004.
[6] Edward Nawy, G., Fundamentals of high-performance concrete, John Wiley & Sons: New York, 2001
[7] Eurocode 2: Design of concrete structures. Part 1-1: Basic rules and regulations for buildings (HRN ENV 1992-1-1), Zagreb, 2004.
[8] Rashid, M.A., Mansur, M.A., Reinforced High-Strength Concrete Beams in Flexure, ACI Structural Journal, Vol. 102, No. 3, pp. 462–471, 2005
[9] Eurocode 8: Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions Rules for Buildings, CEN, Brussels, 2003.
[10] Collins, M. P., Mitchell, D., McGregor, J. G., Structural Design Considerations for High-Strength Concrete, Concrete International, Vol. 15, No. 5, pp. 27–34, 1993.