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VIS 13

Design Manual

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DISCLAIMER

Given the state of the art and the limits of computer science, CSi Italia cannot guarantee that the program and its manuals are entirely free of errors.

Considerable time, effort and expense have been devoted to the development and testing of this software and the associated documentation, however, by choosing to make use of it, the user agrees and understands that no guarantee of the program accuracy and reliability is expressed or implied by the developer or distributor.

The program is intended only to provide information to the user but not to replace him in his professional judgment. It is the user's responsibility to understand the basic assumptions of the software, compensate for aspects not addressed, and independently verify the results.

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1. General

1.1. PMM strength at ULS

The strength of a concrete section at the Ultimate Limit State in terms of axial force – biaxial moment interaction diagram is calculated by the program based on the selected code and on one of two user-selected procedures: either the *rectangular stress block* or the *parabolic-rectangular stress strain* constitutive model.

The generic concrete section is defined by the section corner coordinates, the concrete properties, and the location, area and steel properties of the reinforcing bars.

The design procedure calculates the ultimate strength of the generic reinforced concrete section relative to a given number of neutral axis locations. These are obtained by varying the distance and the rotation from the section centroid. The ultimate strength values thus obtained belong to the surface of the strength interaction diagram; the 3D surface is subsequently obtained by interpolation in the $\boldsymbol{N}$, $\boldsymbol{M_3}$, $\boldsymbol{M_2}$ space.

The rectangular stress block is built from the neutral axis, where the location of the neutral axis is *x* and the height of the block is $x = \beta \cdot x_c$.

The parabolic-rectangular constitutive model idealizes the stress distribution as a rectangle going from the concrete outermost compression fiber (where concrete strain is ϵ_{cu}) to a location where the strain is ε_{c1}. From there to the neutral axis, the stress diagram is assumed parabolic.

The figure below refers to the parabolic rectangular stress block approach, where the various fields define the strength of a section subject to axial loading and biaxial bending.

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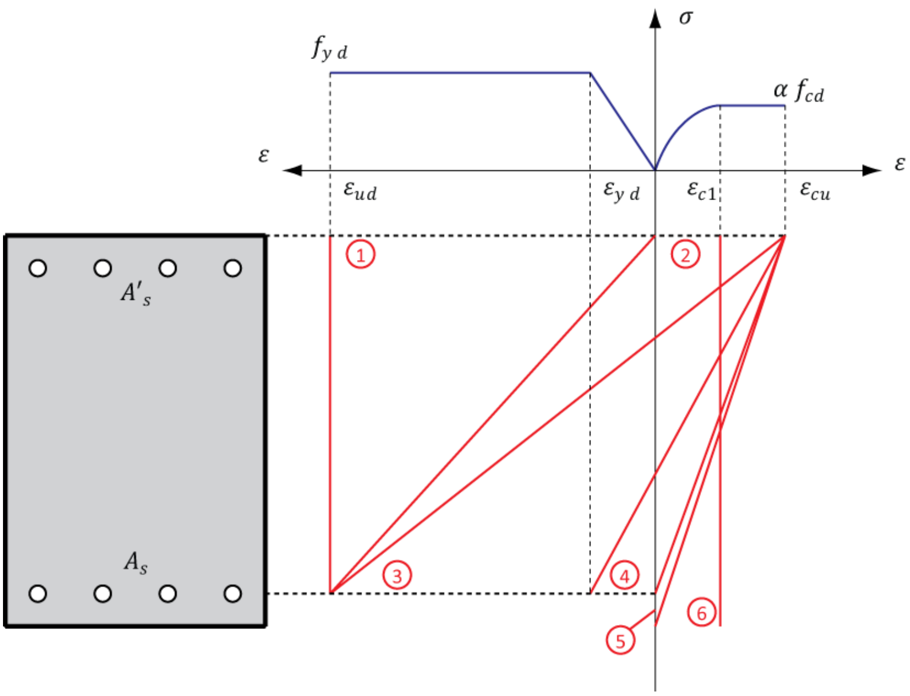
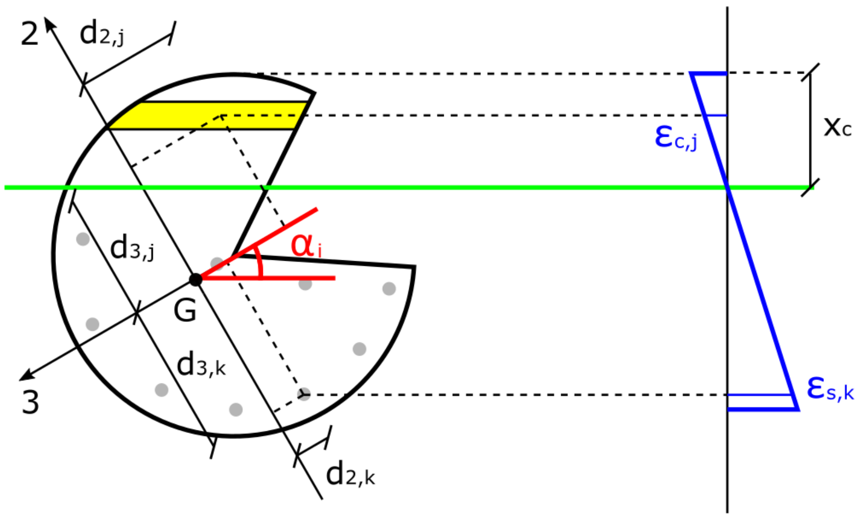


1.1.1. Interaction diagrams

The neutral axis is assigned iteratively several possible locations. At each location the ultimate curvature is then calculated by limiting the concrete compression strain to ϵ_{cu} and the steel strain to ϵ_{su}.



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Finally, the three internal forces are calculated for the limit strength condition:

$$M_{3,i} = -\sum_{j=1}^{n} \sigma_{c,j} \cdot A_{c,j} \cdot d_{2,j} - \sum_{k=1}^{nb} \sigma_{s,k} \cdot A_{s,k} \cdot d_{2,k}$$

$$M_{2,i} = -\sum_{j=1}^{n} \sigma_{c,j} \cdot A_{c,j} \cdot d_{3,j} - \sum_{k=1}^{nb} \sigma_{s,k} \cdot A_{s,k} \cdot d_{3,k}$$

$$N_i = \sum_{j=1}^{n} \sigma_{c,j} \cdot A_{c,j} + \sum_{k=1}^{nb} \sigma_{s,k} \cdot A_{s,k}$$

Where:

$M_{3,i}$	bending moment about x_α axis (x axis rotated by α)
$M_{2,i}$	bending moment about y_α axis (y axis rotated by α)
N_i	Axial force
i	calculation step (neutral axis on lower side of i strip)
α_i	rotation angle
$\sigma_{c,j}$	concrete stress of strip j. If the stress block approach is used the concrete stress is $\alpha_c f_{ck}/\gamma_c$, for strips within the stress block, null if for strips outside. If the parabola rectangle approach is used, the concrete stress varies according to the $\sigma - \varepsilon$ law defined by the code.
$A_{c,j}$	area of strip j
$d_{2,j}, d_{3,j}$	centroid coordinates of strip j
$A_{s,k}$	area of rebar k
$d_{2,k}, d_{3,k}$	centroid coordinates of rebar k
$\sigma_{s,k}$	stress of rebar k (f_{yk}/γ_s or $E_s\varepsilon_{s,k}$ depending on the rebar deformation as compared to the steel yielding deformation)

Gamma factors and yield point deformation values depend upon the selected code. They can also be user assigned.

Any subsequent iteration generates a set of internal forces in the $\boldsymbol{N}$, $\boldsymbol{M_3}$, $\boldsymbol{M_2}$ space, defining a point on the surface of the interaction diagram.

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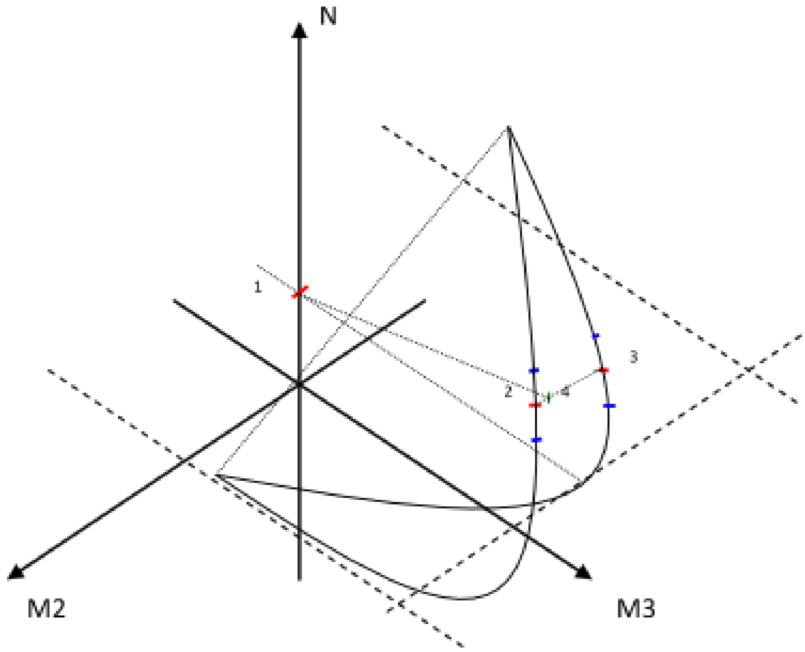
1.1.2. Interpolation

The procedure above generates a cloud of points in the $\boldsymbol{N}$, $\boldsymbol{M_3}$, $\boldsymbol{M_2}$ space, representing the internal forces of the section, at ultimate strength, for a discrete number of locations of the neutral axis. Using conservative interpolation techniques, these points are used to generate planar curves in the planes $\boldsymbol{M}-\boldsymbol{N}$ and $\boldsymbol{M_3}-\boldsymbol{M_2}$. The curves thus obtained are used for all subsequent strength checks.



1.1.3. Demand Capacity ratios

Demand Capacity ratios are “radial”. They are calculated in the $\boldsymbol{N}$, $\boldsymbol{M_3}$, $\boldsymbol{M_2}$ space along a radial segment. This segment starts from the origin, goes through the force, and continues to the surface of the interaction diagram.

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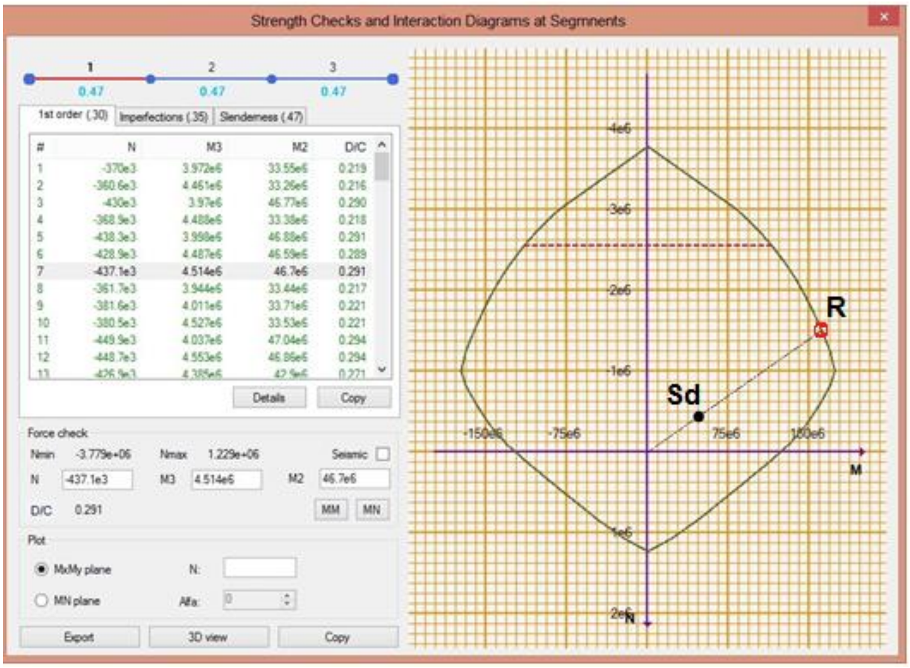

If the design forces are represented by point Sd and the corresponding section strength is represented by point R, where Sd and R are on the same N-M plane, the corresponding D/C value is:

$$D/C = \frac{|N_{Sd}, M_{3,Sd}, M_{2,sd}|}{|N_R, M_{3,R}, M_{2,R}|}$$

1.2. Elastic PMM domain

Certain codes sometimes impose to limit the internal forces in the elements so that they remain in the "pseudo-elastic" field, which means below the yielding point of the steel and of the concrete in compression. This is the case, for example, of structures designed according to a "non-dissipative" seismic behavior, in which the yielding of the elements should be prevented due to the lack of detailing for the local ductility.

The procedure used to calculate the elastic domain is substantially similar to the one related to the ULS domain, with the difference that the ultimate curvature corresponding to the i-th position of the neutral axis is limited to the first yielding of the concrete in compression or the reinforcing.

1.3. Imperfections and second order effects

In the design of columns, VIS accounts for both imperfections and second order effects. Three different type of PMM checks are performed:

- *Analysis* – The design forces comes directly from the analysis. The checks are performed for every load combinations and for every output station along the column.
- *Imperfections* – The design moments are amplified to account for the effect of geometrical imperfections and then compared with the code minimum design moments. The checks are performed for each output station along the element but only for the load combinations for which the column is under compression.
- *Slenderness* – The design moments are amplified to account for second order effects and then compared with the code minimum design moments. The checks are performed only for slender columns under compression.

1.3.1. Effects of imperfections

For each column under axial compression, the effects of imperfections are accounted by means of additional moments defined as:

$$M_i^{maj} = N_{Sd} \cdot e_i^{maj}$$

$$M_i^{min} = N_{Sd} \cdot e_i^{min}$$

where

M_i^{maj}, M_i^{min}	additional moments due to imperfections about the principal directions of the column
e_i^{maj}, e_i^{min}	eccentricities due to imperfections along the principal directions of the column
N_{Sd}	design axial force corresponding to the present load combination

All the possible permutations of eccentricity are considered for each principal direction separately. Thus, for every station (s) and every load combination (C), four different cases are checked:

$$C(s) \rightarrow \begin{cases} \left(N_{Sd}; \quad M_{Sd}^{maj} + N_{Sd} \cdot e_i^{maj}; \quad M_{Sd}^{min}\right) \\ \left(N_{Sd}; \quad M_{Sd}^{maj} - N_{Sd} \cdot e_i^{min}; \quad M_{Sd}^{min}\right) \\ \left(N_{Sd}; \quad M_{Sd}^{maj}; \quad M_{Sd}^{min} + N_{Sd} \cdot e_i^{min}\right) \\ \left(N_{Sd}; \quad M_{Sd}^{maj}; \quad M_{Sd}^{min} - N_{Sd} \cdot e_i^{min}\right) \end{cases}$$

where

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$M_{Sd}^{maj}, M_{Sd}^{min}$ are the analysis moments rotated in the principal reference system of the section

These moments are finally compared with the code minimum design moments and then rotated back to the local 2-3 reference system of the section.

If the effects of imperfections have already been included in the analysis, the user can disable the calculation of the additional moments inside the window "General settings > Strength design > Imperfections". In this case the analysis moments acting about the principal directions will simply be compared with the minimum design moments and then rotated back to the local 2-3 system.

The results of these checks are reported in the "Imperfections" tab of the window "Strength > Check PMM Frame". Further details will be available by clicking on the "Details" button.

1.3.2. Second order effects

If the analysis type is flagged as "1st order" or as "2nd order with P-Δ effects" element's internal forces will be amplified to account for slenderness effects. On the contrary, if the analysis type is "2nd order with P-Δ and P-δ effects", the internal forces will not be further incremented. The analysis type can be set inside the "General settings > Strength design > Second order" window.

Second order effects are calculated with reference to a condition of uniform bending about the principal axes of the column, due to equivalent first order moments, M_0, including the effects of imperfections.

If imperfections have not been included in the analysis, for each load combination four different configurations of equivalent moments are considered:

$$C \rightarrow \begin{cases} C\#1 & \left(M_0^{maj};\quad M_0^{min}\right) = \left(M_{0,Sd}^{maj} + N_{Sd} \cdot e_i^{maj};\quad M_{0,Sd}^{min}\right) \\ C\#2 & \left(M_0^{maj};\quad M_0^{min}\right) = \left(M_{0,Sd}^{maj} - N_{Sd} \cdot e_i^{maj};\quad M_{0,Sd}^{min}\right) \\ C\#3 & \left(M_0^{maj};\quad M_0^{min}\right) = \left(M_{0,Sd}^{maj};\quad M_{0,Sd}^{min} + N_{Sd} \cdot e_i^{maj}\right) \\ C\#4 & \left(M_0^{maj};\quad M_0^{min}\right) = \left(M_{0,Sd}^{maj};\quad M_{0,Sd}^{min} - N_{Sd} \cdot e_i^{maj}\right) \end{cases}$$

where

M_0^{maj}, M_0^{min} are the equivalent first order moments about the principal directions

$M_{0,Sd}^{maj}, M_{0,Sd}^{min}$ are the equivalent first order moments about the principal directions calculated with reference to the analysis moments

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Otherwise, if the effects of geometric imperfections have already been included in the analysis, the equivalent first order moments are directly calculated with reference to the analysis moments:

$$C \rightarrow \left(M_0^{maj}; \quad M_0^{min}\right) = \left(M_{0,Sd}^{maj}; \quad M_{0,Sd}^{min}\right)$$

Once the equivalent first order moments have been determined, slenderness checks are performed and, eventually, additional second order moments are calculated. The additional moments are computed for the slender directions only and are expressed by the following equations:

$$M_2^{maj} = N_{Sd} \cdot e_2^{maj}$$

$$M_2^{min} = N_{Sd} \cdot e_2^{min}$$

where

M_2^{maj}, M_2^{min}	are the second order moments about the principal directions
e_2^{maj}, e_2^{min}	are the second order eccentricities along the principal directions
N_{Sd}	design axial force corresponding to the present load combination

The resulting moments are finally compared with the code minimum design moments and then rotated back to the local 2-3 reference system of the section.

The results of these checks are reported in the “Slenderness” tab of the window Strength > Check PMM Frame. Further details will be available by clicking on the “Details” button.

Second order effects are calculated for generic sections and for arbitrary rotations. The only simplifications adopted are the following:

- columns are considered to be loaded only at ends;
- the axial load is assumed constant along the element (maximum value is used);
- the size of the section remains the same throughout the column.

With reference to slender column design, most code provisions refer to rectangular sections as the basis of their theoretical approach. Not much information is provided for sections that are not rectangular; hence, some approximate design approach is required in order to meet code provisions for the general case.

In particular, most Code formulas make use of the notion of section width and height. To expand this concept to sections that are not rectangular, VIS applies the bounding box concept. Referring to the Principal Axes:

- height h is assumed to be the section max dimension in the Minor Direction;

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- width b is assumed to be the section max dimension in the Major Direction.

1.3.3. Effective lengths

Column effective lengths are calculated as the product of column length by the corresponding effective length factors K:

$$L_{cr}^{maj} = K^{maj} \cdot L$$

$$L_{cr}^{min} = K^{min} \cdot L$$

Effective length factors are automatically calculated by the software according to the code provisions. The calculation algorithms are based on:

- the relative flexibilities of rotational restraints at the ends of the column;
- the type of the structure (sway or not sway);
- the analysis type.

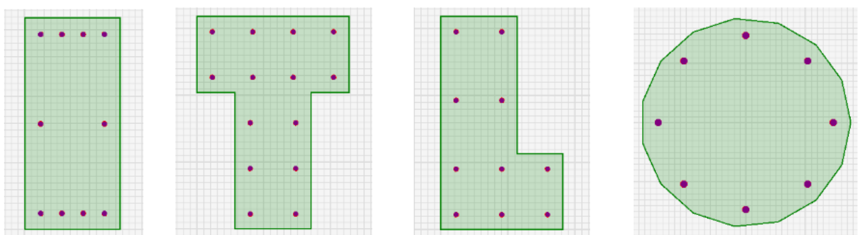
For sway frames where the analysis type is "1st order", the effective length factors will always be greater than 1; while for non sway frames or when the analysis type is "2nd order" the effective length factors are always lower or equal than 1.

The user has always the possibility to overwrite the values of the K factors inside the menu "Define > Frames > Columns"; or by selecting the columns and going in the "Edit > Slenderness factors" tab.

1.4. Shear

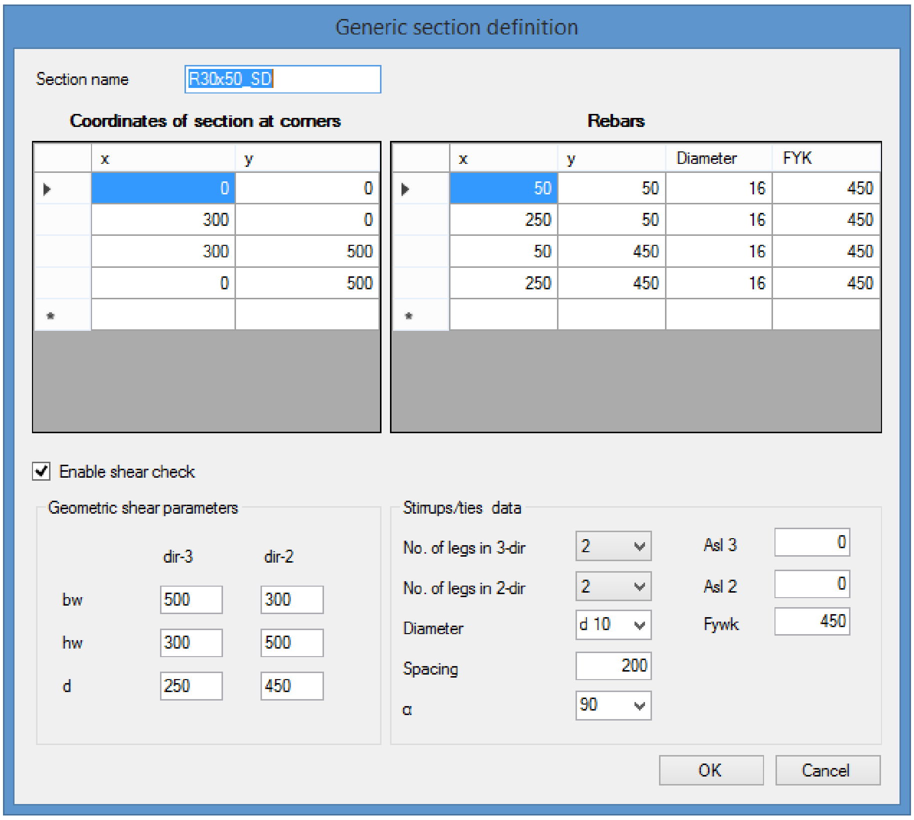
1.4.1. Section types

Shear checks are fully automated for the following type of sections: Rectangular, L shape, T shape and Circular. Reference axes are the local 2 and 3 axes.



Shear strength is calculated for each local direction and compared with the design shear. For L and T shapes, shear strength along the 2 direction is calculated referring to web strength, along the 3 directions referring to flange strength.

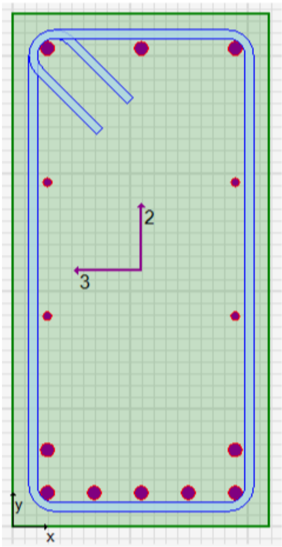
Any generic section (defined assigning corner coordinates) can be checked against shear by analogy with an equivalent rectangular section. By checking the option "Enable shear check" in the section's definition window, it will then be possible to specify the geometric and reinforcing data needed for the design.

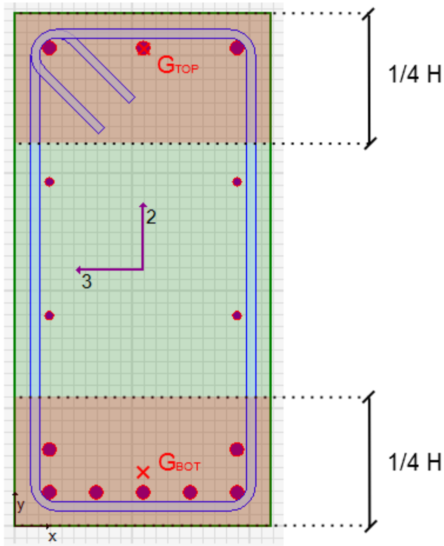


1.4.2. Section effective depth

The program calculates the effective depth as the distance from the outermost compression fiber to the centroid of the tension reinforcement, where the tension reinforcement includes all rebars on the tension side of the section, starting from las quarter of the section.

Where the effective depth varies in the two opposite directions, for economy of calculation effort, only the shortest depth is considered. The only exception is the case where reinforcing is entirely missing on one side.





For generic sections, effective heights are taken from the section definition window.

1.4.3. Circular sections

The Eurocodes do not have specific provisions for circular sections. VIS implements a design approach similar to the approach recommended in ACI 318.

The shear strength of the circular section is assumed as the strength of an equivalent section, having the following properties:

- the section is assumed to be square, with sides $l = \sqrt{\pi} \cdot r$
- the effective depth d is calculated as from the previous paragraph
- the reinforcing area $\mathrm{A_v}$ is twice the area of the circular tie or spiral

1.4.4. Check

The design procedure includes the following steps:

1. Based on the reinforcing layout, calculate the section effective depth d
2. Calculate the section strength components:
   – $V_{Rd,c}$ unreinforced concrete strength
   – $V_{Rd,max}$ strength of compression strut
   – $V_{Rd,s}$ strength of shear reinforcement
3. With reference to the design shear $\mathrm{V_{Ed}}$, perform the following checks:

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- If unreinforced and $V_{Ed} > V_{Rd,c}$ then section is not adequate
- If reinforced and $V_{Ed} > V_{Rd,max}$ then section is not adequate
- If reinforced and $V_{Ed} > V_{Rd}$ then section is not adequate
- Otherwise section is adequate.

4. D/C ratio are then calculated for all the combos.

1.5. Serviceability limit states

1.5.1. Stress limitation

These checks are based on the typical working stress assumptions for concrete design:

- sections remain planar;
- concrete is compression only, with linear compression behavior;
- steel has linear behavior both in tension and compression;
- n=Es/Ec is the modular ratio of steel to concrete.

The location of the neutral axis is calculated using an iterative algorithm, for any PMM load combination. Concrete and steel stresses are checked against code allowed maxima.

1.5.2. Decompression

Based on a linear stress distribution, these calculations provide a close form solution to detect if tension was reached in the concrete section.

1.5.3. Crack formation

Based on a linear stress distribution as above, these calculations provide a close form solution to detect if the crack formation limit was reached in the concrete section.

1.5.4. Crack opening

Crack opening calculations are based on stresses reached by the reinforcing bars, assuming the section is cracked and concrete is compression only. The procedure is based on direct calculation of crack opening defined by the design code. Checks are performed at each single rebar location in the tension zone.

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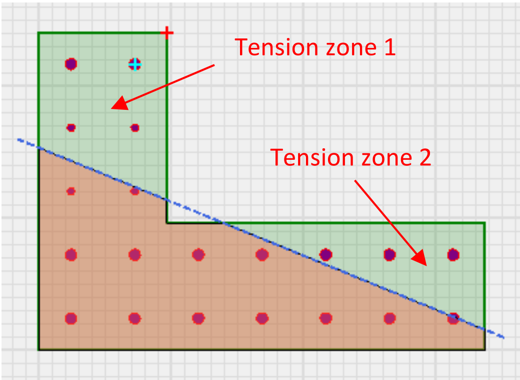

1.6. Walls

1.6.1. PMM strength

Axial force – biaxial moment strength design checks are performed as previously described for beams and columns. The program calculates the wall interaction diagram and the Demand Capacity ratios for the various loading conditions. No reduction is applied to the wall resisting section.

1.6.2. Shear strength

Shear walls that are planar, i.e. have a single leg, have strength calculations in the strong direction based on the amount of horizontal reinforcing. By default, the weak direction strength is based on concrete only. The contribution of reinforcing in the weak direction can be included, if desired, by clicking the corresponding check box in the wall definition window. If desired, shear check in the weak direction can be completely ignored by changing the preferences in the wall definition window.

For multiple leg shear walls, the shear design can follow two different approaches:

- if, when creating the section cuts, the option “Create separate legs for shear check” has been activated, the checks will be performed separately for each leg with reference to the corresponding shear force. The procedure will be the same presented earlier for the planar walls.
- Alternatively, if the option “Create separate legs for shear check” has not been checked, the checks will be performed with reference to the shears action on the whole section. In this case, the resulting strength will be calculated as the sum of

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the strengths of the single legs projected in the direction under consideration. Weak direction contributions from each leg are disregarded.

2. Beam and column design according to EC2 2005/EC8 2005

2.1. Strength

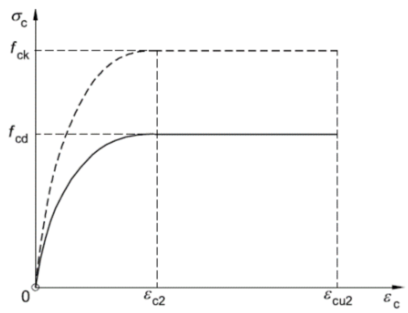
2.1.1. Axial force and biaxial bending

The PMM strength checks are performed with reference to the factored internal forces from imported load cases and/or combinations.

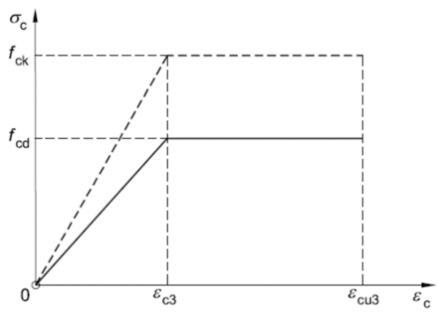
Complete 3D interaction surfaces are computed for each section.

Of the three suggested code procedures, VIS implements two: either the *rectangular stress block* or the *parabolic-rectangular stress block*.

a. *Rectangle-parabola stress-strain distribution*



b. *Bilinear stress-strain distribution*



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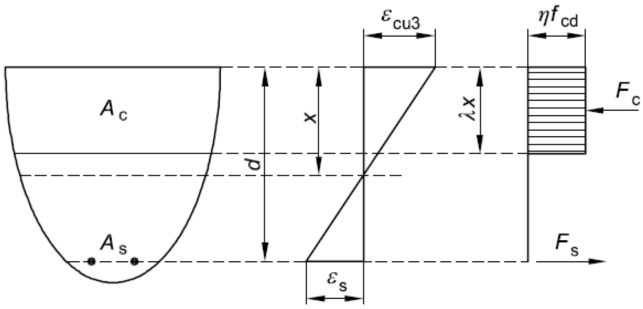
c. *Stress Block distribution*



For concrete strengths $f_{ck} < 50$ MPa:

$$\varepsilon_{cu2} = \varepsilon_{cu3} = 3.5‰$$

$$\varepsilon_{c2} = 2.00‰$$

$$\lambda = 0.8$$

$$\eta = 1$$

For higher strengths:

$$\varepsilon_{cu2} = \varepsilon_{cu3} = 2.60‰ + 35‰[(90 - f_{ck})/100]^4$$

$$\varepsilon_{c2} = 2.00‰ + 0.085‰(f_{ck} - 50)^{0.53}$$

$$\lambda = 0.8 - (f_{ck} - 50)/400$$

$$\eta = 1$$

The stress reduction factor for long term loading, α, is defined as:

$\alpha = 0.85$ if $f_{ck} < 50$ MPa

$\alpha = 0.85\left(1 - \frac{f_{ck}-50}{200}\right)$ otherwise

The value of α can be overwritten in the “Strength” preferences from the “General settings” Tab.

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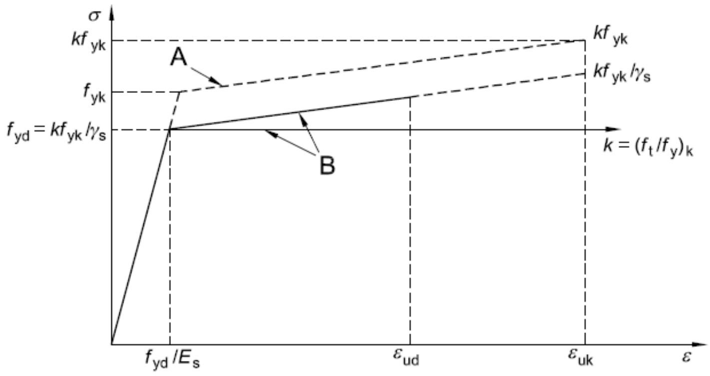
Based on Code provisions, the behavior of reinforcing steel is implemented by the program using an elastic-perfectly plastic stress-strain diagram.



Steel elastic modulus E_s= 200.000 N/mm^2

2.1.2. Effects of imperfections

For all columns subject to axial loading, an additional eccentricity due to imperfection is computed:

$e_i = l_0/400$	for non sway structures
$e_i = \theta_i \cdot l_0/2$	for sway structures

where

$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m$	
l_0	effective length in the present principal direction
$\theta_0 = 0.005$	basic value of geometric imperfection
$\alpha_h = 2/\sqrt{l}$	reduction factor for length $(2/3 \leq \alpha_h \leq 1)$
$\alpha_m = \sqrt{0.5 \cdot (1 + 1/m)}$	reduction factor for number of members

The corresponding moments ($N_{Ed}e_i$) are added to the analysis moments rotated in the principal directions. All the possible permutations of eccentricity are considered for each principal direction separately. Thus, for every station (s) and every load combination (C), four different cases are checked.

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The resulting moments are then compared with the minimum design moments (the higher value is used in the design) and rotated back in the local 2-3 reference system:

$$M_{min} = N_{Ed} e_{min}$$

where

$$e_{min} = max\left(\frac{h}{30}; 20mm\right)$$

h = cross section height

The results of these checks are reported in the "Imperfections" tab of the window Strength > Check PMM Frame. Further details will be available by clicking on the "Details" button.

2.1.3. Second order effects

Second order effects are calculated with reference to a condition of uniform bending about the principal axes of the column, due to equivalent first order moments, M_0, including the effects of imperfections:

$$M_{0Sd}^{maj} = 0.6 \cdot M_{02}^{maj} + 0.4 \cdot M_{01}^{maj} \geq 0.4 \cdot M_{02}^{maj}$$

$$M_{0Sd}^{min} = 0.6 \cdot M_{02}^{min} + 0.4 \cdot M_{01}^{min} \geq 0.4 \cdot M_{02}^{min}$$

where

$M_{01}; M_{02}$	with $\lvert M_{02}\rvert \geq \lvert M_{01}\rvert$. Bending moments (including imperfections) acting at the ends of the column in the present principal direction

Second order effects are a function of the slenderness of the column:

$$\lambda^{maj} = \frac{l_0^{maj}}{i^{maj}}$$

$$\lambda^{min} = \frac{l_0^{min}}{i^{min}}$$

where

$l_0^{maj}; l_0^{min}$	effective lengths about the principal directions
$i^{maj}; i^{min}$	radii of gyration of the uncracked concrete section

These slenderness are compared with the limit values defined by the code:

$$\lambda_{lim}^{maj} = 20 \cdot A \cdot B \cdot C^{maj} \cdot \frac{1}{\sqrt{n}}$$

$$\lambda_{lim}^{min} = 20 \cdot A \cdot B \cdot C^{min} \cdot \frac{1}{\sqrt{n}}$$

where

$A = 1/(1 + 0.2 \cdot \varphi_{ef})$

φ_{ef} effective creep ratio (default value 2.1429)

$B = \sqrt{1 + 2\omega}$

$\omega = A_s f_{yd}/(A_c f_{cd})$ mechanical reinforcement ratio

$C = 1.7 - r_m$

$r_m = M_{01}/M_{02}$ with $|M_{02}| \geq |M_{01}|$. Ratio between the end moments (including imperfections) acting along the principal directions

$n = N_{Ed}/(A_c \cdot f_{cd})$

Along the slender directions, where $\lambda > \lambda_{lim}$, the equivalent bending moments are amplified by an additional eccentricity e_2:

$$M_{Sd} = N_{Sd} \cdot [e_0 + \mathrm{sign}(e_0) \cdot e_2]$$

where:

$e_0 = M_{0Sd}/N_{Sd}$ is the equivalent first order eccentricity

e_2 is the second order eccentricity

For the determination of the second order eccentricity the code allow the use of two different approaches: “nominal stiffness method” (EC2 5.8.7) or “nominal curvature method” (EC2 5.8.8). VIS actually implements both the methods and the user has the option to select the one to use.

Method based on nominal stiffness

The second order eccentricities are defined as:

$$e_2^{maj} = e_0^{maj} \cdot \frac{\beta}{\frac{N_B^{maj}}{N_{Ed}} - 1}$$

$$e_2^{min} = e_0^{min} \cdot \frac{\beta}{\frac{N_B^{min}}{N_{Ed}} - 1}$$

where

$\beta = \pi^2/c_0$

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c_0	coefficient which depends on the distribution of the first order moment (default value is 8)
$N_B = \pi^2 EI/l_0^2$	buckling load along the present principal direction based on nominal stiffness
N_{Ed}	design axial force

The nominal stiffness to be used for the calculation of the buckling loads are defined by the following equations:

$$(EI)^{maj} = K_c^{maj} \cdot E_{cd} \cdot I_c^{maj} + K_s \cdot E_s \cdot I_s^{maj}$$

$$(EI)^{min} = K_c^{min} \cdot E_{cd} \cdot I_c^{min} + K_s \cdot E_s \cdot I_s^{min}$$

in cui

$K_c = k_1 k_2/(1 + \varphi_{ef})$	factor for effects of cracking and creep
$k_1 = \sqrt{f_{ck}/20}$	factor which depends on concrete strength class
$k_2 = n \cdot \lambda/170 \leq 0.2$	factor which depends on axial force and slenderness
$E_{cd} = E_{cm}/\gamma_{cE}$	design value of the modulus of elasticity of concrete
I_c	moment of inertia of concrete cross section
$K_s = 1$	
$E_s = 200000\ MPa$	modulus of elasticity of reinforcement
I_s	moment of inertia of reinforcement with reference to the centroid of the concrete section

Method based on nominal curvature

The second order eccentricities are defined as:

$$e_2^{maj} = \frac{1}{r^{maj}} \cdot \left(\frac{{l_0^{maj}}^2}{c}\right)$$

$$e_2^{min} = \frac{1}{r^{min}} \cdot \left(\frac{{l_0^{min}}^2}{c}\right)$$

where

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$1/r = K_r \cdot K_\varphi \cdot 1/r_0$	curvature along the present principal direction
$K_r = (n_u - n)/(n_u - n_{bal}) \leq 1$	correction factor depending on axial load
$n_u = 1 + \omega$	
n	relative axial force
n_{bal}	relative axial force at maximum moment resistance
$K_\varphi = 1 + \beta\varphi_{ef} \geq 1$	correction factor depending on creep effects
$\beta = 0.35 + f_{ck}/200 - \lambda/150$	
$1/r_0 = \varepsilon_{yd}/(0.45 \cdot d)$	basic curvature along the present principal direction
$\varepsilon_{yd} = f_{yd}/E_s$	

The default and suggested method is the “nominal stiffness”, however the user has the option to select the method to use in the window General Settings > Strength design > Second order.

The resulting moments are finally compared with the code minimum design moments and then rotated back to the local 2-3 reference system of the section.

The results of these checks are reported in the “Slenderness” tab of the window Strength > Check PMM Frame. Further details will be available by clicking on the “Details” button.

If both global and local second order effects have already been accounted in the analysis, it is possible to exclude the calculation of the additional moments by selecting “2nd order with P-Δ and P-δ effects” in the window General Settings > Strength design > Second order.

2.1.4. Shear

Shear strength checks are performed with reference to the factored internal forces from imported load cases and/or combinations.

The strength of members without shear reinforcement is as follows:

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k(100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d$$

$$V_{Rd,c} \geq \left(v_{min} + 0.15\sigma_{cp}\right) \cdot b_w d$$

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When the average tension stress is greater than $\boldsymbol{f_{ctd}}$, the shear resistance is assumed zero.

The strength of members with shear reinforcement is calculated according to the variable strut inclination method:

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

In order to prevent crushing of the concrete compression struts, the strength is also limited by the maximum sustainable shear force:

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \upsilon_1 \cdot f_{cd} \cdot \frac{(cot\,\theta + cot\,\alpha)}{(1 + cot^2\,\theta)}$$

Where the inclination of the concrete struts, θ, is limited as follow:

$$1 \le cot\,\theta \le 2.5$$

The effective shear resistance is given by:

$$V_{Rd} = min\left(V_{Rd,s}; V_{Rd,max}\right)$$

2.2. Capacity

To avoid brittle behavior, structures designed for ductility class high (DCH) or medium (DCM) need to comply with additional seismic provisions, as prescribed from Eurocode 8. VIS implementation is the following.

2.2.1. DCM shear at beams

In primary seismic beams, the design shear forces $\boldsymbol{V_{Ed}}$ are assessed in accordance with the capacity design rule. The forces are calculated from beam equilibrium under transverse loading at seismic conditions and with end moments corresponding to the positive and negative plastic hinge formation.

$$V_{Ed} = \gamma_{Rd} \frac{\sum_{i=1}^{2} M_{i,d}}{l} + V_g$$

where:

$$\gamma_{Rd} = 1.0$$

$$M_{i,d} = M_{Rb,i} min\left(1; \frac{\sum M_{Rc}}{\sum M_{Rb}}\right)$$

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$M_{Rb,i}$	resisting moment of beam at end i
$\sum M_{Rc}$	sum of the resisting moments of columns framing on end i
$\sum M_{Rb}$	sum of the resisting moments of the beams framing on end i
V_g	shear due to transverse seismic loads

All possible sign permutations are considered.

2.2.2. DCH shear at beams

Same as DCM, with:

$$\gamma_{Rd} = 1.3$$

2.2.3. DCM shear at columns

In primary seismic columns, the design shear forces are assessed in accordance with the capacity design rule. These forces are calculated based on column equilibrium under end moments $M_{1,d}$ and $M_{2,d}$, corresponding to the plastic hinge formation in both directions.

$$V_{Ed} = \gamma_{Rd} \frac{\sum_{i=1}^{2} M_{i,d}}{l}$$

where:

$$\gamma_{Rd} = 1.1$$

$$M_{i,d} = M_{Rc,i} min\left(1; \frac{\sum M_{Rb}}{\sum M_{Rc}}\right)$$

$M_{Rc,i}$	resisting moment of column at end i
$\sum M_{Rc}$	sum of the resisting moments of columns framing on end i
$\sum M_{Rb}$	sum of the resisting moments of the beams framing on end i

All possible sign permutations are considered.

2.2.4. DCH shear at columns

Same as DCM , with:

$$\gamma_{Rd} = 1.3$$

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2.2.5. DCM axial force at columns

In primary seismic columns, the value of the normalized axial force is not allowed to exceed 0.65.

2.2.6. DCH axial fore at columns

In primary seismic columns, the value of the normalized axial force is not allowed to exceed 0.55.

2.2.7. DCM weak beam - strong column condition

In multi-story buildings, Eurocode 8 recommends prevention of soft story mechanisms. To satisfy this requirement, in frame buildings and frame-equivalent buildings, having two or more stories, the weak beam-strong column condition is enforced. In accordance, at all joints where beams frame into primary seismic columns, the column reinforcing designed by the program complies with the following:

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb}$$

where:

$\sum M_{Rc}$	sum of the design values of the resisting moments of the columns framing into the joint. The resisting moments are the lowest corresponding to the full range of axial forces from seismic loading
$\sum M_{Rb}$	sum of the design values of the resisting moments of the beams framing in the joint
γ_{Rd}	1.3

All possible sign permutations are considered.

2.2.8. DCH weak beam - strong column condition

Same as DCM.

2.2.9. DCH shear at joints

To ensure adequate shear strength to beam-column joints, the program calculates the shear forces that could act in the joint, based on the beam reinforcing, and makes sure that the joint shear strength is adequate. The joint section is assumed

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the same as that of the bottom column framing into it. Only joints having a bottom column are considered.

Where primary seismic beams and columns frame into each other, the horizontal shear acting on the core of the joint is calculated accounting for the most adverse seismic conditions from the beams, and the lowest compatible values of shear forces in the columns.

$$V_{jbd} = \gamma_{Rd}(A_{s1} + A_{s2})f_{yd} - V_C \qquad \text{at interior joints}$$

$$V_{jbd} = \gamma_{Rd}A_{s1}f_{yd} - V_C \qquad \text{at exterior joints}$$

where:

$\gamma_{Rd} = 1.2$

A_{s1}, A_{s2} top and bottom reinforcement areas of the beams

V_C shear force in the column above the joint, from analysis with seismic loading

All possible sign permutations are considered.

The joint shear capacity is determined by a strut and tie mechanism. The diagonal compression induced in the joint is checked not to exceed the compressive strength of concrete in the presence of transverse tensile strains. The program applies the following:

$$V_{jbd} \le \eta f_{cd} b_j h_{jc} \sqrt{1 - \frac{\upsilon_d}{\eta}}$$

where:

$\eta = 0.6\left(1 - \frac{f_{ck}}{250}\right)$ at interior joints

$\eta = 0.48\left(1 - \frac{f_{ck}}{250}\right)$ at exterior joints

f_{ck} characteristic compressive strength of concrete (MPa)

υ_d normalized axial force in the column above the joint

h_{jc} distance between the outermost layers of column reinforcement

$$b_j = \begin{cases} min(b_c; b_w + 0.5 \cdot h_c) & if\ b_c > b_w \\ min(b_w; b_c + 0.5 \cdot h_c) & if\ b_w > b_c \end{cases}$$

b_c column width

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b_w beam width

To limit the diagonal tensile stress of concrete to f_{ctd}, adequate confinement of the joint could be provided. The program applies the following:

$$\frac{A_{sh} f_{ywd}}{b_j h_{jw}} \geq \frac{\left[V_{jbd}/b_j h_{jc}\right]^2}{f_{ctd} + \nu_d f_{cd}} - f_{ctd}$$

where:

A_{sh} total area of horizontal hoops

h_{jw} distance between outermost layers of beam reinforcement

As an alternate option, the integrity of the joint could be ensured by horizontal hoop reinforcement, after diagonal cracking has occurred. To this purpose, the following total area of horizontal hoops is required by the program:

$$A_{sh} f_{ywd} = \gamma_{Rd}(A_{s1} + A_{s2}) f_{yd} \cdot (1 - 0.8\upsilon_d) \quad \text{at interior joints}$$

$$A_{sh} f_{ywd} = \gamma_{Rd} A_{s1} f_{yd} \cdot (1 - 0.8\upsilon_d) \quad \text{at exterior joints}$$

Where the normalized axial force,υ_d, refers to the upper column for interior joints, to the lower column for exterior joints.

2.3. Serviceability

2.3.1. Stress limitation

Concrete compressive stress should be limited in order to avoid longitudinal cracks, micro-cracks or high levels of creep, when these occurrences result unacceptable for the proper performance of the structure. In accordance with EC2, the program can check the compressive stress of concrete for the following limits:

$\sigma_c \leq k_1 f_{ck}$ with characteristic load combinations

$\sigma_c \leq k_2 f_{ck}$ with quasi-permanent load combinations

Reinforcement tensile stress should be limited in order to avoid inelastic strain, unacceptable cracking or deformation. In accordance with EC2, the program can apply the following limits to the tensile stress of steel:

$\sigma_s \leq k_3 f_{yk}$ with characteristic load combinations

k_1, k_2 e k_3 can be assigned by user. The default values are respectively 0.6, 0.45 and 0.8, as recommended by the Code.

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2.3.2. Crack control

Cracking should be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. VIS check is carried out with reference to characteristic load combinations, and the user h specify the limit state to be controlled.

Decompression

The program checks the axial stresses acting in the section to be negative or, at least, equal to zero.

Crack formation

The program checks that maximum axial stress acting in the concrete meets the following:

$$\sigma_c \leq \frac{f_{ctm}}{1.2}$$

Crack opening

The program checks the expected crack opening to be lower than w_{max}.

The limit value should be specified by the user taking into account the exposure class of the building and other significant parameters.

The expected crack opening is estimated through the algorithm described in EC2 § 7.3.4.

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm})$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \frac{f_{ctm}}{\rho_{eff}}(1 + \alpha_e \rho_{eff})}{E_s} \geq 0.6 \frac{\sigma_s}{E_s}$$

where:

σ_s	stress in the tension reinforcement calculated assuming a cracked section
$k_t = 0.4$	for long term loading
f_{ctm}	mean tensile strength of concrete
$\rho_{eff} = \frac{A_s}{A_{c,eff}}$	reinforcement ratio in the effective tension area

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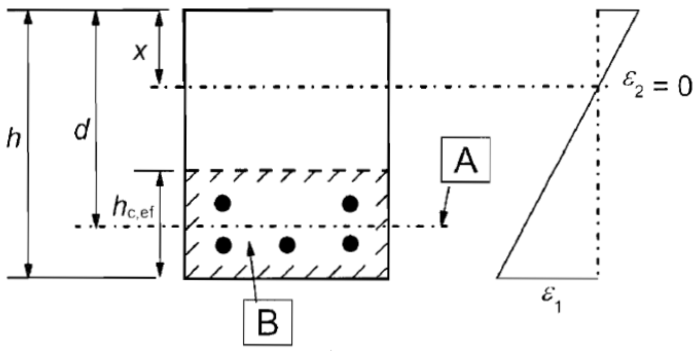
$$\alpha_e = \frac{E_s}{E_{cm}}$$

The depth of the effective tension area ($h_{c,ef}$) is calculated as follows:

$$h_{c,ef} = \min\left\{2.5\cdot(h-d); \frac{h-x}{3}; \frac{h}{2}\right\}$$



$$\Delta_{smax} = \begin{cases} k_3 c + k_1 k_2 k_4 \emptyset/\rho_{eff} & if \quad s \le c\cdot(5+\emptyset/2) \\ 1.3(h-x) \ \text{otherwise} \end{cases}$$

Con

$k_3 = 3.4$

c cover to the longitudinal reinforcement

$k_1 = 0.8$

$$k_2 = \frac{\varepsilon_1 + \varepsilon_2}{2\varepsilon_1}$$

$\varepsilon_1 \ge 0$ greater tensile strain at the boundaries of the section

$\varepsilon_2 \ge 0$ lower tensile strain at the boundaries of the section

$k_4 = 0.425$

$$\emptyset = \frac{\sum_i n_i \emptyset_i^2}{\sum_i n_i \emptyset_i}$$

s spacing of the tensile reinforcement inside the effective tension area

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2.4. Detailing provisions

For each beam and column in the model, the following detailing provisions are checked by the program. A detailed report provides results.

2.4.1. Beams

EC2 §9.2.1 – Longitudinal reinforcing

- The area of longitudinal tension reinforcement should not be taken as less than:

$$A_{s,min} = max\left\{0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t d; 0.0013 \cdot b_t d\right\}$$

- Outside lap splice locations, the cross-sectional area of tension or compression reinforcement should not exceed:

$$A_{s,max} = 0.04A_c$$

- The section at supports should be designed for a negative bending moment arising from partial fixity of at least 0.15 of the maximum bending moment in the span.
- The section at supports should be designed for a positive bending moment of at least 0.25 of the maximum bending moment in the span.

EC2 §9.2.2 – Shear reinforcing

- The ratio of shear reinforcement:

$$\rho_w = \frac{A_{sw}}{b_w s \cdot sin\,\alpha}$$

should be greater than the minimum value:

$$\rho_{w,min} = \frac{0.08 \cdot \sqrt{f_{ck}}}{f_{yk}}$$

- The maximum longitudinal spacing between shear assemblies should not exceed:

$$s_{l,max} = 0.75 \cdot d \cdot (1 + cot\,\alpha)$$

EC8 §5.4.1.2.1 + EC8 §5.5.1.2.1 – Geometrical limitations

- The width of a primary seismic beam shall satisfy the following expression:

$$b \leq min\{b_c + h; 2b_c\}$$

- The width of a primary DCH beam shall be greater than 200 mm.

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EC8 §5.4.3.1.2 + EC8 §5.5.3.1.3 – Longitudinal reinforcing

- At least two high bond bars with d = 14 mm that run along the entire length of the beam shall be provided both at the top and the bottom of primary DCH beam.
- Along the entire length of a primary seismic beam, the reinforcement ratio ρ of the tension zone shall not be less than the following minimum:

$$\rho_{min} = 0.5 \cdot \frac{f_{ctm}}{f_{yk}}$$

- In the critical regions of primary seismic beams the reinforcement ratio of the tension zone ρ shall not exceed the following maximum:

$$\rho_{max} = \rho' + \frac{0.0018}{\mu_\varphi \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$$

in addition, the ratio relative to the compression reinforcing ρ_{comp} should be:

$$\rho_{comp} \geq 0.5\rho + \rho'_{design}$$

- In primary DCH beams, at least one quarter of the maximum top reinforcement at the supports should run along the entire beam length.

EC8 §5.4.3.1.2 + EC8 §5.5.3.1.3 – Shear reinforcing

- Within the critical region of primary seismic beams the hoops diameter shall not be less than 6 mm.
- Within the critical region of primary seismic beams, the spacing of hoops shall not exceed:

DCM $$s = min\{1/4 \cdot h_w\,; 225mm; 8 \cdot d_{bl}; 24 \cdot d_{bw}\}$$

DCH $$s = min\{1/4 \cdot h_w\,; 175mm; 6 \cdot d_{bl}; 24 \cdot d_{bw}\}$$

2.4.2. Columns

EC2 §9.5.2 – Longitudinal reinforcing

- Longitudinal bars should have a diameter of not less than 8 mm.
- The total amount of longitudinal reinforcement should not be less than:

$$A_{s,min} = max\left\{0.10 \cdot \frac{N_{Ed}}{f_{yd}}; 0.002 \cdot A_c\right\}$$

- The area of longitudinal reinforcement should not exceed:

$$A_{s,max} = 0.04 \cdot A_c$$

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- For columns having a polygonal cross-section, at least one bar should be placed at each corner. The number of longitudinal bars in a circular column should not be less than four.

EC2 §9.5.3 – Shear reinforcing

- The diameter of the transverse reinforcement should be:

$$d_w \geq max\{6mm; 0.25 \cdot d_{l,max}\}$$

- The spacing of the transverse reinforcement along the column should not exceed:

$$s_{max} = min\{400mm; 20 \cdot d_{l,min}; L_{min,col}\}$$

EC8 §5.5.1.2.2 – Geometrical limitations

- For primary DCH columns, the minimum section dimension should not be less than 250mm.

EC8 §5.4.3.2.2 e §5.5.3.2.2 – Longitudinal reinforcing

- In DCH or DCM primary seismic columns the total longitudinal reinforcement ratio ρ shall be:

$$0.01 \leq \rho \leq 0.04$$

EC8 §5.4.3.2.2 e §5.5.3.2.2 – Shear reinforcing

- In DCH or DCM primary seismic columns, the diameter of hoops and/or cross ties should not be less than:

DCM	*6mm*
DCH	$0.4 \cdot d_{bL,max} \cdot \sqrt{f_{ydL}/f_{ydw}}$

- In DCH or DCM primary seismic columns, the spacing of hoops should not be less than:

DCM	$\mathrm{s} = \min\{1/2 \cdot \mathrm{l_{min}}\,; 175\mathrm{mm}; 8 \cdot \emptyset_l\}$
DCH	$\mathrm{s} = \min\{1/3 \cdot \mathrm{l_{min}}\,; 125\mathrm{mm}; 6 \cdot \emptyset_l\}$

- Within the critical regions of DCH and DCM primary seismic columns the value of the mechanical volumetric ratio of confining hoops ω_{wd} shall be:

$$\alpha\omega_{wd} \geq 30\mu_{\varphi}\upsilon_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_0} - 0.035$$

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- In addition, within the critical regions of DCH and DCM primary seismic columns the value of the mechanical volumetric ratio of confining hoops ω_{wd} shall be:

DCM	$\omega_{wd} \geq 0.08$	at base critical regions
DCH	$\omega_{wd} \geq 0.12$	at base critical regions
DCH	$\omega_{wd} \geq 0.08$	at all critical regions above the base

3. Wall design according to EC2 2005/EC8 2005

3.1. Strength

3.1.1. Definitions

The Eurocode defines the following types of two-dimensional elements:

- *Walls* (Piers for VIS): vertical elements with a length to thickness ratio of 4 or more (EC2 9.6.1(1));
- *Coupling beams* (Spandrels for VIS): ductile beams connecting two walls, and capable of reducing by at least 25% of the sum of the base bending moments of the individual walls when working separately (EC8 5.1.2). If the span to depth ratio is less than 3 the beams are classified as deep (EC2 5.3.1(3)).
- Among the wall elements, a further distinction is made between:
- *Ductile walls*: walls fixed at the base so that the relative rotation of the base with respect to the rest of the structural system is prevented, and designed and detailed to dissipate energy in a flexural plastic hinge zone free of openings or large penetrations, just above the base (EC8 5.1.2).
- *Large lightly reinforced walls*: walls with large cross-sectional dimensions (that is, horizontal dimensions at least equal to 4 meters or two-thirds of the height, whichever is less), and expected to develop limited cracking and inelastic behavior under seismic design situations (EC8 5.1.2).

The following design procedures cover the design of shear walls (*piers*) and coupling beams (*spandrels*) in terms of strength, capacity design and detailing provisions. Deep beams are not presently addressed.

3.1.2. Axial force and bending

PMM strength checks are performed with reference to the factored internal forces from imported load cases and/or combinations.

Complete 3D interaction surfaces are computed for each section with reference to the assumptions made in § 1.1.1 and § 2.1.1.

3.1.3. Shear

Shear checks are performed with reference to the factored internal forces.

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Shear strength is calculated in accordance with the assumptions made in § 1.6.2 with reference to the failure modes presented in § 2.1.4.

In addition, the possible sliding failure of large lightly reinforced walls is checked in accordance with EC2 6.2.5:

$$V_{Ed} \leq V_{Rd,S}$$

where:

$$V_{Rd,S} = \left[c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot (\mu \cdot sin\, \alpha + cos\, \alpha)\right] \cdot zb$$

$$V_{Rd,S} \leq (0.5 \cdot \nu \cdot f_{cd}) \cdot zb$$

V_{Ed}	acting shear
$z = 0.9 \cdot (0.8 \cdot h)$	internal forces arm
b	wall width
c, μ	interface roughness factors
$\rho = A_s / A_i$	
A_s	area of rebars
A_i	area of section
$\alpha = 90°$	
$\nu = 0.6 \cdot [1 - f_{ck}/250]$	

Multiple leg shear walls have sliding resistance computed for each leg separately. The single contributions are then projected along the two main directions and added together.

3.2. Capacity

3.2.1. DCM compression and bending at ductile walls

Primary ductile walls (*piers*) that are slender (with height to length ratio greater than 2) tend to have uncertainties in the bending moments distribution along the wall height. For this purpose, the Code recommends that the design bending moment envelope, be vertically displaced from the envelope of the analysis bending moments.

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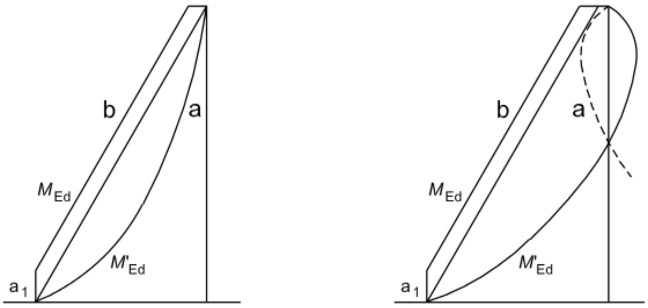

VIS calculates the amplified moment diagrams for slender walls based on the picture above. The tension shift a1 is calculated in accordance with EC8 § 5.4.2.4.

3.2.2. DCH compression and bending at ductile walls

Same as DCM.

3.2.3. DCM compression and bending at large lightly reinforced walls

According to EC8, large walls in compression and bending should take into account additional dynamic axial forces due to uplifting from the soil, or to opening and closing of horizontal cracks.

The dynamic component of the wall axial force is assumed by the program as being 50% of the axial force in the wall due to the gravity loads from the seismic design situation. This force is applied with a plus or a minus sign, whichever more unfavorable. If the structure behavior factor “q” does not exceed 2, the effect of the dynamic axial force is neglected.

3.2.4. DCH compression and bending at large lightly reinforced walls

Lightly reinforced walls are not allowed in DCH structures.

3.2.5. DCM axial force limitation at ductile walls

The normalized axial load of primary ductile walls (*piers*) is required not to exceed 0.4.

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3.2.6. DCH axial force limitation at ductile walls

The normalized axial load of primary ductile walls (*piers*) is required not to exceed 0.35.

3.2.7. DCM shear at ductile walls

According to EC8, the shear forces from analysis need to be amplified by 50% for capacity design. In addition, in dual systems containing slender walls, the shear design envelope should never be lower than half the amplified shear at the base.

The programs runs shear capacity checks for both simple walls (i.e. single leg) and composite walls (i.e. multi leg). Simple walls are checked in the strong direction only. Composite walls are checked in both directions.

The possible failure mechanisms are:

- compression failure of concrete struts;
- yielding of horizontal reinforcing.

The design shear strength is computed as for the columns, assuming an internal lever arm equal to 80% of the wall length.

3.2.8. DCH shear at ductile walls

The design shear forces are calculated in accordance with the following:

$$V_{Ed} = \varepsilon \cdot V'_{Ed}$$

where:

V'_{Ed} shear force from the analysis

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1 \cdot \left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q \qquad \text{for slender walls}$$

$$\varepsilon = \gamma_{Rd} \cdot \frac{M_{Rd}}{M_{Ed}} \leq q \qquad \text{for squat walls}$$

In addition, for dual systems containing slender walls, the design envelope of shear forces is not allowed to be lower than half the amplified base shear.

The programs runs shear capacity checks for both simple walls (i.e. single leg) and composite walls (i.e. multi leg). Simple walls are checked in the strong direction only. Composite walls are checked in both directions.

Shear checks are the following:

- compression failure of the concrete strut;
- yielding of the horizontal reinforcing;

- sliding of concrete within critical regions.

The *shear strength of the concrete strut* is computed as for columns (assuming a 45° inclination of the struts and an internal lever arm equal to 80% of the wall length). A 0.4 reduction factor is applied at critical regions.

The calculation of *shear reinforcement* takes into account the shear ratio $\alpha_s = M_{Ed}/(V_{Ed} \cdot l_w)$. If α_s is greater than 2, the shear strength is computed as for columns (assuming a 45° inclination of the struts and an internal lever arm equal to 80% of the wall length).

Otherwise, the design strength is the following:

$$V_{Rd,s} = V_{Rd,c} + 0.75 \cdot \rho_h \cdot f_{yd,h} \cdot b_w \cdot \alpha_s \cdot l_w$$

In addition, the ratio of horizontal (ρ_h) and vertical (ρ_v) reinforcing needs to meet the following:

$$\frac{\rho_h \cdot f_{yd,h} \cdot b_w \cdot z}{\rho_v \cdot f_{yd,v} \cdot b_w \cdot z + min\, N_{Ed}} \leq 1$$

The *sliding shear* resistance within critical regions is evaluated as follows:

$$V_{Rd,S} = V_{dd} + V_{id} + V_{fd}$$

Where:

$$V_{dd} = \begin{cases} 1.3 \cdot \sum A_{sj} \cdot \sqrt{f_{cd} \cdot f_{yd}} \\ 0.25 \cdot f_{yd} \cdot \sum A_{sj} \end{cases}$$

$$V_{id} = f_{yd} \cdot \sum A_{si} \cdot cos\, \phi$$

$$V_{fd} = \begin{cases} \mu_f \cdot \left[\left(\sum A_{sj} \cdot f_{yd} + N_{Ed}\right) \cdot \xi + M_{Ed}/z\right] \\ 0.5 \cdot \eta \cdot f_{cd} \cdot \xi \cdot l_w \cdot b_{w0} \end{cases}$$

$$\eta = 0.6 \cdot (1 - f_{ck}/250)$$

$\sum A_{sj}$	sum of the area of the vertical rebars intersecting the sliding plane
ξ	normalized neutral axis depth

$\sum A_{si}$ sum of the area of the inclined rebars intersecting the sliding plane

ϕ angle between the inclined bars and the potential sliding plane

For squat walls, the following additional condition is checked:

$$V_{id} > V_{Ed}/2$$

3.2.9. DCM shear at large lightly reinforced walls

The design shear forces are calculated in accordance with the expression:

$$V_{Ed} = \frac{q+1}{2} \cdot V'_{Ed}$$

Where V'_{Ed} is the shear force from analysis.

The programs runs shear capacity checks for both simple walls (i.e. single leg) and composite walls (i.e. multi leg). Simple walls are checked in the strong direction only. Composite walls are checked in both directions.

Shear checks are same as strength checks (§ 3.1.3):

- compression failure of the concrete strut;
- yielding of the horizontal reinforcing;
- sliding of concrete.

3.2.10. DCH shear at large lightly reinforced walls

Large lightly reinforced walls are not allowed in DCH structures.

3.2.11. DCM shear at coupling beams

Coupling beams (*spandrels*) are treated as primary beams.

3.2.12. DCH shear at coupling beams

Spandrels are treated as primary beams if at least one of the following is satisfied:

$$V_{Ed} \leq f_{ctd} b_w d$$

$$l/h \geq 3$$

Otherwise, the resistance to seismic actions is provided by diagonal reinforcement, in accordance with the following expression:

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$$2 \cdot A_{si} \cdot f_{yd} \cdot sen\alpha \geq V_{Ed}$$

Where

A_{si} = total area of rebars in each diagonal direction

3.3. Detailing provisions

For wall in the model, the following detailing provisions are checked by the program. A detailed report provides results.

<u>EC2 §9.6.2 – Vertical reinforcing</u>

- The total area of the vertical reinforcement, $A_{s,v}$, should lie between the following limits:

$$0.002A_c < A_{s,v} < 0.004A_c$$

- The distance between two adjacent vertical bars is checked not to exceed 3 times the wall thickness or 400 mm, whichever is less.

<u>EC2 §9.6.3 – Horizontal reinforcing</u>

- The total area of the vertical reinforcement, $A_{s,h}$, should be:

$$A_{s,h} > \max\left(0.001A_c; 0.25A_{sv}\right)$$

- The distance between two adjacent horizontal bars shall not exceed 400 mm.

<u>EC2 §9.6.4 – Transverse reinforcing</u>

- In DCH or DCM primary seismic columns, the diameter of hoops and/or cross ties should not be less than 6mm or a quarter of the maximum vertical steel's diameter.
- The distance between two consecutive ties should not be greater than:

12 times the minimum vertical steel's diameter

2.4 times the thickness of the wall

240 mm

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EC8 §5.4.3.4.2 – Geometrical limitations for ductile walls

- The thickness, b_w, of the confined parts of the wall section should not be less than 200mm.

  In addition:

  $b_w \geq h_s/15$ if $l_c \leq 2 \cdot b_w$ e $l_c \leq 0.2 \cdot l_w$

  $b_w \geq h_s/10$ otherwise

  where h_s is the story height.

EC8 §5.4.3.4.2 – Vertical reinforcing for ductile walls

- The longitudinal reinforcing ratio in the boundary elements of DCM and DCH ductile walls should not be less than 0.005.
- The longitudinal reinforcing ratio outside the boundary elements of DCH ductile walls should not be less than 0.002.

EC8 §5.5.3.4.5 – Horizontal reinforcing for ductile walls

- The horizontal reinforcing in DCH ductile walls should have a diameter of not less than 8 mm , but not greater than one-eighth of the width of the web.
- The horizontal reinforcing in DCH ductile walls should be spaced at not more than 250 mm or 25 times the bar diameter, whichever is smaller.
- The horizontal reinforcing ratio of DCH ductile walls should not be less than 0.002.

EC8 §5.4.3.4.2 – Transverse reinforcing for ductile walls

- Within the boundary elements of DCH and DCM ductile walls with $v_d > 0.15$, the value of the mechanical volumetric ratio of confining hoops ω_{wd} shall be:

$$\alpha \cdot \omega_{wd} \geq 30 \cdot \mu_\varphi \cdot \frac{M_{Ed}}{M_{Rd}} \cdot (v_d + \omega_v) \cdot \varepsilon_{ysd} \cdot \frac{b_w}{b_0} - 0.035$$

$$\omega_{wd} \geq 0.08$$

- In addition, within the boundary elements of DCH and DCM ductile walls the value of the mechanical volumetric ratio of confining hoops ω_{wd} shall be:

  DCM $\omega_{wd} \geq 0.08$

  DCH $\omega_{wd} \geq 0.12$

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- Above the critical region of DCM and DCH ductile walls, boundary elements should be provided for one more story, with at least half the confining reinforcement required in the critical region.
- Within the boundary elements of DCH and DCM ductile walls, the spacing of hoops and/or cross ties should be less than:

  $$s_{min} = \min(b_0/2\,; 175; 8 \cdot d_{bL})$$

- Within the boundary elements of DCH and DCM ductile walls the distance between two consecutive tied vertical bars should not be greater than 200 mm.
- Outside the boundary elements of DCH ductile walls, the distance between two consecutive tied vertical bars should not be greater than 500 mm.