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AXIAL LOAD AND MOMENT INTERACTION CHARTS FORHIGH PERFOMANCE CONCRETE

GRÁFICOS DE INTERACCIÓN CARGA AXIAL-MOMENTO PARA HORMIGONES DE ALTA RESISTENCIA

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ABSTRACT

This paper presents non-dimensional design charts for High Performance Concrete (HPC). The more realistic stress-strain curves used were proposed by CEB - Bulletin d'Information 228. The development is made applying "Silva Jr.'s Method". The strain failure domains of HPC were defined considering the ultimate concrete strain and the strain that corresponds to the maximum stress. The resultant compression force in the equilibrium equations was obtained by integration of the area under the more realistic stress-strain curve. Non-dimensional interactive charts procedures and practical examples are obtained by using an algebraic and symbolic computer program.

RESUMEN

Este trabajo presenta gráficos adimensionales de dimensionamiento para Hormigones de Alta Resistencia (HAR). Las curvas tensión-deformación más realistas utilizadas fueron propuestos por el CEB - Bulletin d'Information 228. El procedimiento es aplicando el modelo del "Método de Silva Jr.". Los dominios de falla de las deformaciones específicas del HAR fueron definidos en función de las deformaciones últimas y deformaciones correspondientes a las tensiones máximas. La resultante de compresión resultante utilizada en las ecuaciones de equilibrio se obtuvo mediante la integración numérica del área bajo la curva de tensión-deformación más realista. Se obtuvieron procedimientos interactivos adimensionales y ejemplos prácticos mediante el uso de un programa algebraico y simbólico.

Keywords: High Performance Concrete, Axial-Moment Design Charts, Strees-Strain Curves.
Palabras Clave: Hormigón de Alta Resistencia, Gráficos de Dimensionamiento Fuerza Axial-Momento, Curvas Tensión-Deformación

1. INTRODUCTION

High Performance Concrete (HPC) is a technological evolution of conventional concrete, growing in use in buildings, structural repairs, bridges, precast concrete, dams, etc. HPC is obtained by mixing cement, clean and selected aggregates, minerals admixture, chemical admixtures and a lower water/cement ratio (W/C), giving high resistance and durability to concrete. The addition of chemical admixture (superplasticizers) reduces the W/C, improves the mechanical resistance of the concrete, reduces the amount and the diameter of pores, increases the density of the matrix and increases the resistance to external agents' attack. The mineral admixture (silica fume, fly ash, blast-furnace slag, rice-husk ash, etc) has two main purposes: to fill part of the pores and to combine chemically with $Ca(OH)_2$ forming the C–S–H gel (stronger and more resistant). Therefore, HPC has advantageous applications in current times, but it needs a more rigorous control in the selection of its components, in the admixture, transport, placement, consolidation and mainly in the cure of the final product.

Some international standards for concrete structures like CEB-FIP MC90[1], FIP Recommendations-99 [2] and CSA-CAN3-94[3] have already incorporated this material. The ACI 318-99 [4] allows its use.

Figure 1 displays two typical design stress-strain curves[1], one for a conventional concrete with characteristic compressive strength f_{ck}=20 MPa (CEB-Bulletin d'Information N^o 228 equations [5] with a conventional concrete safety coefficient γ_c=1.5) and the other for a HPC with f_{ck}=80 MPa concrete (CEB-Bulletin d'Information N^o 228 equations with a HPC concrete safety coefficient γ_c=[1.5 / (1.1 - f_{ck} / 500)]. The values of the maximum compressive strain $\square_{clim}$ decreases while the concrete resistance increases. For example, for a f_{ck}=80 MPa, the maximum strain of the compressed concrete is 0.0029 (Equation 11), smaller than the value usually used of 0.0035 for a conventional concrete.

[1] In this paper, design stress-strain curves are realistic stress-strain curves considering the concrete safety coefficient and the unfavorable effect of long-term loads in its equations.

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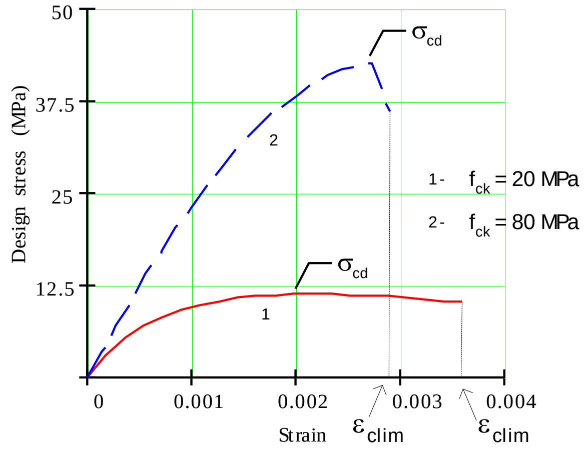

Figura 1 – Design stress-strain curves of a HPC and a conventional concrete.

The stress-strain parabola-rectangle diagram, generally used in the design of conventional concrete, should be substituted by a diagram using the experimental results because HPC stress-strain curve has a different shape. The main characteristics of these curves are the linearity of the initial curve increases with the increase in the concrete strength; the falling branch is much steeper if the compressive strength increases, the strain at peak stress increases if the concrete strength increases and the values of ultimate concrete strain decreases while the concrete resistance increases (Figures 1 and 2).

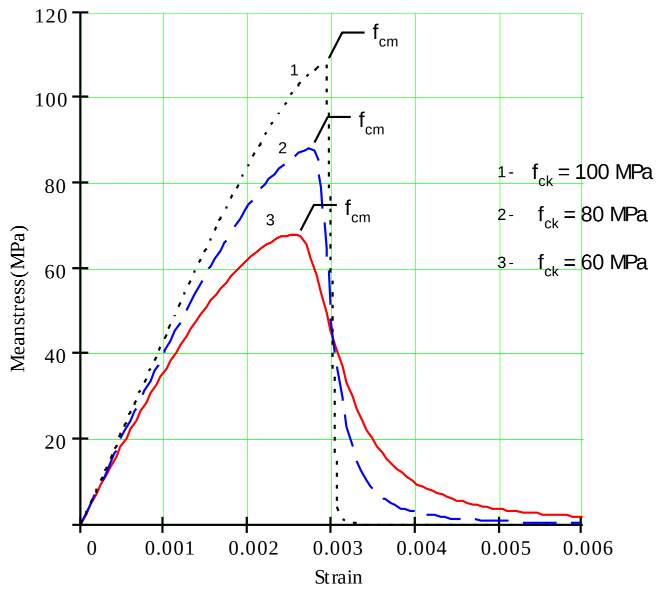

Figure 2 – Realistic curves of 3 different types of HPC for uniaxial compression and short term loads considering the mean compressive strength and showing the entire descending branch.

It is necessary to quantify the resultant force of the compressed concrete given by the integration of area under the realistic design stress-strain curve, to reach the equilibrium of applied and resistant forces. Its manual calculation is quite difficult; thereby it is necessary to have an automatic procedure [7], or tables [7], or axial load-moment interaction diagrams.

At the end of this paper, non-dimensional axial load-moment interaction diagrams are presented for different relationships of d'/h and for concretes with f_{ck}=60 MPa and f_{ck}=80 MPa (Fig. 8, 9, 10 and 11).

In all the equations, the forces' direction shown in Figure 3 corresponds to positive values. Concerning axial force, the positive sign means tensile.

2. HPC STRESS-STRAIN CURVES EQUATIONS

The next expressions are the equations of the more realistic stress-strain curves presented in the CEB-Bulletin d'Information N^{o} 228 (for stresses below 100 MPa), considering the concrete safety coefficient and the long term loads. In all equations, the units of stresses is MPa.

$$\sigma(\varepsilon_c) = \sigma_R(\varepsilon_c) \times \sigma_{cd} \quad (1)$$

$$\sigma_{R\,ascendig}(\varepsilon_c) = \frac{\frac{E_{cd}}{E_{c1}} \times \eta(\varepsilon_c) - [\eta(\varepsilon_c)]^2}{1 + (\frac{E_{cd}}{E_{c1}} - 2) \times \eta(\varepsilon_c)} \quad (2)$$

$$\sigma_{R\,descendig}(\varepsilon_c) = \frac{1}{1 + \left[\frac{\eta(\varepsilon_c) - 1}{\frac{\varepsilon_{c1} + t}{\varepsilon_{c1}} - 1} \right]^2} \quad (3)$$

$$\eta(\varepsilon_c) = \frac{\varepsilon_c}{\varepsilon_{c1}} \quad (4)$$

$$E_{c1} = \frac{\sigma_{cd}}{\varepsilon_{c1}} \quad (5)$$

$$E_{cd} = \frac{22000 \times \left(\frac{f_{cm}}{10} \right)^{0.3}}{\gamma_c} \quad (6)$$

$$\varepsilon_{c1} = 0.0007 \times (f_{cm})^{0.31} \quad (7)$$

$$f_{cm} = f_{ck} + 8 \quad (8)$$

$$\sigma_{cd} = 0.85 \times \frac{f_{ck}}{\gamma_c} \quad (9)$$

$$\gamma_c = \frac{1.5}{1.1 - \frac{f_{ck}}{500}} \quad (10)$$

where

- σ_{cd} is the peak stress (design strength), considering long term loads
- ε_c is the compressive strain
- E_{cd} is the tangent modulus of elasticity at the origin
- E_{c1} is the secant modulus of elasticity at peak stress
- ε_{c1} is the strain that corresponds to the peak stress
- t is a parameter for HPC taken from Table 1
- f_{cm} is the mean value of compressive strength[2]
- f_{ck} is the concrete characteristic compressive strength
- γ_c is the concrete safety coefficient for f_{ck} > 50 MPa.

[2] For some verification in design or for an estimate of other concrete properties it is necessary to refer to a mean value of compressive strength f_{cm} associated with a specific characteristic compressive strength f_{ck} (Equations 6, 7 and Fig. 2).

TABLE 1 – THE PARAMETER "T" FOR HPC

f_{ck} (MPa)	50	60	70	80	90	100
t	0.000807	0.000579	0.000338	0.000221	0.000070	0.000015

Concerning the maximum compressive strain, the next expression is adopted (presented in section 6.2.2.2 of the Bulletin d'Information N^o 228 for a simplified stress-strain diagram, and it could be adopted for a physically adequate stress-strain curve until f_{ck}= 80 MPa):

$$\varepsilon_{c\,\lim} = 0.0025 + 0.002 \times \left(1 - \frac{f_{ck}}{100}\right) \tag{11}$$

3. RESISTANT FORCES

The basic procedures used in the design of conventional concrete are also applied to High Performance Concrete. For HPC the maximum compressive strain used in the design depends on the concrete resistance characteristic.

The compression concrete resultant force and the moment of this resultant in relation to the compressed border are (Fig. 3):

$$R_{cc} = \int_0^x b \times \sigma(\varepsilon_c) \times dy \tag{12}$$

$$M_{Rcc} = \int_0^x b \times \sigma(\varepsilon_c) \times y \times dy \tag{13}$$

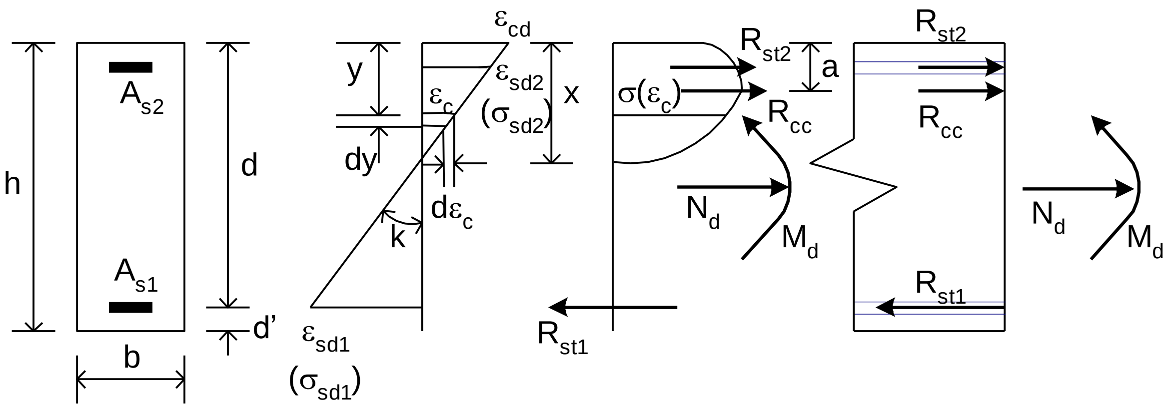

Figure 3 – Stresses and strains diagrams.

- **Transformation of the Expressions to Non-dimensional Values**

In order to simplify, the different formulations are presented in parametric and non-dimensional form (all the terms are shown in Fig. 3 and 4), reduced by h, b and σ_{cd},

where

$\sigma_{cd} (= 0.85 \times f_{cd})$ is the peak compressive stress (design compressive strength), considering long term loads.

Considering Fig. 3 and 4, the terms y and θ can be related to modify the integration variables of Equations 12 and 13:

$$y = \frac{h}{\theta} \times (\varepsilon_{cd} - \varepsilon_c) \qquad dy = -\frac{h}{\theta} \times d\varepsilon_c \tag{14}$$

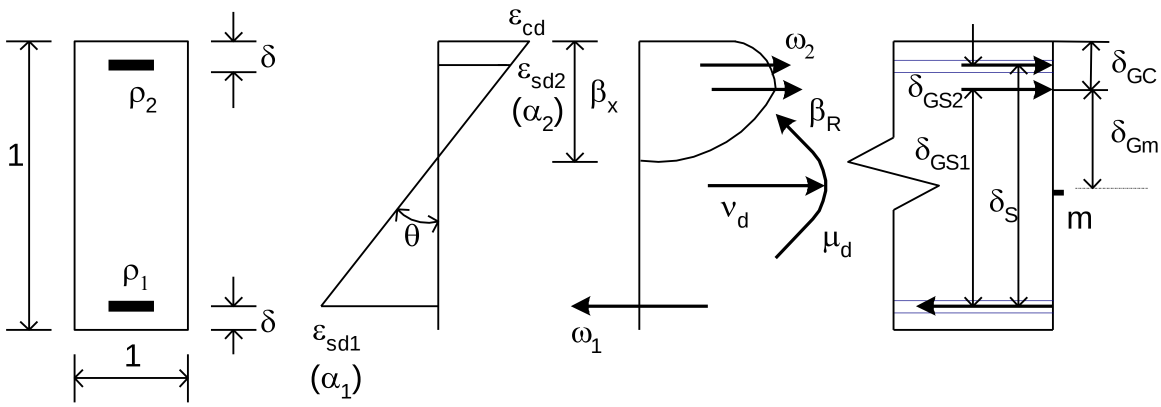

Figure 4 – Stresses and strains diagrams showing the reduced parameters.

Substituting (14) in Equations 12 and 13, parametric equations of the concrete resultant force and its moment with regard to the compressed border can be obtained:

$$\beta_R = \frac{1}{\theta} \times \int_0^{\varepsilon_{cd}} \sigma_R(\varepsilon_c) \times d\varepsilon_c \qquad (15)$$

$$\mu_R = \frac{1}{\theta^2} \times \int_0^{\varepsilon_{cd}} \sigma_R(\varepsilon_c) \times (\varepsilon_{cd} - \varepsilon_c) \times d\varepsilon_c \qquad (16)$$

where $\sigma_R(\varepsilon_c)$ is an equation relating the design stress (considering the long term loads) and strain obtained by the Bulletin d'Information N^o 228.

It is necessary to know the reduced curvature θ, which will be obtained for several positions of the neutral axis, in order to obtain the values of β_R and μ_R. For this purpose, the strain diagram is divided in small intervals, solving the Equations 15 and 16 for each one of these intervals.

4. FAILURE CONFIGURATIONS

The different design domains (strain distribution over the depth of the member) are obtained by modifying the neutral axis position and by rotating the deformation diagram in relation to fixed points, called rotation centers (Fig. 5).

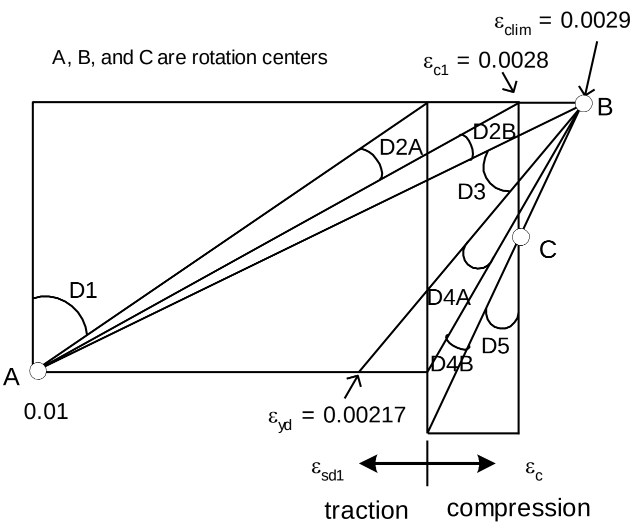

Figure 5 – Reinforced concrete strain domains with f_{ck}=80 MPa and f_{yk}=500 MPa.

The distance between compressed border and neutral axis is taken into account by the reduced parameter $\beta_{x,}$ which is obtained using the relationship of triangles (Figure 4) in function of the strains ε_{sd1} and ε_{cd} (here is adopted a modulus of elasticity of steel E_s=200 GPa).

$$\beta_x = \frac{\varepsilon_{cd}}{\varepsilon_{sd1} + \varepsilon_{cd}} \times (1-\delta) \tag{17}$$

- **Reference Failure**

It is necessary to know the reference value β_{xref}, located when considering the limit between the domains D2B and D3 (Fig. 6), in order to quantify the value of θ. β_{xref} is a comparative value (corresponding to a failure) considered as a reference for the rotation centers change. So, β_{xref} is obtained from Equation 17 with ε_{sd1} = 0.0010 (steel strain limit, other values may apply if the ductility of the steel is known) and ε_d given by the maximum compressive strain ε_{clim}=0.0029 (for f_{ck} = 80 MPa, see Equation 11):

$$\beta_{xref} = \frac{29}{129} \times (1-\delta) \tag{18}$$

The values of reduced curvature θ (obtained from Figures 4 and 5) are the following:

- If $\beta_x \leq \beta_{xref}$, the failure criteria corresponds to the domains D2A, D2B.
  The rotation center is located on point A (ε_{sd1} = 0.01).

$$\theta = \frac{0.01}{1-\beta_x-\delta} \tag{19}$$

- If $\beta_{xref} < \beta_x \leq 1$, the failure criteria corresponds to the domains D3, D4A, D4B.
  The rotation center is located on point B (ε_{cd} = 0.0029).

$$\theta = \frac{0.0029}{\beta_x} \tag{20}$$

- If $\beta_x > 1$, the failure criteria corresponds to the domain D5 (Fig. 6).
  The rotation center is the point C (placed on the vertical line of the strain that corresponds to the peak stress, see Equation 7 and Fig. 6).

$$\theta = \frac{812}{290 \times \beta_x - 10} \tag{21}$$

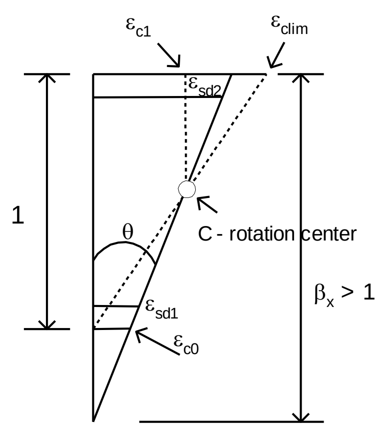

Figure 6 – Failure strain configuration of the domain D5.

5. EQUILIBRIUM EQUATIONS

The equations of the resistant moments are obtained in relation to the centroid of the concrete section and to the centroid of the reinforcing steels. The acting moments are determined after transferring the axial force to the position of the reinforcing steels, and the equilibrium equations are obtained between the external and resistance forces with regard to the reinforcing steels position. Figure 4 shows these terms, which are presented below.

Distance between concrete resultant force position and compressed border:

$$\delta_{GC} = \frac{\mu_R}{\beta_R} \tag{22}$$

Resistant moment of the compressed concrete resultant force in relation to the centroid of the section and distance between concrete resultant force position and centroid of whole section:

$$\mu_{Gm} = \beta_R \times \delta_{Gm} \qquad \delta_{Gm} = 0.5 - \delta_{GC} \tag{23}$$

Resistant moment of the compressed concrete resultant force in relation to the position of the reinforcing steel A_{s1} (tensile reinforcing steel for the applied bending moment) and distance between concrete resultant force position and the reinforcing steel A_{s1} position:

$$\mu_{GS1} = \beta_R \times \delta_{GS1} \qquad \delta_{GS1} = 1 - \delta_{GC} - \delta \tag{24}$$

Resistant moment of the compressed concrete resultant force in relation to the position of the reinforcing steel A_{s2} (compressed reinforcing steel for the applied bending moment) and distance between concrete resultant force position and the reinforcing steel A_{s2} position:

$$\mu_{GS2} = \beta_R \times \delta_{GS2} \qquad \delta_{GS2} = \delta_{GC} - \delta \tag{25}$$

Reduction of the external forces in relation to the position of the two reinforcing steels:

$$\mu_{ext1} = \mu_d - \nu_d \times \frac{\delta_S}{2} \tag{26}$$

$$\mu_{ext2} = \mu_d + \nu_d \times \frac{\delta_S}{2} \tag{27}$$

General equilibrium equations between the external and resistance forces in relation to the reinforcing steels positions are:

$$\nu_d = \omega_1 - \beta_R - \omega_2 \tag{28}$$

$$\mu_{ext1} = \omega_2 \times \delta_S + \beta_R \times \delta_{GS1} \tag{29}$$

$$\mu_{ext2} = \omega_1 \times \delta_S - \beta_R \times \delta_{GS2} \tag{30}$$

6. PRINCIPLES OF SILVA JR.'S METHOD [9]

This method distributes the applied forces, inside of design "regions" created in function of the resistant axial forces and bending moment's values. All the compressed actions and their effects will have negative sign.

Six regions are defined, established in the following way (Figure 7):

- Region *O*, where the concrete balances the applied forces and there is not reinforcing steel (only the minimum reinforcement). It is "the eye" of the diagram obtained with the values of β_R and $\mu_{Gm.}$
- Region *A*, where the concrete is considered uniformly compressed, in other words, the concrete and the reinforcing steel are working with a strain that correspond to the peak stress ε_{c1}=-0.0028 (for f_{ck}= 80 MPa). Usually, it happens in columns under combined compression and bending with small eccentricity.
- Region *C*, that is usually considered a balanced configuration of failure (located in the limit between the domain D3 and *D*4), in other words, the compressed border of the concrete working with their maximum strain ε_{cd}=-0.0029 (ε_{clim} for the concrete with f_{ck}=80 MPa), and the tensile reinforcing steel with a strain corresponding to the beginning of yield ε_{sd1}=0.00217 (for a steel with the characteristic yield stress f_{yk}=500 MPa and E_s=200 GPa).

- Region *E*, where the section is considered uniformly tensiled, the concrete does not contribute with its small tensile resistance, and the reinforcing steels are requested with their maximum strain ε_{sd}=0.01. Usually, it is the case of hangers under combined tensile force and bending with small eccentricity.
- Region *B* is a transition between the zones *A* and *C*, required reinforcing steel A_{s2} (compressed reinforcing steel for the applied bending moment), the other is minimum.
- Region *D* is a transition between the zones *C* and *E*, required reinforcing steel A_{s1} (tensile reinforcing steel for the applied bending moment), the other is constructive.

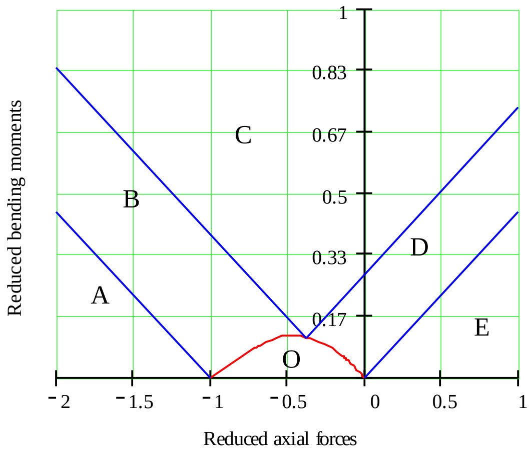

Figure 7 – Diagram of Silva Jr, showing the 6 "regions".

The boundary straight lines separating these "regions" were obtained from Equations 26 to 30 and the boundary conditions of each "region".

7. NONDIMENTIONAL INTERACTIVE CHARTS USING THE MATHCAD PROGRAM

To obtain the design charts, the Silva Jr.'s Method, the Equations 26 to 30 and also the following expressions are used:

Mechanic ratio of the total reinforcing steel

$$\omega = \frac{A_s \times f_{yd}}{b \times h \times \sigma_{cd}} \tag{31}$$

Geometric ratio of the total reinforcing steel

$$\rho = \frac{A_s}{b \times h} \tag{32}$$

Fraction of the reinforcing steel A_{s1}

$$\eta = \frac{\rho_1}{\rho} \tag{33}$$

Fraction of the reinforcing steel A_{s2}

$$(1-\eta) = \frac{\rho_2}{\rho} \tag{34}$$

where ρ_1 and ρ_2 are the geometric ratio of A_{s1} and A_{s2} respectively.

To obtain the charts, initially the intersection points between the ω parametric curves and the boundary straight line dividing the Silva Jr.'s regions have to be found (Figures 8, 9, 10 and 11). To obtain these intersections, the Equations 26 to 34 and the boundary conditions of each region must be considered.

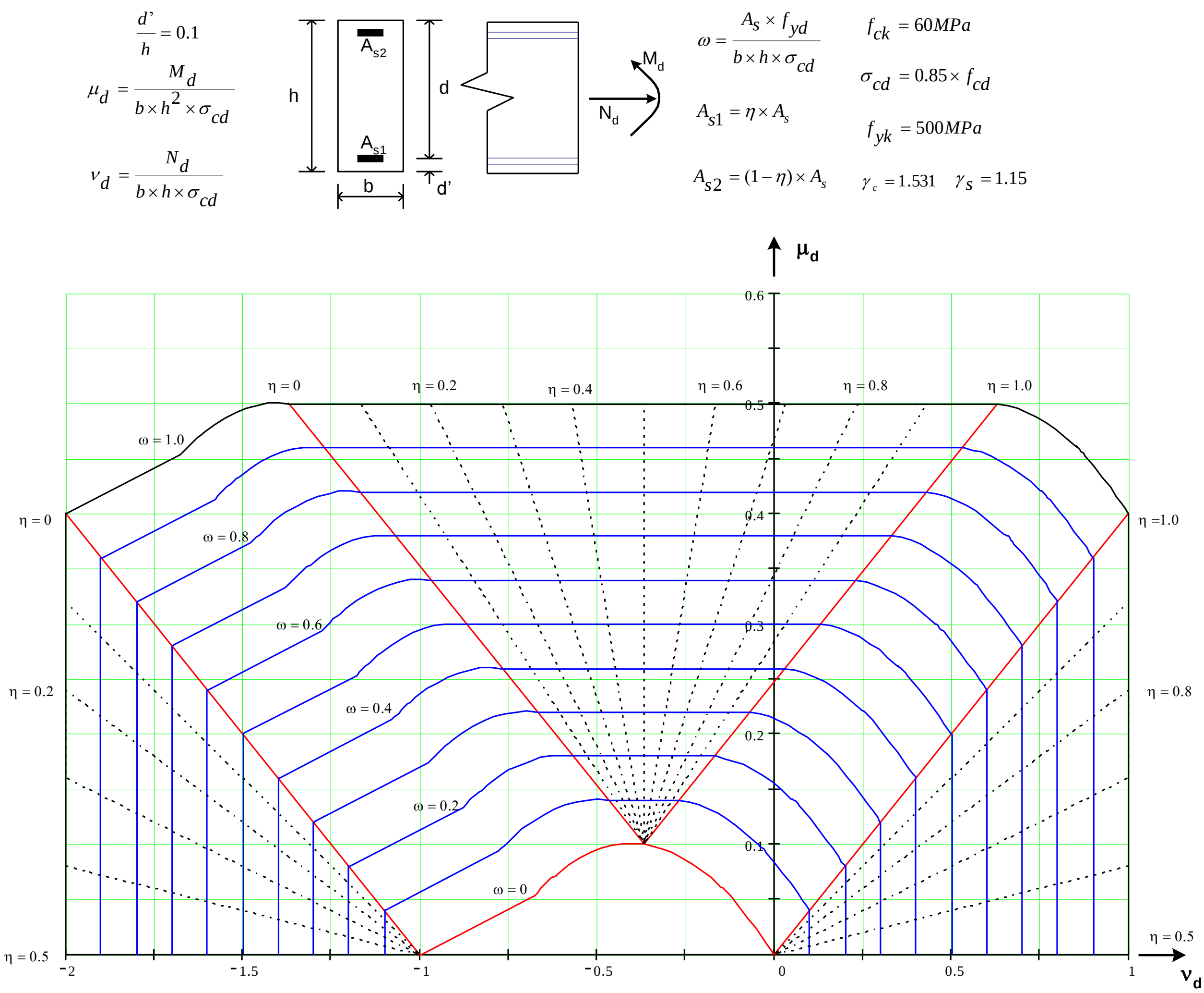

Figure 8 – Design axial load and moment interaction chart for HPC with d'/h=0.10 and fck=60 MPa – CEB Bulletin d'Information No 228.

Each curve corresponds to each value of the mechanical ratio of reinforcing steel ω, which is obtained connecting the intersection points (of the same ω) with the lines dividing Silva Jr.'s regions.

It is demonstrated that ω curves in the regions *A*, *C* and *E* correspond to a straight lines, and in regions *B* and *D* to curves [9]. Therefore, the equations of these parametric curves were obtained considering boundary conditions of those regions, Equations 26 to 34 and the arbitrary value of ω parameter.

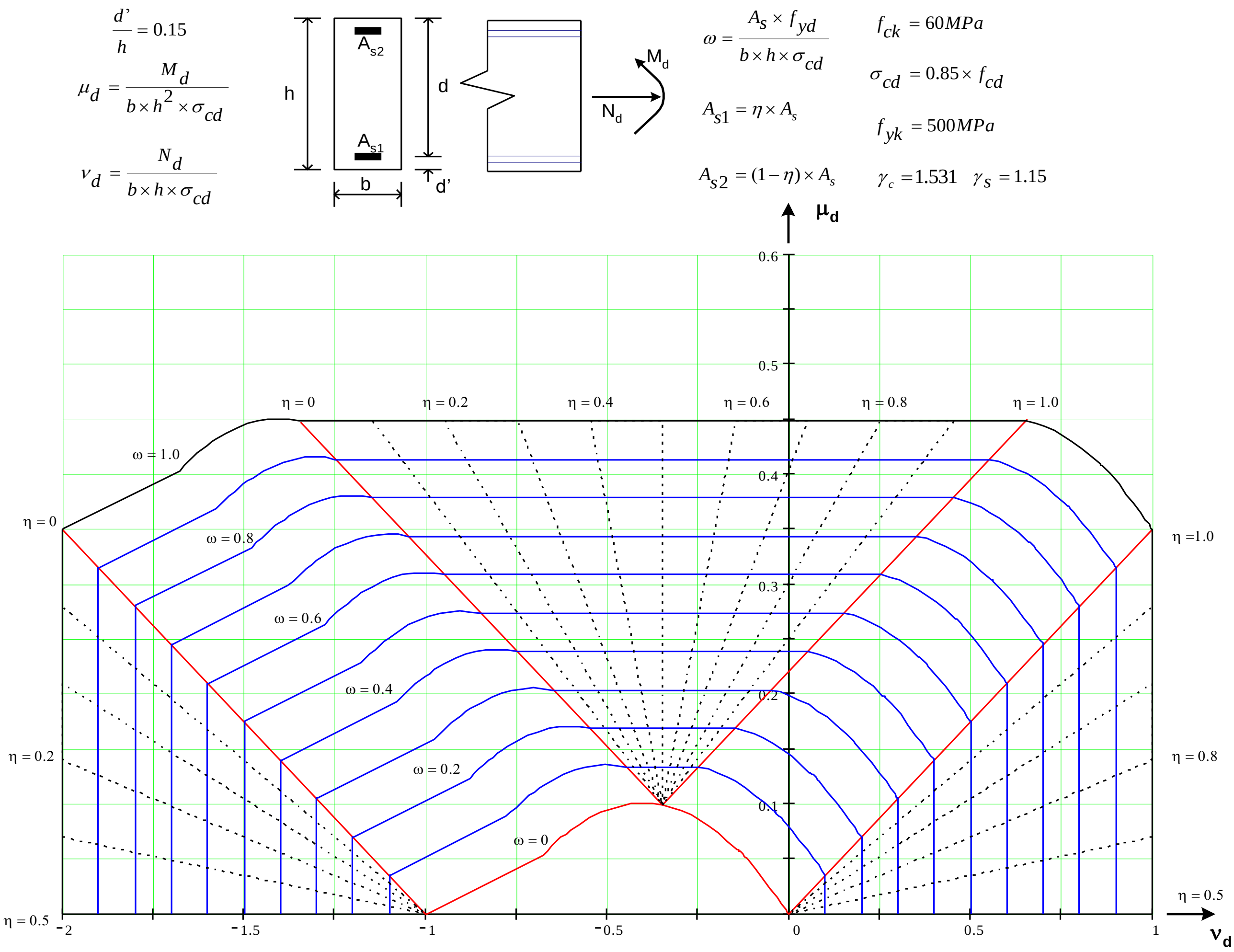

Figure 9 – Design axial load and moment interaction chart for HPC with d'/h=0.15 and f_{ck}=60 MPa – CEB Bulletin d'Information No 228.

As an example, to obtain these charts for a rectangular section, a characteristic concrete strength f_{ck}=60 MPa and f_{ck}=80 MPa, a steel yield stress f_{yk}=500 MPa and values of d'/h=0.1 and 0.15, the Mathcad program resources were used (Fig. 8, 9, 10 and 11).

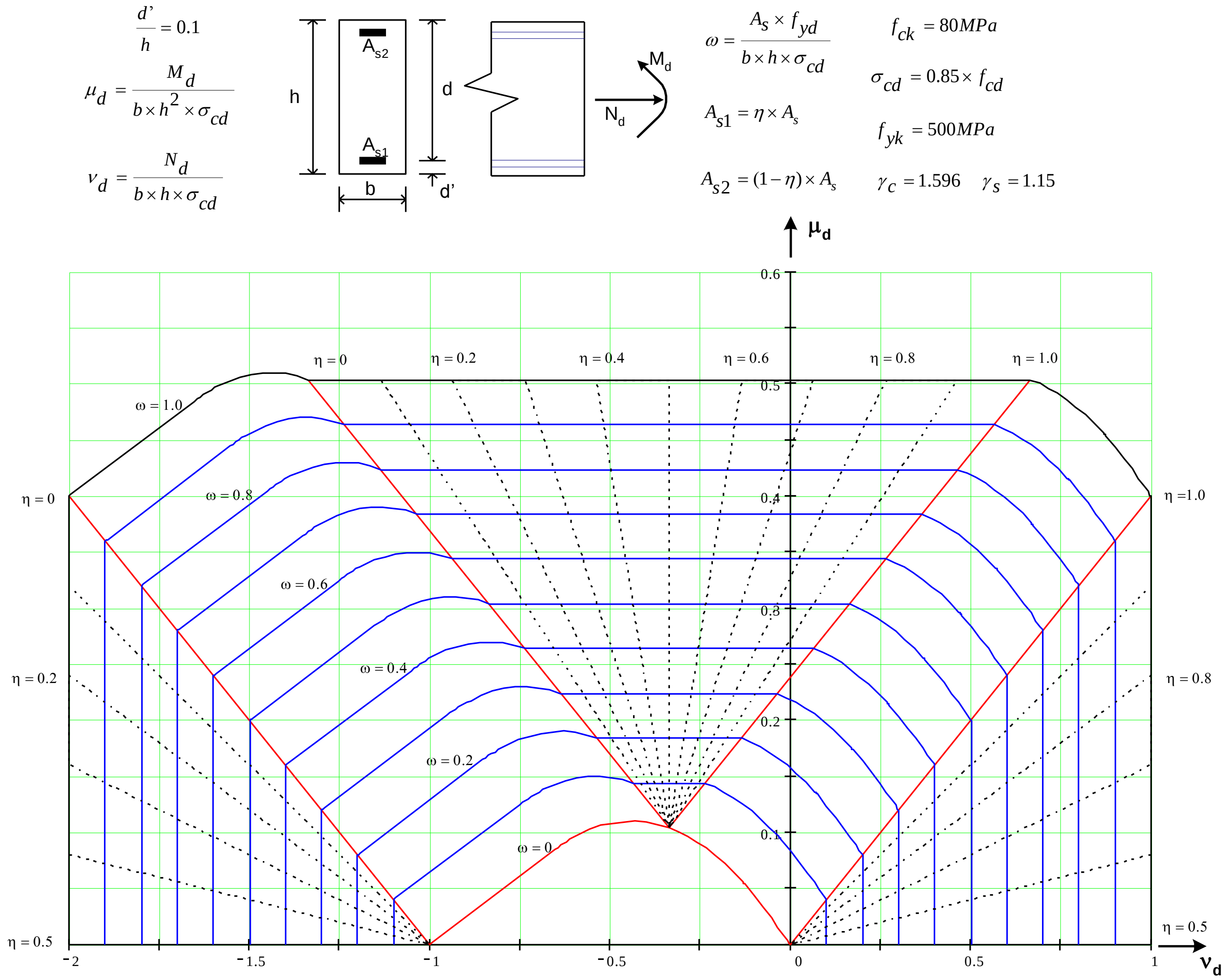

Figure 10 – Design axial load and moment interaction chart for HPC with d'/h=0.10 and f_{ck}=80 MPa – CEB Bulletin d'Information No 228.

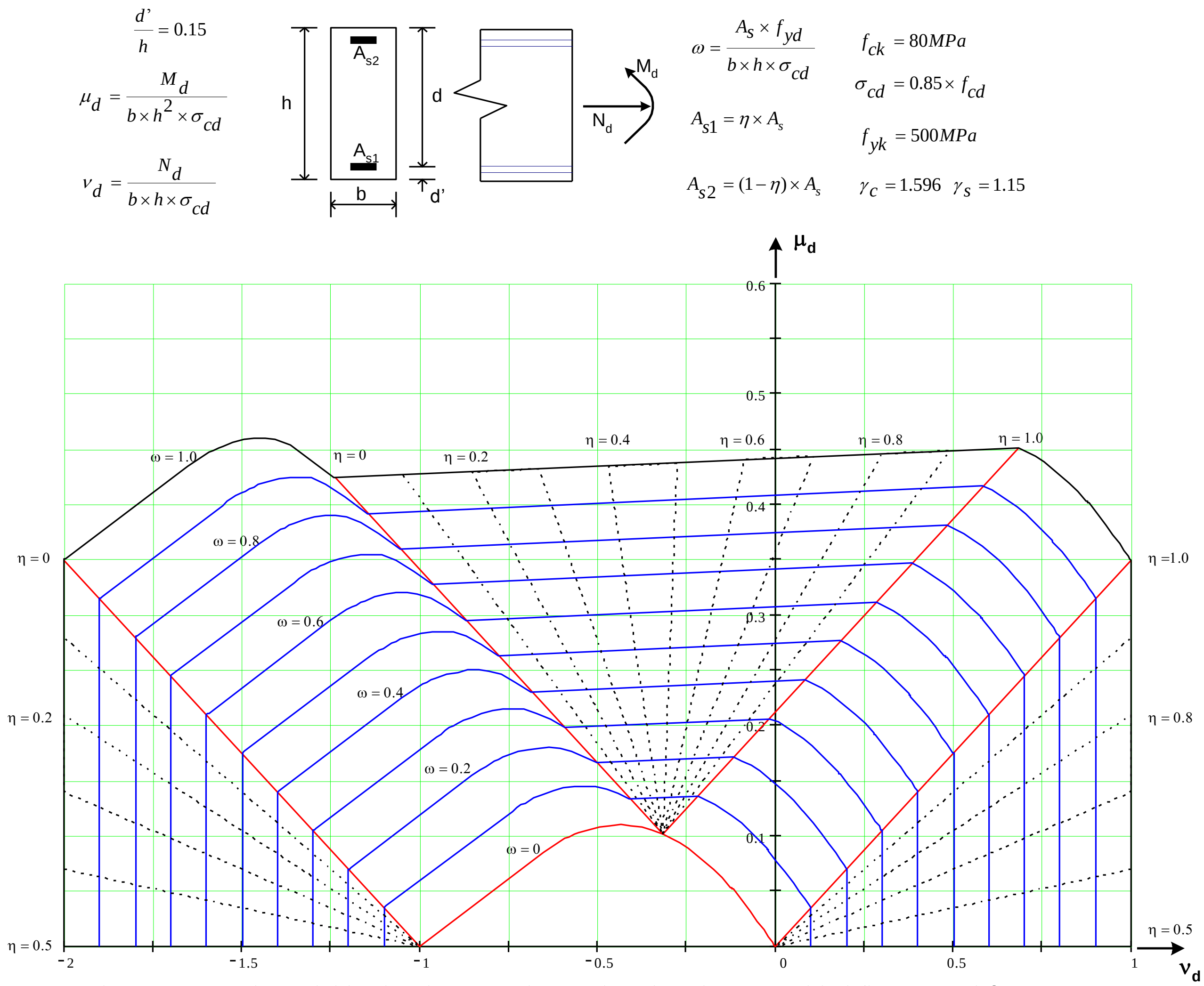

Figure 11 – Design axial load and moment interaction chart for HPC with d'/h=0.15 and f_{ck}=80 MPa – CEB Bulletin d'Information No 228.

The design charts are compared to the automatic design applying the routine of [7]. The results are listed in Table 2 for a concrete section of 25 x 50 cm², a characteristic concrete strength f_{ck}=80 MPa, a steel yield stress f_{yk}=500 MPa and a relationship d'/h=0.1.

TABLE 2 – STEEL REINFORCEMENT OBTAINED BY THE CHARTS COMPARED WITH AN AUTOMATIC DESIGN FOR f_{ck}=80 MPA AND f_{yk}=500 MPA WITH CONCRETE SECTION OF 25 x 50cm^2 AND A RELATIONSHIP d'/h = 0.1.

N_d (kN)	M_d (kNm)	ν_d	μ_d	CHART (Fig. 10) $A_s(cm^2)$				Automatic Design Ref. [7], $A_s(cm^2)$	
				ω	η	A_{s1}	A_{s2}	A_{s1}	A_{s2}
-6274.3	82.9	-1.178	0.031	0.18	0.28	6.17	15.87	6.13	15.66
-3969.5	214.6	-0.745	0.081	0.03	0	0	3.67	0	3.51
0	718.4	0	0.270	0.42	0.90	46.30	5.14	45.96	4.83
-1079.5	533.4	-0.203	0.200	0.25	0.78	23.89	6.74	22.90	6.61
799	369.8	0.150	0.139	0.25	1	30.62	0	29.88	0
1258	140.3	0.236	0.053	0.25	0.78	23.88	6.74	22.53	6.40
-2894.3	72.3	-0.543	0.027	0	0	0	0	0	0

8. CONCLUSIONS

When HPC is used, it is necessary to consider stress-strain diagrams having more realistic characteristics for this material. Codes regulating this material have realistic curves allowing a more appropriate design to fit the experimental results. The traditional procedures developed for the conventional concretes have to be modified for High Performance Concrete design. Advantages of the potential of HPC could be obtained improving the current design procedures.

The simplified uniform stress diagram used in the manual design has no more advantages because now, any design is made automatically. To obtain non-dimensional interaction axial force-bending moment charts, more realistic stress-strain curves can also be used, as it was done in this work.

Considering that HPC presents more brittle failure characteristics than conventional concrete, it is recommended to have a more discerning evaluation in the reinforcing steel placement, adopting more realistic stress-strain curve in order to assure ductility to the section.

In cross sections where compression controls, HPC brings a substantial economy, reducing the amount of reinforcing steel and concrete, this means, smaller dimensions in the columns, increasing the useful area of the place.

The application of Silva Jr.'s Method allows an economical, fast and direct design (without trials)

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