THE USE OF INFORMATICS IN THE DESIGN AND DETAILING PROCESS OF CONCRETE STRUCTURES ACCORDING TO THE EC2

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Abstract

The use of Eurocodes as the actual structural design standards implies the safety verifications with some level of difficulty due several reasons. Therefore the development of computer software as helpful tools to minimize the processes of verification is a very important aspect to have in consideration.

The purpose of this work is to establish the methodologies to automate the process for the design of reinforced concrete structures according the Eurocodes, including checking the safety of ultimate and serviceability limit states and detail of the reinforcement. A computer program capable of performing the procedures for carrying out the processes defined above was developed. This work covers the aspects recommended in Eurocode 2[1] to be taken in checking the safety of ultimate states of flexure and shear, as well the aspects to be considered for safety assessment of the serviceability limit state for cracking and deflection.

The work is developed on rectangular and flanged beams. The computer program checks the safety of beams for serviceability and ultimate limit states with different types of support and number of spans and does the detailing of longitudinal and shear reinforcement. It's possible to store the obtained results as image and text files.

The computer program is tested with two practical examples. In one of the examples the obtained results are compared with results obtained manually.

Keywords:
Automate; Eurocode 2; Reinforced concrete; Beams; Reinforcement details; Limit states.

1 Introduction

The introduction of the Eurocodes as the current standards for the structural design and safety verification marked the beginning of a new age within the framework of the design of buildings and other civil engineering works.

The verification conditions specified in the Eurocodes usually requires repetitive and time consuming procedures needing the use of numerical methods of complexity and subject to human error. The use of computer software to support the various areas of structural project thus becomes an indispensable tool.

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2 Basis for verification

2.1 Limit states and the partial safety factors method

Limit states are the states which the structure no longer fulfils the relevant design criteria. A fundamental distinction is made between ultimate limit states and serviceability limit states.

Ultimate limit states (ULS) are associated to collapse or with other similar forms of structural failure. Serviceability limit states (SLS) correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met.

According to the partial safety factors method, a structure is considered safe if no relevant limit state is exceeded, when calculation design values for actions or effects of actions and resistance are applied in the design models [2].

2.1.1 Verification of limit states

When considering an ultimate limit state of rupture or excessive deformation of a section, member or connection, it shall be verified that:

$$E_d \leq R_d \qquad (2.1)$$

Where E_d is the design value of the effect of action; R_d is the design value of the corresponding resistance.

In serviceability limit states it shall be verified that:

$$E_d \leq C_d \qquad (2.2)$$

Where E_d is the design value of the effects of actions; C_d is the limiting design value of the serviceability criterion.

2.2 Combination of actions

To verify the structural reliability the limit states and design situations shall be specified first.

The combination of actions for ultimate limit sates is expressed as:

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{ " + " } \gamma_p P \text{ " + " } \gamma_{Q,1} Q_{k,1} \text{" + " } \sum_{i\geq 1} \gamma_{Q,i}\, \psi_{0,i} Q_{k,i} \qquad (2.3)$$

The combination of actions used to verify the serviceability limit states depends on the nature of the effects of actions.

a) Characteristic combination of actions (EN1990 [2] expression (6.14)):

$$\sum_{j\geq 1} G_{k,j} \text{ " + " } P_k + Q_{k,1} \text{"+"} \sum_{i\geq 1} \psi_{0,i} Q_{k,i} \qquad (2.4)$$

b) Frequent combination (EN1990 [2] expression (6.15)):

$$\sum_{j\geq 1} G_{k,j} \text{ " + " } P_k \text{ "+" } \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (2.5)$$

c) Quasi-permanent (EN1990 [2] expression (6.16)):

$$\sum_{j\geq 1} G_{k,j} \text{ " + " } P_k \text{ "+"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (2.6)$$

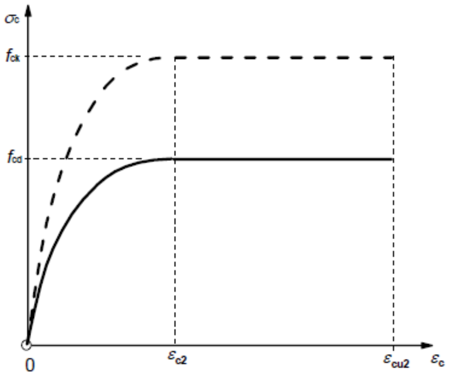
2.3 Materials

2.3.1 Concrete

The strength and deformation characteristics for concrete extensions are found in Eurocode En 1992-1-1 [1] on table 3.1

The concrete safety factor γ_c is equal to 1,5 in persistent and transient design situation

For the design of cross-section the Eurocode 2 [1] recommends the use of the parabola-rectangle diagram of stress-strain relationship (Figure 2.1) obtained from expressions (2.7) and (2.8).



$$\sigma_c = f_{cd}\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \qquad \textit{(2.7)}$$

$$for\ 0 \leq \varepsilon_c \leq \varepsilon_{c2}$$

$$\sigma_c = f_{cd}\ for\ \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \qquad \textit{(2.8)}$$

The parameters of the expressions are found in table 3.1 [1]

Figure 2.1 Parabola-rectangle diagram for concrete under compression.

2.3.2 Reinforcing steel

The value recommended in Eurocode 2 [1] for the steel yield strength f_{ck} is 400 to 600 Mpa.

The partial factor γ_s is equal to 1,15 in persistent and transient design situation.

For normal design purposes the Eurocode 2 recommends idealized and design stress-strain diagrams for reinforced steel (Figure 2.1).

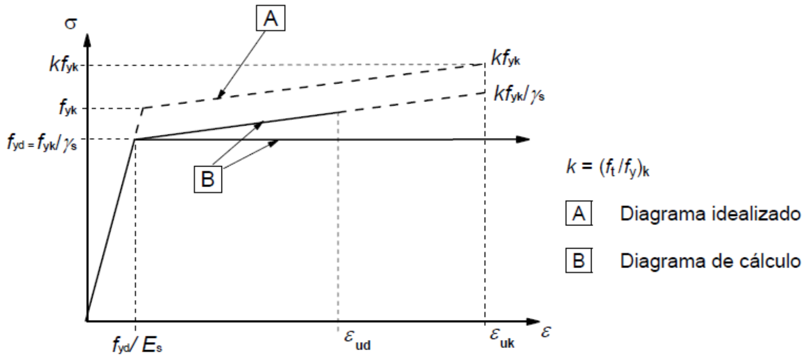


Figure 2.2 - Idealized and design stress-strain diagrams for reinforced steel

The design value of the modulus of elasticity, $E_s$may be assumed to be 200 Gpa.

2.3.3 Creep

The creep is expressed by the creep coefficient $\varphi(t,t_0)$, the creep deformation of concrete $\varepsilon_{cc}(\infty, t_0)$ at time $t = \infty$ for a constant compressive stress $\sigma_c$at the concrete age t_0, is given by:

$$\varepsilon_{cc}(\infty, t_0) = \varphi(\infty, t_0)(\sigma_c/E_c) \qquad \textit{(2.9)}$$

The values of creep coefficient may be determinate using the data from figure 3.1 of Eurocode 2 [1] when high accuracy is not necessary. The method for determining the creep coefficient with more precision is specified in annex B of Eurocode 2.

3 Verifications of the serviceability limit states according to EC2

According to Eurocode 2 [1] (Section 7.4.2) Generally, it is not necessary to calculate the deflections explicitly as simple rules, for example limits to span/depth ratio may be formulated, which will be adequate for avoiding deflection problems in normal circumstances. More rigorous checks are necessary for members which lie outside such limits, or where deflection limits other than those implicit in simplified methods are appropriate.

The simplified method used in EC2 (Section 7.4.2) consists on limiting span/depth ratio multiplied by correction factors to allow for the type of reinforcement used and other variables. The limiting span/depth ratio is given by the following expressions:

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3{,}2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right]\ if\ \rho \le \rho_o \qquad (3.1)$$

(Expression 7.16a of Eurocode EN1992-1-1 [1])

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] if\ \rho > \rho_o \qquad (3.2)$$

(Expression 7.16b of Eurocode EN1992-1-1 [1])

3.1 Crack control

Cracking shall be limited to a width that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. The Eurocode 2 [1] accepts cracking in reinforced concrete as a normal and inevitable occurrence.

3.1.1 Calculation of crack width

The crack width may be calculated from expression

$$W_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) \qquad (3.3)$$

$s_{r,max}$ is the maximum crack spacing; ε_{sm} is the mean strain in the reinforcement; ε_{cm} is the mean strain in the concrete between cracks.

3.1.2 Minimum reinforcement areas

According to the Eurocode 2 [1] the minimum reinforcement area $A_{s,min}$ for control of cracking is determinate by:

$$A_{s,min} = k_c\, k\, f_{ct,eff}\frac{A_{ct}}{\sigma_s} \qquad (3.4)$$

4 Verifications of the ultimate limit states according to EC2

4.1 Bending without axial force

The design of cross sections of reinforced concrete consists in evaluate and limit the steel and concrete strains due loads effects. Figure 4.1 outlines the behavior of a cross-section subjected to pure bending. Four distinct cases are noted, depending on the established limits resumed in Figure 4.1

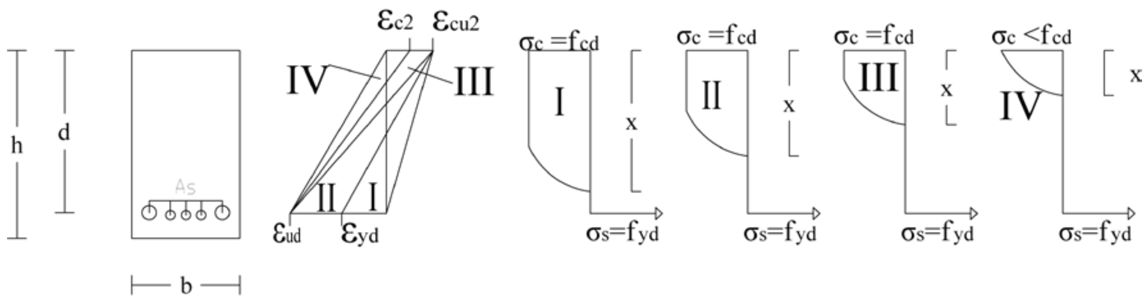


Figure 4.1 Limit strains of concrete and steel and the respective tensions distribution.

The failure of the cross-section occurs if the concrete achieves strain ε_{cu2} or if the steel achieves the ultimate strain ε_{ud}

Table 4.1 Possible yield cases.

Case	Concrete	Rupture	Steel	Rupture
I	$\varepsilon_c = \varepsilon_{cu2}; \sigma_c = f_{cd}$	✓	$0 < \varepsilon_s < \varepsilon_{yd}; \sigma_s < f_{yd}$	
II	$\varepsilon_c = \varepsilon_{cu2}; \sigma_c = f_{cd}$	✓	$\varepsilon_{yd} \leq \varepsilon_s \leq \varepsilon_{ud}; \sigma_s \geq f_{yd}$	
III	$\varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2}; \sigma_c = f_{cd}$		$\varepsilon_s = \varepsilon_{ud}; \sigma_s \geq f_{yd}$	✓
IV	$\varepsilon_c \leq \varepsilon_{cu2}; \sigma_c < f_{cd}$		$\varepsilon_s = \varepsilon_{ud}; \sigma_s \geq f_{yd}$	✓

For design criteria it is considered the hypothesis that the failure of the cross-section is caused by the rupture of concrete while the steel reaching the design strength (case II) thereby ensuring a ductile rupture, as exemplified in the following schematic of the beam behavior.

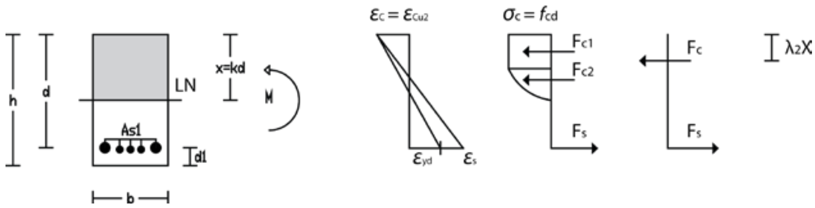


Figure 4.2 Cross-section under pure bending behavior

Using the expressions (2.7) and (2.8), for the stress-strain relationship determining the following parameters are obtained

$$\lambda_1 = 1 - \frac{1}{n+1} \cdot \frac{\varepsilon_{c2}}{\varepsilon_{cu2}} \tag{4.1}$$

$$c = \left(1 - \frac{\frac{1}{2} - \frac{1}{(n+1)(n+2)} \left(\frac{\varepsilon_{c2}}{\varepsilon_{cu2}}\right)^2}{1 - \frac{1}{n+1}\left(\frac{\varepsilon_{c2}}{\varepsilon_{cu2}}\right)} \right) x \tag{4.2}$$

$$c = \lambda_2 x \tag{4.3}$$

4.1.1 Non-dimensional forms

The action effects are often reduced to a non-dimensional form [3]

$$\mu = \frac{M}{bd^2 f_{cd}} \tag{4.4}$$

$$\omega = \frac{A_s f_{yd}}{bd f_{cd}} \tag{4.5}$$

$$\alpha = \frac{x}{d} \tag{4.6}$$

Were μ is the reduced bending moment; ω is the mechanical reinforcement ratio; x is the height of compression zone; d is the distance between centroid of reinforcement and concrete fibre ith minimum strain.

The relation between ω and μ are obtained by the following expressions:

$$\omega = \frac{f_{cd}\lambda_1 bx}{bd f_{cd}} = \lambda_1 \alpha \tag{4.7}$$

$$\mu = (1 - \lambda_2 \alpha)\lambda_1 \alpha \tag{4.8}$$

4.1.1 Neutral axis depth limitation

To ensure a minimum level of available ductility the neutral axis depth should be limited the value of this limitation depends of the structural analysis provided in the Eurocode 2 [1] (Section

5) used in determining the internal forces in the structure. The recommended analyses are linear elastic analysis, linear elastic analysis with limited redistribution and plastic analysis.

Cross sections whose neutral axis depth reaches the limits defined above represent undesirable situations. The solution to overcome this situation is to increase the height of the section or use compressive reinforcement. For a limited neutral axis depth the following expressions are made:

$$\alpha_{lim} = \frac{x_{u,lim}}{d} \quad (4.9)$$

$$\omega_{lim} = \lambda_1 \alpha_{lim} \quad (4.10)$$

$$\mu_{lim} = (1 - \lambda_2 \alpha_{lim}) \lambda_1 \alpha_{lim} \quad (4.11)$$

If $\mu_{sd} \leq \mu_{lim}$ the cross-section is designed without compressive reinforcement

If $\mu_{sd} \geq \mu_{lim}$ the cross-section is designed with compressive reinforcement

4.1 Shear

In section 6.2 of Eurocode 2 [1] indicates de condition of shear safety expressed by:

$$V_{Ed} \leq V_{Rd} \quad (4.12)$$

V_{Ed} is the design value of the applied shear force; V_{Rd} is the design shear resistance.

The design of members with shear reinforcement is based on a truss model indicated in Eurocode 2 [1] section 6.2.

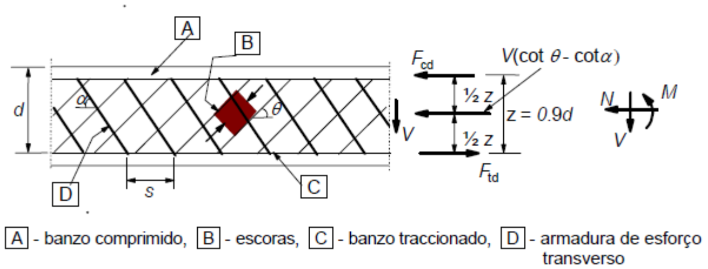


Figure 4.3 Truss model

The values of $V_{Rd,s}$ and $V_{Rd,max}$ are determined by the following expressions:

$$V_{Rd}, s = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta \ + \cot\alpha) \sin\alpha \quad (4.13)$$

$$V_{Rd,max} = \frac{\alpha_{cw} b_w Z v_1 f_{cd} (\cot\theta + \cot\alpha)}{1 + \cot^2\theta} \quad (4.14)$$

4.1.1 Effect on longitudinal reinforcement

The action of shear influences the internal forces resulting from bending. Eurocode 2 defines an additional tensile force ΔF_{td} due shear that may be calculated from:

$$\Delta F_{td} = 0{,}5 V_{Ed} (\ \cot\theta - \cot\alpha\ ) \quad (4.15)$$

The resulting force is given by:

$$F_s = \frac{M_{Ed}}{z} + \Delta F_{td} < \frac{M_{Ed,max}}{z} \quad (4.16$$

Where $M_{Ed,max}$ is the maximum moment along the beam

5 Program overview

As part of this work a computer program was developed the E2VIGA for checking the safety of limit states of reinforcement concrete beams, submitted to continuous loads according the methods of Eurocode 2 [1].

5.1 Aspects included in computation.

The minimum area of longitudinal tension reinforcement is calculated by the simplified expression from Eurocode 2 (section 9.2.1.1).

$$A_{s,min} = 0{,}26\frac{f_{ctm}}{f_{yk}}b_t d \qquad (5.1)$$

The cross-sectional area of tension or compression reinforcement is determined by:

$$A_{s,max} = 0{,}04A_c \qquad (5.2)$$

5.2 Amount of reinforcement needed for crack control

The method used for assure the safety to serviceability limit states is the following

For a given longitudinal reinforcement area obtained to satisfy the ultimate limit states of bending the following expression is considered:

$$A_s = A_{s,elu} + A_{s,els} \qquad (5.3)$$

With:

$A_{s,elu}$ is the reinforcement area necessary for ultimate limit states of bending

$A_{s,els}$ is the reinforcement area needed to satisfy the serviceability limit sats.

In a first approach the value of $A_{s,els}$ is equal to zero.

1- Detailing is made
2- The crack width is calculated
   If $w_k \geq w_{max}$ the section satisfy the safety to serviceability limit states
   If $w_k \leq w_{max}$ the section is considerd unsafed to seviceability limit states
   The reinforcement area of serviceability limit states is increased up to a value of $0{,}01\ cm^2$ then a new area is determinate. Returning to step 1 and repeating the process.

5.3 Spacing of bars and number of bars

The maximum spacing of bars considered in the program is calculated by the following expression:

$$S_{min} = K\emptyset_{varao}\ (mm) \qquad (5.4)$$

With:

K is a multiplication parameter defined by the user.

The maximum number of spaces n_{esp} allowed for a section with width b in each layer of bundles is determinate by:

$$n_{esp} = \frac{b}{S_{min}} \qquad (5.5)$$

Thus the number of bars $n_{varões}$ in each layer is calculated by

$$n_{varões} = n_{esp} + 1 \qquad (5.6)$$

5.4 Longitudinal reinforcement at supports

The section at supports should be designed for a bending moment at least 15% of the maximum bending moment in the span:

$$A^{-}_{s,apoio} \geq A_s(15\% M^{+}_{vão}) \approx 15\%\ A^{+}_{s,vão} \geq A_{s,min} \qquad (5.7)$$

The area of bottom reinforcement providet at supports should be at least 25% of the area of steel provided in the span

$$A^{+}_{s,apoio} \geq 25\%\ A^{+}_{s,vão} \geq A_{s,min} \qquad (5.8)$$

5.5 Minimum area of shear reinforcement

The minimum ratio of shear reinforcement is given by:

$$\rho_{w,min} = \frac{\left(0{,}08\sqrt{f_{ck}}\right)}{f_{yk}} \tag{5.9}$$

Thus the minimum area of shear reinforcement is given by:

$$A_{sw,min=}\rho_{w,min}.\, s.\, b_w.\sin\alpha \tag{5.10}$$

5.6 Shear reinforcement spacing

The maximum longitudinal spacing between shear assemblies is given by:

$$S_{l,max} = 0{,}75d(1 + \cot\alpha\,) \tag{5.11}$$

The transverse spacing of the legs in a series of shear links is given by:

$$S_{t,max} = \min\{0{,}75d, 600\}\ [mm] \tag{5.12}$$

For members with shear reinforcement the additional tensile force ΔF_{td} may be estimated by shifting the moment curve a distance a_l. According Eurocode 2 [1] a_l is given by:

$$a_l = \frac{(\cot\theta - \tan\alpha)}{2} \tag{5.13}$$

6 Example of application

Perform the safety verifications for limit states of a beam simply supported of Figure 6.1 , under the action of self weight, variable action (domestic and residential activities) and other permanent actions the materials to be used are concrete C20/25 steel S400

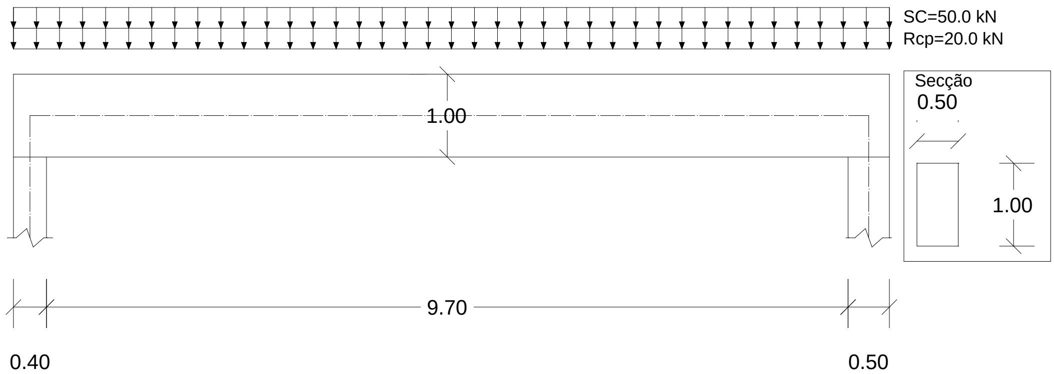


Figure 6.1 Example of application.

6.1.1 Definition of the design model

The necessary input data needed to define the design model are:

Material: C20/25 A400,

Cross section geometry: rectangular, height $h = 1\,m$, width $b = 0{,}5$ considered covers $d1 = d2 = 0{,}04\,m$

Beam geometry: one span, effective span length $l = 10{,}15\,m$ simply supported in on both extremes

Actions: self weight density $\gamma_c = 25\,kN/m^3$; other permanent actions $Rcp = 20\,kN$ variable action $Qk1 =\ 50\,kN$ type A

Load case fully loaded

The design model can now be visualized in program and has the following representation:

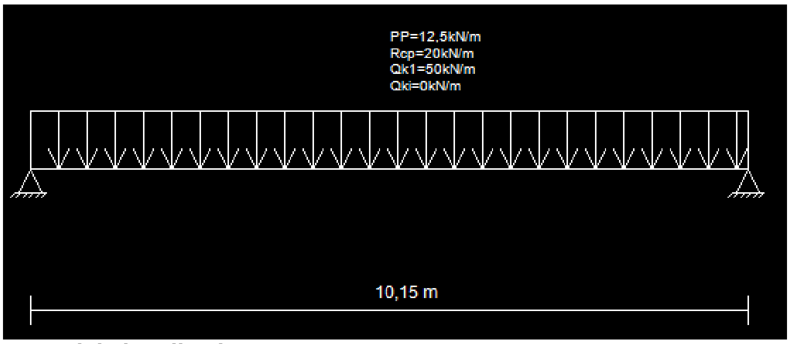


Figure 6.2 Design model visualization.

6.1.2 Analysis of results

It is necessary to define the analysis parameters so that the program may proceed with the limit states verifications. The needed parameters are;

ULS: strut compression angle considered $\theta = 30°$; shear links inclination $\alpha = 90°$; linear analysis

SLS: Combination of action Quasi-permanent; Relative Humidity $RH = 50\%$, age of loading $t_0 = 28\ days$ long term effects $t = \infty$; maximum crack $W_{max} = 0{,}3\ mm$

After the definition of these parameters it is possible to see the results of the analysis carried by the program.

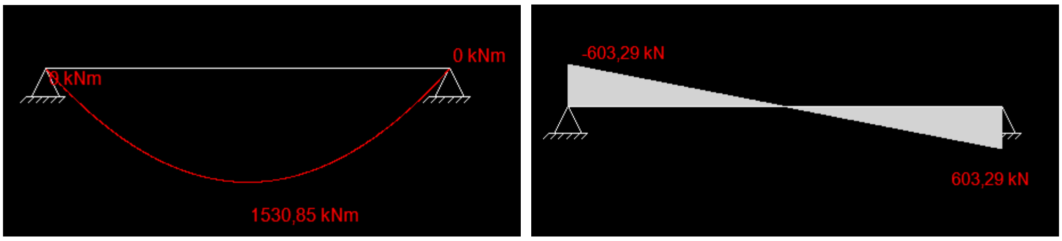
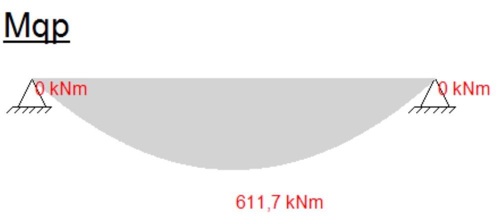


Figure 6.3 Visualization of the moment curve and shear curve

The visualization of the service limit states verifications are



Estados Limites de Deformação

	Viga	K	L/d	L/d Lim	Verificação
▶	Viga_1	1	10,573	13,223	Dispensa

Estados limites de Fendilhação

	Viga	MQp [kNm]	φ [-]	Mcr [kNm]	σs [Mpa]	Sr,máx [m]	εsm-εcm [‰]	Wk [mm]	Wmax [mm]
▶	Viga_1	611,696	2,65	326,866	127,41	0,172	0,497	0,086	Verifica

Figure 6.4 Visualization of service limit states results

Schemes of purposed reinforcement detail obtained by the program are the following being possible to decide among others according to user criteria

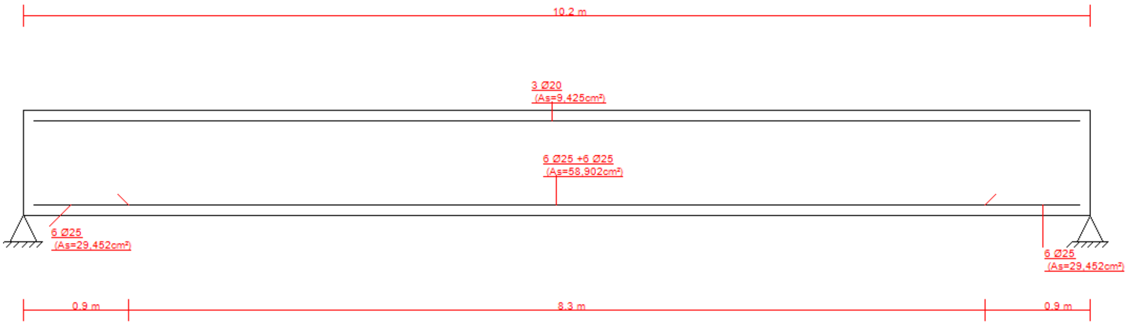


Figure 6.5 Longitudinal reinforcement detail

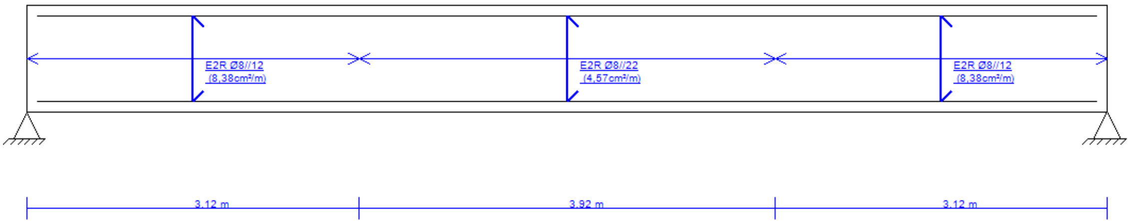


Figure 6.6 Shear reinforcement detail

The engineer should use this detailing as a draft solution that requires revision and the engineer judgment obtained from experience.

7 Conclusions

The objectives purposed for this work to establish the methodologies of atomization of the design process for concrete beams according to Eurocode 2 [1] and develop a program capable of perform this objectives have been achieved. The E2VIGA is capable of perform the safety verification of beams with rectangular and T cross-section submitted to distributed actions being possible to make the analysis for different load cases The verification of safety is made for ultimate limit states for flexure and shear also for cracking and deformation serviceability limit states at the same time reinforcement detailing is made. The program was tested with a manual example where it is shown to be possible adopt more than one valid solution for the problem.

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