RC SEC EN

1   Introduction

RC-SEC-EN is a software for the calculation of reinforced concrete section according to EN 1992-1-1 and EN 1998-1 and eventual national application annexes.

For brevity in the remainder of this user guide (and in the notes of the program) EN 1992-1-1 will be referred to as EC2, and EN 1998-1 as EC8. This software is a standalone application used in addition to a standard static or seismic analysis for checks of reinforced section belonging to beams, columns and walls. In seismic checks is considered the class of ductility demand (DCL, DCM or DCH) and whether or not the section is within a critical region of the member. If the checks are not seismic you must select the low ductility class (DCL) and so only EN 1992-1-1 is applied.

RC-SEC-EN performs checks for:

Ultimate Limit State (ULS):

• Uniaxial or biaxial Bending with or without axial force (§6.1 EC2-§5.4.3 EC8). It is also provided the eventual design of longitudinal and/or transverse reinforcement.

• Interactions domains N-Mx, Mx-My, N-Mx,My, with numerical results and graphical representation.

• Uniaxial and biaxial Shear (§6.2 EC2-§5.4.3 EC8) and Torsion (§6.3 EC2) force, with eventual design of stirrups.

• Instability by the method based on nominal curvature with uniaxial or biaxial bending. The calculation of curvature is performed according to equation (5.34) EC2 or by diagram moment-curvature (also in biaxial bending).

Serviceability Limit State (SLS):

• Limitation of normal stress (§7.2 EC2) by bending moment and/or axial force (performed methods: simplified, AAEM, EM).

• Limitation of Crack widths by means direct calculation according to §7.3.4 EC2 .

• Limitation of deflections of a beams (isolated or extracted from a frame) by double integration of curvatures and considering cracking, creep and shrinkage.

Ductility control:

• Direct check of curvature ductility factor mF (§5.2.3.4 EC8) by moment-curvature diagram (uniaxial or biaxial) taking into account (or not) of the eventual confining effect of hoops.

CROSS-SECTION TYPE

A set of practical cross-sections ("Predefined 12") like Rectangular, T, L, Flanged and Circular can be used as well as a free shape cross-sections ("General 26") formed by one or more concrete regions (polygonal or circular, full or empty) with free positioning of longitudinal steel bars and stirrups.

For Predefined cross sections is provided a faster input and the ability to perform the checks but also a first automatic design of reinforcement (for bending and/or shear forces) for a set of uniaxial combinations (up to 60) of internal forces. A special input is provided for rectangular section od columns 38 and walls 50 (especially for seismic checks).

For general sections is also possible, once defined the geometry of the section, to check or design the envelope of reinforcement in a single calculation cycle up to 60 combinations for each of the above types of combinations (ULS and SLS).

For general section type is also possible to import geometry of concrete and longitudinal bars (diameters and positions) from a *.dxf file.

CHECKS

ULS checks are performed by determining of the bidimensional interaction domain for uniaxial forces N-Mx, and of the threedimensional domain for biaxial bending N-Mx-My. For each design combination the program calculates:

• Axial force and bending moments od resistance, in reference to the assumed stress path 9.

• Ultimate position of the neutral axis.

• Stress and strain at ULS of all bars and vertices of the concrete section.

• Shear-torsion checks.

• Graphics of interaction domains.

• Stability analysis for one span column by nominal curvature method

SLS checks involve the determination of the following results:

• Normal stress in all bars and vertices of concrete section.

• Neutral axis position.

• Crack widths.

• Axial strain and curvature (AAEM (Age Adjusted Effective Modulus) method or EM method.

• Deflections in a beam-column isolated or belonging to a frame.

INTEGRATED TOOLS

In separate windows it is possible to run the following tools to perform:

- Definition of Creep and Shrinkage coefficients[73] according to §3.1.4 EN 1992-1-1 to use with AAEM or EM methods in Serviceability Limit State checks.
- Definition of Concrete Cover[75] according to §4.4.1 EC2-1

1.1 Scope of the program

The program is applicable only to RC sections for which can be considered valid the hypothesis of conservation of plane sections and the perfect bond of the reinforcement to the concrete. The sections to be calculated must be compact (as generally appear to be the majority of the RC sections) in such a way that the shear center can be regarded virtually coincident with the geometric centroid (shear does not generate torsion); in particular, are not considered thin-walled open sections.

The individual combinations of assigned internal forces are meant applied to the section in a single phase load (and not in stages such as in the calculation of prestressed sections).

For the same section can be done at the same time both the capacity checks (SLU) and the serviceability checks (SLS for normal stresses and crack opening).

The calculation methods and the code references used in the preparation of the program are described in detail in another section of the manual (Theoretical background).

The RC sections taken into account by the program are divided into the following four groups described in the remainder paragraphs:

- Predefined sections
- General sections
- Rectangular sections of columns
- Rectangular sections of walls
- Polygonal sections subject to torsion

At the start of each calculation in addition to the choice of one of the above five section types must be defined (in the General Data[9] tab) a set of parameters and options including the choice between seismic (DCM, DCH ductility class) and non-seismic (DCL ductility class) calculation and the position of the section in the structural member (in reference to the distance with the ends of the member). This allows to control of different prescriptions on

the minimum and maximum quantities of longitudinal and transverse reinforcement provided by the code rules. It is important to remark that for the first two types of sections the user must declare the kind of member (beam or column) in which that section is included.

1.2 Tipology of the sections

Predefined sections

They are the ones most frequently used in the current RC structures and consist of a single region of concrete having one of the following shape: rectangular, circular, T, rectangular with flanges (in this sub-case are included also C and L shapes).

For all the above sections except for the circular ones the armature must be defined by two rows of bars (upper and lower) to be assigned by the relative diameter and concrete cover.

For all predefined sections the bending moment to be assigned can have only the component Mx (uniaxial bending around the X axis of the reference with a positive sign if compresses the upper fibers of the section) and the corresponding neutral axis is always parallel to the X axis. This is strictly true only for symmetric sections about the Y-axis, but it is also common for unsymmetrical sections belonging to decks of buildings or to foundation decks. In these cases, in fact, is valid the prevailing hypothesis of rigid floor in its own plane; the existence of this constraint conditions allow to assume the neutral axis in the horizontal direction (i.e. in the direction perpendicular to the vertical loads plane).

The ULS capacity checks (up to 60 for a single section) can be performed in the presence (even simultaneously) of N normal force, bending moment Mx, shear and torsion Vy, T. These checks lead to the determination of resistance forces to compare to design assigned forces.

It is also possible, from the fixed size of the assigned concrete section, the design of the minimum envelope reinforcement (top, bottom bars and stirrups) able to cope with the assigned forces.

For circular predefined sections of columns it is also possible to perform an instability check[23] according to the simplified method of "nominal curvature".

For each section you can generate the ULS interaction diagram N-Mx .

To a single section can be assigned up to 60 SLS combinations of forces (each consisting of normal stress N and Mx) for each of the three possible types of SLS combinations (characteristic, frequent, quasi-permanent). The program determines the maximum normal stress and

the crack openings by comparing their values with the critical values in EC2.

General sections

Can be formed by one or more concrete regions (up to 20) polygonal shaped or circular shaped. To the concrete regions can be attributed different classes of resistance. Reinforcing bars can be assigned individually entering their coordinates and/or by means automatic generations of corner bars or linear and circular generations (thus allowing a much faster input).

The ULS checks are performed with axial and biaxial bending and shear set of forces: N, Mx, My, Vy, Vx (up to 60 combinations).

For each combination are also calculated two-dimensional diagrams of interaction Mx, My at constant axial force and also the three-dimensional interaction domain N-Mx, My. It is also possible to compute a general sections imposing the neutral axis to be parallel to X reference axis (see Forces⌐60¬). Biaxial serviceability checks are also performed.

It is also possible, from the fixed size of the assigned concrete section, the design of the minimum envelope reinforcement (top bars, bottom bars and stirrups) able to cope with the assigned design forces.

Rectangular sections of columns

In current projects the majority of the columns is characterized by a rectangular section and symmetrical reinforcement with respect to the principal axes of inertia. The seismic verification almost always involve the simultaneous presence of the components forces N, Mx, My, Vy, Vx.

It is therefore seemed appropriate to provide a specific type of calculation for these sections, starting from a very fast input of the geometry of the section. Automatic design of bars and stirrups is allowed. For this typology it is also possible to perform the instability check⌐47¬ according to the "nominal curvature" method.

Rectangular sections of walls

Walls elements are characterized by an elongated cross-section with a length to thickness ratio greater than 4 (§5.1.2 EC8). The program distinguishes between ductile walls and large lightly reinforced walls (see definition in § 5.1.2 EC8). Once defined the type of wall, the size of the section, the reinforcement and the design forced (from the analysis), the program performs bending (biaxial), shear checks (uniaxial) and ductility checks (detailing too).

Polygonal section in Torsion

Torsion check is performed for sections (hollows too) with a generic polygonal contour. If shear is also present the check should be manually completed applying the eq. (6.29) EC2. Shear reinforcement must be added to shear reinforcement already calculated.

SLS deflections in single beam

A single beam can be formed by one or more sub-elements each characterized by a single rc cross-section. Short time and long time deflection are assessed from the differential equation of bending line by double integration. The calculation is performed taking in accounting for curvature evaluation cracking, creep and shrinkage (AAEM/EM method[118]) if a large number od cross-section along the length of the beam.

1.3   Units

Units (expressed in S.I.) in this program are those most commonly used and they are not editable:

  ➢   Forces: kN, m
  ➢   Stress: Mpa (N/mm²)
  ➢   Section dimension: cm
  ➢   Diameter bars: mm

In each input and output text field is indicated then right unit assumed.

2   Basic working tools
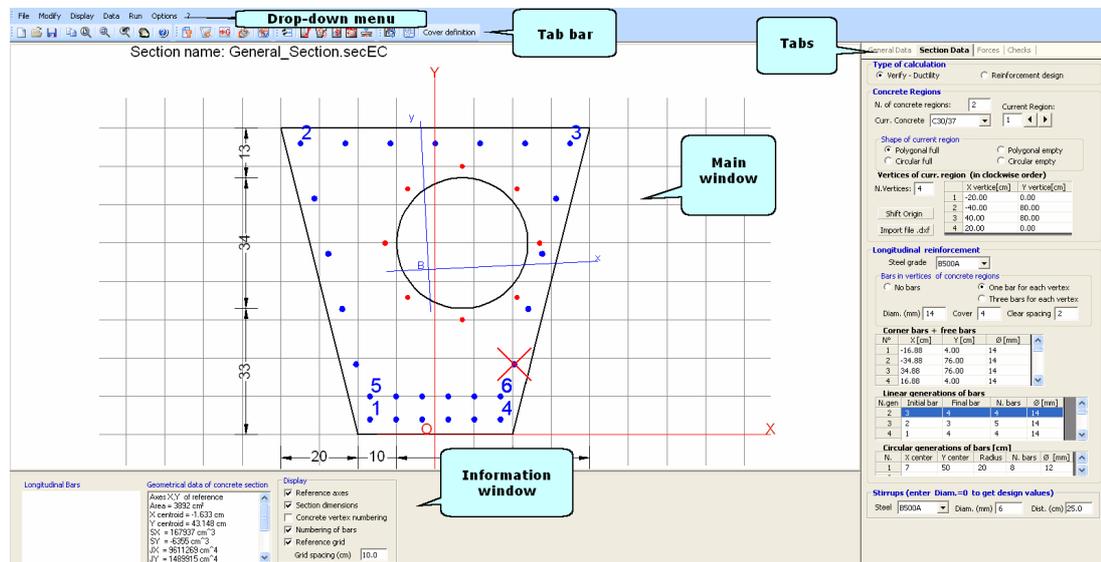
The user interface consists of several co-operating parts grouped together.

- Drop-down menu - It is the first row in the screen. These menu enable the management of all functions of the program.
- Navigator Tabs - Set of commands that open underlying windows for input and output data.
- Tabs bar (ribbon) - Set of tabs commands for quick access to specific functions of the program.
- Main window - In relation to selected command in the Navigator Tab the window shows the input and output data in graphical form. The content may be an image or a diagram with text.
- Information window - Actual information (in text form) related to selected Navigator Tab are shown in this window
- Printing window - In this window (not visible in the above figure), after checks, you can view, and eventually print on paper, all data is input and output.

The Navigator Tabs consists of five underlying windows:

- General Data
- Section Data
- Forces
- Checks
- Stirrups-Ductility

The first three tabs correspond to the first three commands on the menu "Data". The "Checks" Tab corresponds to the command "Save and run" in the Run menu. The last tab "Stirrups-Ductility" is active only after the results were displayed by the "Checks" tab; with "Stirrups-Ductility" tab, you can edit and/or assign hoops and cross-ties to all sections and immediately obtain new results selecting the "Checks" Tab. Also through the last tab you can generate moment-curvature diagrams (also biaxial) aimed at the evaluation of the curvature ductility factor (§ 5.2.3.4 EC8).

Once assigned the characteristics of materials (concrete and steel) in the materials library[67], available by the "Data" menu or by the direct command , the modeling of the input section and its subsequent resolution can be controlled interactively by the use of only five tabs above.

Under the graphics window is finally present an information window whose content varies depending on the type of current graphic representation. If you are in the stage of data entry section it displays the legend of the bars assigned geometric data (area, coordinates of the centroid, moments of inertia and static of concrete section) and a series of check box options on how with which he likes display the section in question. Last tab "Stirrups-Ductility" is available only after a first run: you can later to modify or assign hoops and/or cross-ties with immediate shear check reporting. In the same window are present command to generate moment-curvature diagrams (biaxial too) for confined or not confined sections (for structures in seismic zones).

2.1   General Data

This is the first window to fill for a new project. These are the principal input data:

Typology of the section

The five choices provided are:
- Predefined sections[12]: rectangular, T, rectangular with flanges, circular. Bending and shear forces are only uniaxial. Checks or design of reinforcement.

- General sectionsi[26]: are formed by one or more concrete regions polygonal or circular shaped with reinforcement however arranged. The bending moment and shear forces can be selected as uniaxial or biaxial. In general it is expected to only checks calculation. For polygonal sections formed by a single concrete region can also be performed the design of longitudinal bars.
- Rectangular section of colums[38]: the arrangement of the reinforcement is always doubly symmetrical. The calculation is always biaxial in bending and shear. Check or design of reinforcement.
- Rectangular section of walls[50]: the arrangement of the reinforcement is always doubly symmetrical. Program performs only checks calculation in biaxial bending and uniaxial shear.
- Polygonal section torsion[54]: simple torsion check for polygonal sections (hollow or not).
- SLS deflections in simple beam[126]: the beam can be isolated or belongs to a frame.

Exposure class

This selection, according to § 4.2 EC2, affects the definition of the concrete to be used in the project of the section. The user is responsible for the consistent choice of the concrete class (program do not perform any compatibility check between exposure class and concrete class).

Member characteristics

Defines the type of member to which the section belongs: beam, column, beam, foundation beam in seismic zone, beam without stirrups. If the project is in seismic zone the user must indicate if the current section is located within or not a critical region according to the section 5 of EC8. The checks take in account the detailing rules according to §9 EC2 and §5 EC8.

Load path for moments of resistance

The calculation of the resistance moment force for a given design moment and axial force may be performed by program with a constant ratio M/N or keeping N= constant. The second way must be choose in seismic checks.

Options for serviceability limit states (SLS): user may choose between simplified method (n method)[118] that assimilates rheological phenomena of the conglomerate placing equal to 15, the coefficient n of homogenization (n=Es/Ec is editable in the appropriate grid of the materials library[67]), and il AAEM method[118] that take in account the

values assigned to creep, shrinkage and ageing in the concrete grid in the materials library 67. It should be noted that the assignment of the serviceability combination forces is optional unlike those of resistance that must always be assigned (at least one). In order to set always zero the tension strength in cracking zones of concrete for crack width calculations, you may select the appropriate check box. If you want to calculate the crack width setting e2 of (7.13)EC2 as the value at the edge of effective tension region of concrete you may select the appropriate check box.

Ductility Class

Low ductility class DCL includes all section to be calculated only with EC2 standards while Height (DCH) and Middle (DCM) Classes include all the section required by EC8 in seismic zones. For beams in DCH or DCM classes and within critical regions of the member you should assign the curvature ductility factor m in order to respect the eq. (5.11)EC8. Idem for columns in order to respect the eq. (5.15)EC8 when expected; alternatively for direct check of local ductility it is possible to generate the moment-curvature diagram selecting the Stirrups-Ductility 21 Tab.

2.2 Predefined sections

This dialog window is opened if in General Data 9 Tab you have selected the typology "Predefined Section". The expected data are:

- Type of calculation

  you may choose between the verification (checks) of the section and the Reinforcement Design. In the first case it is necessary to assign (in the same window) geometric data relating to the shape of the section and the numbers, diameters and covers of the bars, in the second simply assign the concrete geometric data and the covers of the two rows (without numbers and diameters of the bars) .

- ## Section shape

  in this drop-down menu you can choose one of the various shape of sections provided: Rectangular, T, Rectangular with flanges, Circular. Depending on the choice made, appropriate text boxes appear for entering the dimensional data. The drawings shown above illustrates the input of a L section selecting the shape "Rectangular section with flanges" and filling the appropriate text boxes.

- ## Concrete class

  in this drop-down menu you can choose one of the concrete classes present in the Materials library 67.

- ## Longitudinal reinforcement

  in this drop-down menu you can choose one of the steel grade classes present in the Materials library 67.

- ## Section sizes

  for each of the provided shapes appear the appropriate text boxes to enter the sizes: the correspondence between the numerical data to be entered and the position of the corresponding sizes is always present in the graphic scheme at the top of this Tab window.

- ## Lower cover

  concrete cover [cm] of the lower row of bars measured from centroid of bars to lower fiber of concrete. This concrete cover should be assigned by the user respecting the minimum concrete cover required by §4.4.1 EC2. To this end it is available the separate program concrete cover 75 to load from the dedicate command in the principal Tab Bar. The cover to assign is measured from the centroid of the bar (different from edge cover obtained by cover 75 sub-program). Between section results are always expected the edge cover of longitudinal and stirrups bars. No conformity check to EC2 rules - exposure class, strength class, etc. - is done by the program on these values.

- ## Upper cover

  concrete cover [cm] of the upper row of bars measured from centroid of bars. The same considerations carried out in the previous data.

- ## Lateral cover

  concrete cover [cm] of bars measured from centroid of bars to lateral edge fiber sides of the principal rectangular part of the section.

- Bottom bars

  a single row of bottom longitudinal bars may be assigned by input of their number and diameter. You may assign (in this same row) even bars of two different diameters. In this input the user must be careful to respect the maximum permitted spacing (see §8.2 EC2). Minimum spacing between longitudinal bars is calculated (as one of results) and compared with the value assigned in code and reinforcement options 70. If you want to assign more than one row of bar you should input the section as a general section in General Data 9.

- Top Bars

  a single row of bottom longitudinal bars may be assigned by input of their number and diameter.

- Web Bars: (not present in the above figure)

  two lateral web row are provided only for rectangular section of columns. The possible presence of these lateral bars increases bending and torsional resistance of the section.

- Stirrups

  in a verification calculation you may assign diameter and pitch of one or two hoop. You can also assign one or more cross-ties (if intermediate longitudinal bars are present). If you assign the null value to the diameter (or to the pitch) the program performs the automatic design of stirrups using the user default diameters in code and reinforcement options 70.

2.2.1 Reinforcement design

Given dimensions and strength class of the concrete section and the steel grade of longitudinal bars and stirrups you can require to the program a first design of longitudinal bars and stirrups selecting the design option in Section Data dialog window (frame "Type of calculation"). For a circular section of a column such dialog appears as below:

The design is referred to all ULS combination forces assigned in the pertinent dialog window (Forces 60 Tab). For SLS forces program execute only checks calculation after ULS design of longitudinal bars.

Selecting Checks 41 Tab the program performs the design and a subsequent complete checks. After first check you can modify, as many time you want, the reinforcement data in order to refine the results to your need.

In order to require to design only stirrups (if longitudinal bars already assigned) you shall input zero value in the Diameter (or spacing) text box in the stirrups frame. The stirrups design also relates to the control of the local ductility in critical section of columns in seismic zones.

2.2.2 Results and N-Mx diagram
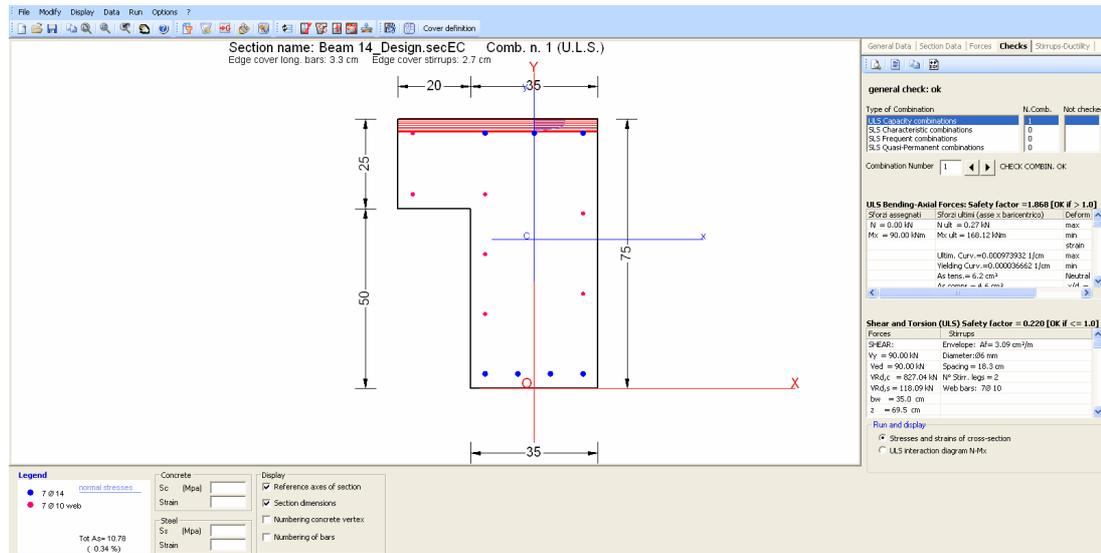
Selecting Checks Tab the program calculates and shows for each assigned combinations (ULS and SLS):

- [*Main window*] the graphical data (with eventual dimensions) of section with the concrete compressive area (red colored) and the corresponding stress diagram (blue colored); for for T and flanged sections the program generate new bars magenta colored (defined as web bars in the Legend and with diameter as defined in Code and reinforcement option 70) to sustain the horizontal secondary hoop; if torsion force is present the number of these web bar is such to sustain the longitudinal steel area necessary for torsion.

- [*Checks Tab window*] the principal results for Bending and Shear (capacity checks for ULS) and for Bending (stress control and crack widths for SLS).

- [*Information window*] the legend of longitudinal bars, the display options and the stress and strain of bars and of concrete vertices selected by mouse.

Command button of the above Checks Tab bar (from left to right):

- Print preview. Capture the image present in the main window and paste it in the Microsoft XPS Document Writer (useful to print the diagram directly on paper):

-     Print Checks. Print data input and all the checks in a special Text window allowing a comfortable viewing; you can save the same text to a file with *. doc, *. rtf, *. txt formats.

- 📋   Copy.   This command copy the image at moment displayed in the main window in the clipboard memory so you can paste it in any document you want:

- 📊   Export Dxf.   You can save in *.dxf file format the image at moment displayed in main window using a dialog box.

ULS INTERACTION N-Mx DIAGRAM

If you select the N-Mx diagram option in Checks window program display the strength diagram N-Mx in the Main window:

STRENGTH DOMAIN N-Mx (ULTIMATE LIMIT STATE)
Section name: Beam 14_Design.secEC
File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\CALCOLI_SEZ\Beam 14_Design.secEC
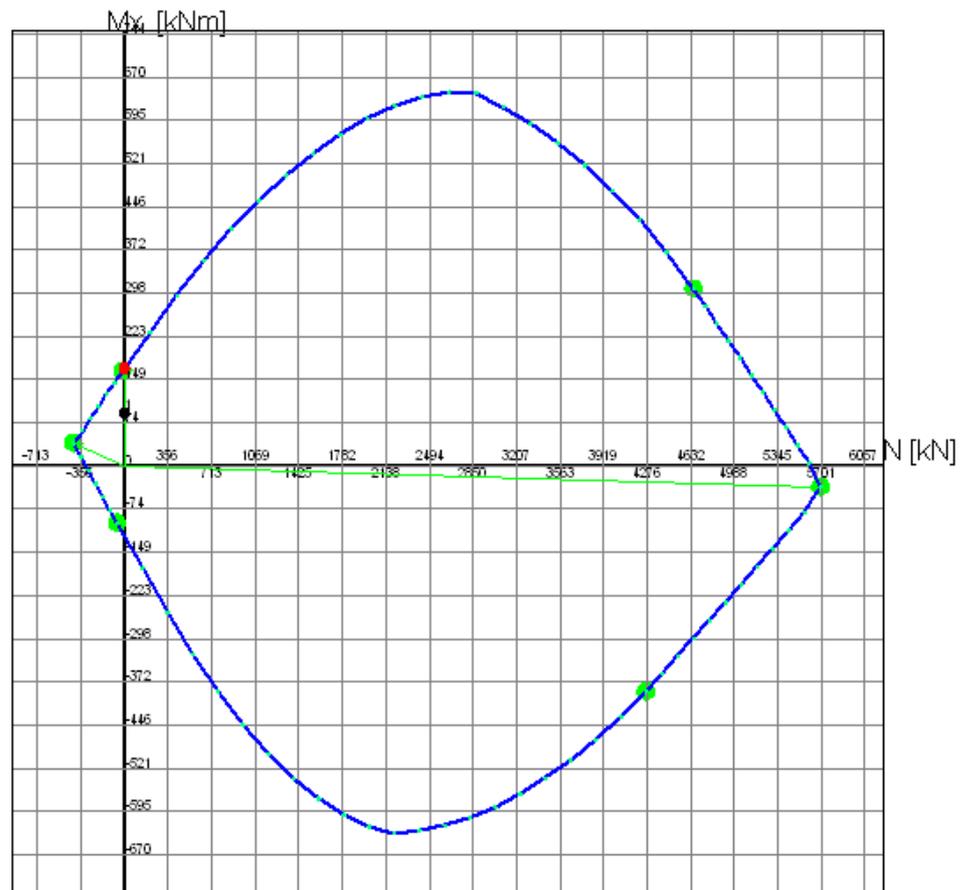
Grid spacing Mx = 74 kNm   Grid spacing N = 356 kN

The corresponding Information window shows:

In N, Mx text boxes shall appear the values of calculated points of the domain boundary selected on the graph by mouse. Drawing scale and grid spacings may be modified (rounding their values). Display options can modify the graphical aspect of the diagram.

2.2.3 Stirrups

After performing calculation (Checks Tab) you can display and/or change stirrups in the Stirrups-Ductility Tab window. In the below figure is shown an example of a beam section with two hoops and two cross-ties as transverse reinforcement.
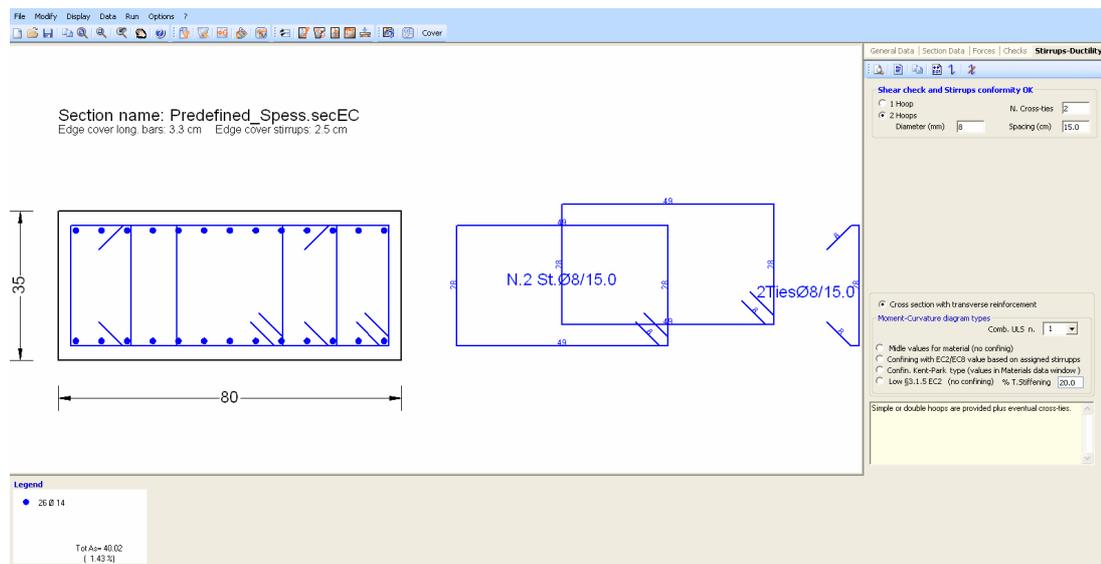
Options, text boxes and graphic commands are provided in the dialog box to change the initial proposal for hoops and cross-ties. For each change that you make the program performs an immediate shear check showing in the upper frame caption if this check is Ok or Not:
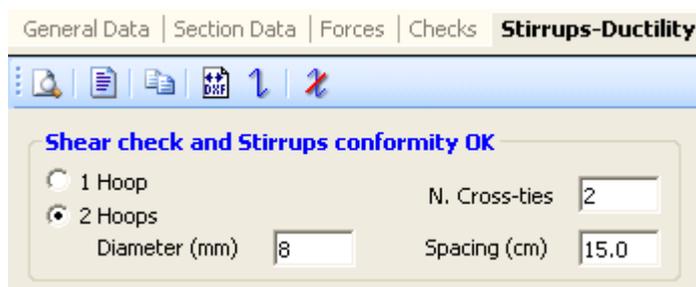
For predefined section the choice is limited to the number of hoops (one or two) and the number of cross-ties. For more complex shapes of stirrups is recommended to model the section as a general one.

The commands in Tab bar are (from left to rigth):

-    Print preview. Capture the image present in the main window and paste it in the Microsoft XPS Document Writer (useful to print the diagram directly on paper):
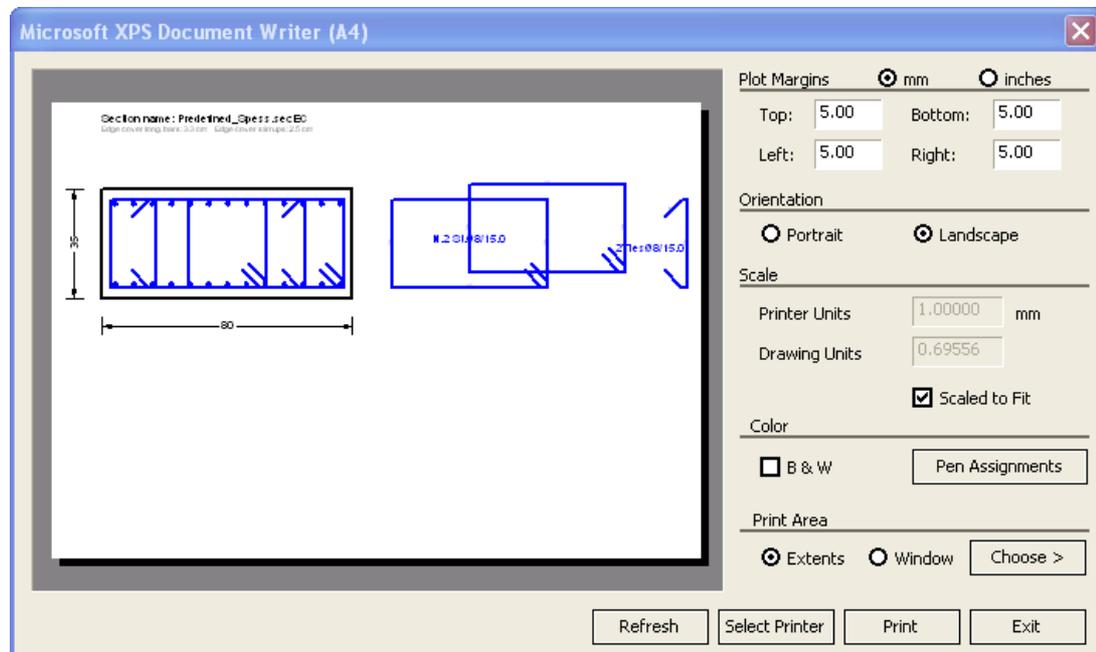
-     Print numerical results for ductility check. Unavailable in the present display of stirrups.

-    Copy. This command copy the image at moment displayed in the main window in the clipboard memory so you can paste it in any document you want.

-     This command generate a dxf file of the image at moment displayed in the main window.

-     Graphical generation of a cross-tie. Unavailable for predefined sections

-     Delete a cross-tie. Unavailable for predefined sections

Diameter and pitch of hoops can be changed in the corresponding text-boxes. The diameter of cross-ties is that assigned in Code and Reinforcement Options 70 window.

2.2.4   Ductility check

Ductility checks concern the following positions of sections of structures in seismic zones:

• within critical regions at the ends of all the beams (not secondary beams)

- within critical regions at the base of columns in DCM ductility class. (are excluded all the other critical areas of the columns on the upper floors on the first)
- within all critical region of columns in DCH ductility class.
- within critical regions of ductile walls

To respect the required local ductility in the above critical regions the curvature ductility factor μ_Φ must comply with the rule in §5.2.3.4 EC8:

$$\mu_\Phi \geq 1{,}5 \ (2q_0 - 1) \qquad \text{if} \qquad T_1 \geq T_C \qquad (5.4)$$
EC8

$$\mu_\Phi \geq 1{,}5 \ [1+2(q_0 - 1)] \ T_C / T_1 \quad \text{if} \qquad T_1 < T_C \qquad (5.5)$$
EC8

where:

q_0 basic value of the behaviour factor

T_1 fundamental period of the building

T_C period at the upper limit of the constant acceleration of the spectrum

For beams EC8 allows to replace (5.4)-(5.5) above control with the equations (5.11), (5.12).
For columns EC8 allows to replace (5.4)-(5.5) above control with the relation (5.15).

Building a moment-curvature diagram of the cross-section it is possible to evaluate μ_Φ so as to allow a direct check of the above (5.4)-(5.5).

In the figure below is shown such a diagram generated for a confined circular column in seismic zone.
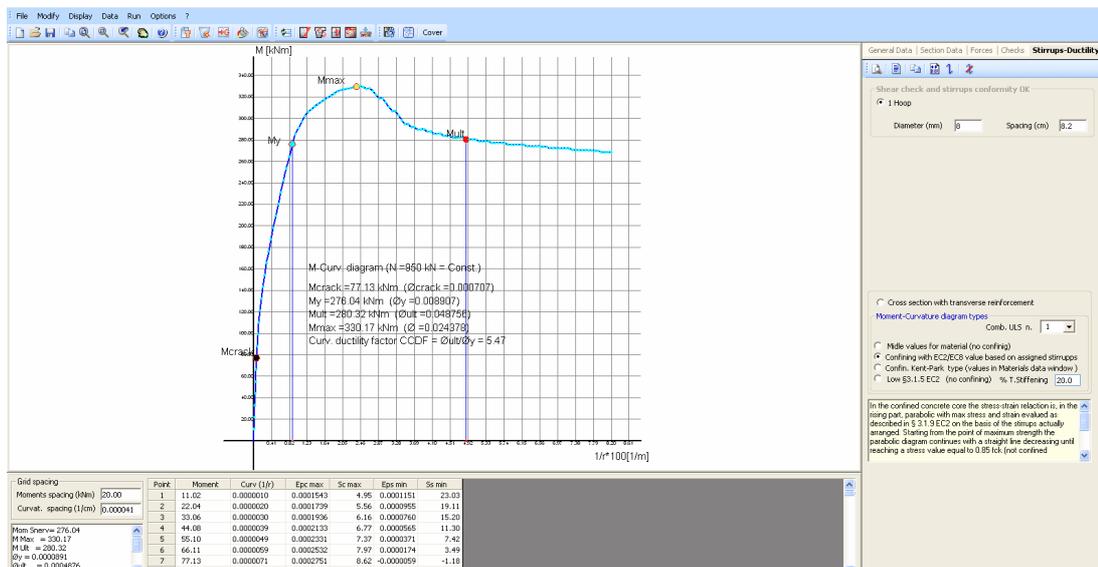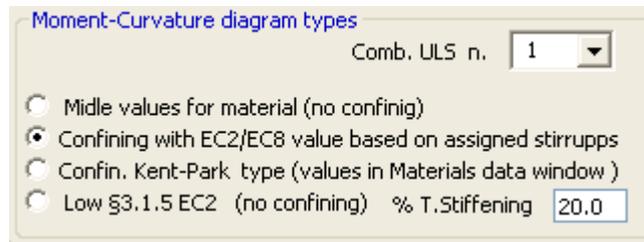
The moment-curvature diagrams can be of the following types:

- The first option generate a diagram using for concrete and steel the same stress-strain relationship used for design check but with the substitution of the design values with the mean ones.
- The second option refers to a diagram that take in account the confining of concrete[107] core by stirrups and the spalling of concrete cover at strain strength.
- The third type refers to a concrete stress-strain relation parabola-trapeze (Kent-Park Type) whose parameters are assigned in the Materials Library[67].
- Last type generate a diagram base on the relation shown in §3.1.5 EC2. You can take into account a certain percentage of tension stiffening.

2.2.5   Instability of circular columns

For predefined circular section of column you can check the instability of the column with nominal curvature method[107] (§5.8.8 EC2). This method is primarily suitable for isolated member with constant normal force and defined length l_0 (pre-calculated according to § 5.8.3.2 EC2).

For columns with rectangular section the same type of checks is performed by program in the pertinent typology "Rectangular section of columns[47]". For the present circular section this type of calculation The "Section Data" Tab looks like:

The relevant input data in this window are:

- Diameter of concrete circular section
- Concrete Class and Steel grade
- Number and diameter of bars arranged along the circumference with a given concrete cover (measured from centroid of bars)
- Design constant axial force N_d

- Effective creep ratio according to eq. (5.19) EC2: $_{ef} = _{(\infty,t0)}$ MX_{0qp}/MX_{0d} where MX_{0qp} is the first order moment in SLS quasi-permanent combination

- First order bending moment in ULS combination: MX_{0d}
- Eccentricity eY: $= l_{0Y}/400$ to take into account geometric imperfections according to simplified alternative in § 5.2(9) EC2.
- Effective length l_{0Y}: to evaluate in advance according to § 5.8.3.2 EC2

The curvature $1/r_0$ in eq (5.34) can be assessed by the program (see in above figure the option "Mode of calculation of curvature 1/r") with the direct formula in § 5.8.8.3 EC2 or by means the moment-curvature diagram as in "model column" method. Below is an example of such a diagram for a circular section:

MOMENTS-CURVATURES DIAGRAM (Nd = 500 daN)

Section name: Circ_buckling.secEC

File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\CALCOLI_SEZ\Circ_buckling.secEC

Moments grid spacing = 20 kNm   Curvatures spacing = 0.000012678 1/cm
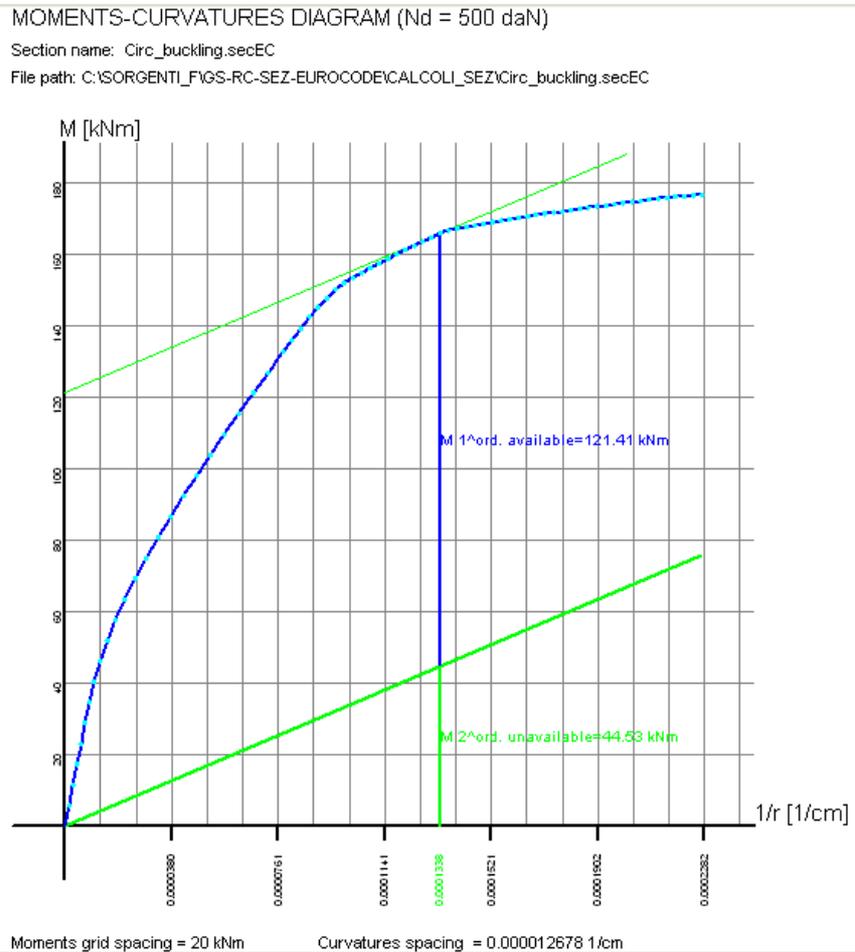
2.3 General sections

This dialog window is opened if in General Data ⌐9⌐ Tab you have selected the typology "General Section".

For this type of sections are expected checks for ULS and SLS combinations in uniaxial or biaxial bending and shear forces with or without axial force. The general sections may be formed by one or more regions of concrete (up to 20) each of which can have a polygonal shape or circular. Each region can be can be either full or empty (hollow sections). The bars can be assigned individually or automatically generated by the program on the basis of specific data shown below.

It is important to assign reinforcement only after having defined the number and the shape (circular or polygonal) of concrete regions. In order to accelerate the input of bars without the need to manually provide all the coordinates of the individual bars, may be assigned a bar at each vertex of the polygonal region by means of the appropriate button (1 or 3 bars for each vertex).

Any intermediate bars along the sides may be assigned by one or more linear generations of bars. In the example below the only coordinates assigned directly (in free bars grid) are those of the isolated bars n. 5, 6.

First you have to choose between the check option (assigning all geometrical and reinforcement data) and the design of reinforcement option (at geometrical data of concrete already assigned). In second case the program provides a tubular design distribution of longitudinal bars along the perimeter of concrete regions (that you can later freely change).

- Concrete Regions

In this frame the first input data is the number of concrete regions of the section (max 20). The above example section is formed by two concrete regions; the first is a full polygonal, the second is an empty circular one. The remaining data in this frame relate to each region (to select in the text box named Current Region) in which it is necessary to assign:

- Current concrete class: to choose in the corresponding drop-down menu between the classes assigned in the Materials Library [67].

- Shape of current region: to each region you must the pertinent polygonal or circular shape (full or empty).

- Vertices coordinates region: for polygonal regions these coordinate of vertices are to be assigned in clockwise order; for circular region the input data are the centre coordinates and the radius. Not allowed vertices in alignment with the two contiguous vertices (each straight side of polygonal regions must be defined only by the two end vertices).

- Shift origin: this optional command lets you to change all coordinate to a new reference system with origin in the concrete centroid of the section (particularly useful for imported sections from dxf file [64]).

- Import file .dxf: this optional command lets you import all coordinates of vertices of a concrete region and the positions and diameters of bars contained in a file of type *. dxf built according to the details given in Import general section from dxf file [64].

- Longitudinal Reinforcement

For its definition it is necessary to assign:

- Steel grade: to choose in the corresponding drop-down menu between the classes assigned in the Materials Library [67].

- Bars in vertices of concrete regions: if you choose one or three bars in each vertex the program generate bars in all vertices of the polygonal section. The bars diameter of the first assignment is the default setting in the text box named "Diam.". The spacing between the three bars in each vertex is the default value in the text box named "Clear spacing". You can then change the diameter of the individual bars in the grid named "Corner bars + free bars": to change the diameter with a unique value just assign that value in the text box named "Diam.".

- Diam.: it is the default diameter [mm] of vertex bars (if present) initially set equal to the default value assigned in code and

reinforcement options⎡70⎤ window. Changing this value all diameters of corner bars assume the same new value.

  - Cover: it is the default value [cm] of vertex bars initially set equal to the default value assigned in code and reinforcement options⎡70⎤ window. This cover value is measured from the centre of bars. Changing this value all cover of bars assume the same new value. If bars are imported from a .dxf file (and then text box "bars in vertices of concrete regions" is set = 0) the cover value must be assigned directly by user as a mean value to define the effective width "d" of section in uniaxial or biaxial shear calculation and the cover c in eq. (7.11) EC2. No conformity check to EC2 rules - exposure class, strength class, etc. - is done by the program on these values.

  - Clear spacing: this value is active only if user select the "Three bar for each vertex" option. The value refers to the minimum distance between perimeters of vertex bars. The change in its value has immediate effect on all the vertex bars.

  - Corner bars + free bars: in the first rows of this grid are placed the coordinates and diameters of the vertex bas. In subsequent rows can be entered manually more isolated bars. In the above example have been directly assigned the free bars number 5 and 6.

  - Linear generations of bars: in this grid you can assign a number of bars equidistant from each other and placed on the line joining any two bars already defined in the grid "Corner bars + free bars". In the above example have been assigned five linear generation: the first of which is between bar n.1 (Initial bar) and bar n. 2 (Final bar) and consists of n. 4 bars with diameter equal to 14 mm.

  - Circular generations of bars: in this grid you can assign a number of equidistant bars placed on a circumference of a certain radius. In the above example we have assigned one circular generation by means the X centre, Y centre coordinates, the Radius, the number of bars and their diameters.

• Stirrups

doing a first verification calculation you may assign steel grade, diameter and pitch of only one initial virtual hoop. After the first calculation you can assign one or more hoops and cross-ties defined in the Stirrups-Ductility⎡33⎤ Tab. If you set the null value to the diameter (or to the pitch) the program performs the automatic design of the virtual hoop using the user default diameters in code and reinforcement options⎡70⎤.

2.3.1 Reinforcement design

Given dimensions, strength class of the concrete section and the steel grade of longitudinal bars and stirrups you can require to the program a first design of longitudinal bars and stirrups selecting in "Section Data" dialog window the option "Reinforcement design". For a general section such dialog appears as below:

Diameters of bars (longitudinal and for stirrups) used in design are those stored in Materials Library 67 (that you can modify as you want).

Longitudinal bars provided by the program after the first run ("Checks" Tab selection) are uniformly distributed along the perimeter of the concrete regions. You can easily modify their position and diameter and run again the checks.

After the first run the program provides only for a virtual hoop around the perimeter of the concrete region. In the Stirrups-Ductility 33 Tab window you can generate (in the appropriate grid) one or more hoops (up to 9) and cross-ties (up to 80). Selecting Checks Tab the program performs new checks for the modified stirrups. And so on interactively.

Print results on the pertinent window (command 📄)to evaluate in detail eventual non conformity of checks.

2.3.2   Results and diagram Mx-My
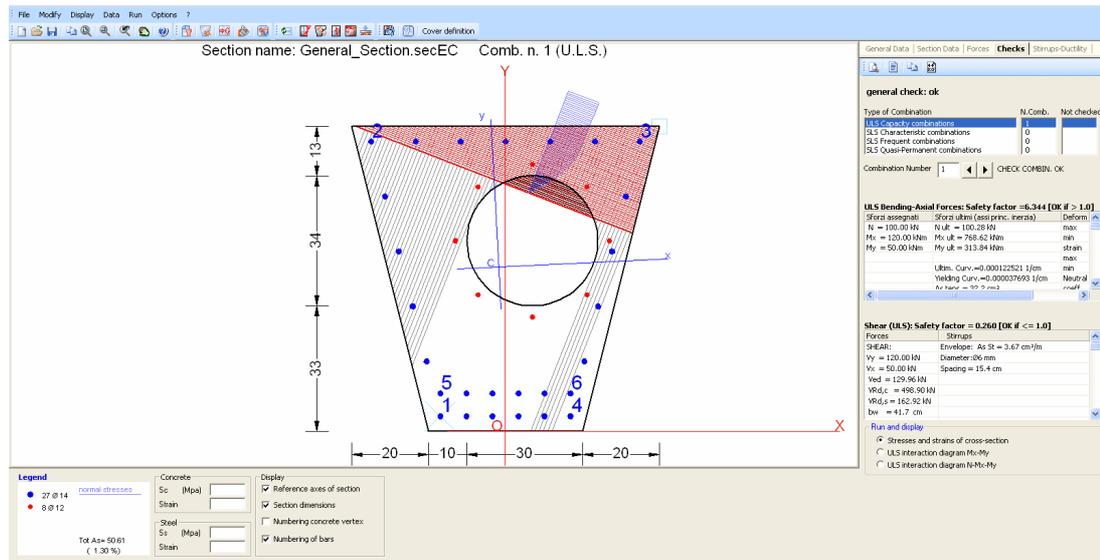
Selecting the "Checks" Tab (or the equivalent command) the program calculates and shows for each assigned combinations (ULS and SLS):

- [*Main window*] the graphical data (with eventual dimensions) of section with the concrete compressive area (red coloured) and the corresponding stress diagram (blue coloured); the black parallel line (orthogonal to neutral axis) represent the active strips shear resisting.

- ["*Checks" Tab window*] the principal results for Bending and Shear (capacity checks for ULS) and for Bending (stress control and crack widths for SLS).

- [*Information window*] the legend of longitudinal bars, the display options and the stress and strain of bars and of concrete vertices selected by mouse.

Command button of the above "Checks" Tab bar (from left to right):

- 　　　　Print preview. Capture the image present in the main window and paste it in the Microsoft XPS Document Writer (useful to print the graph directly on paper):
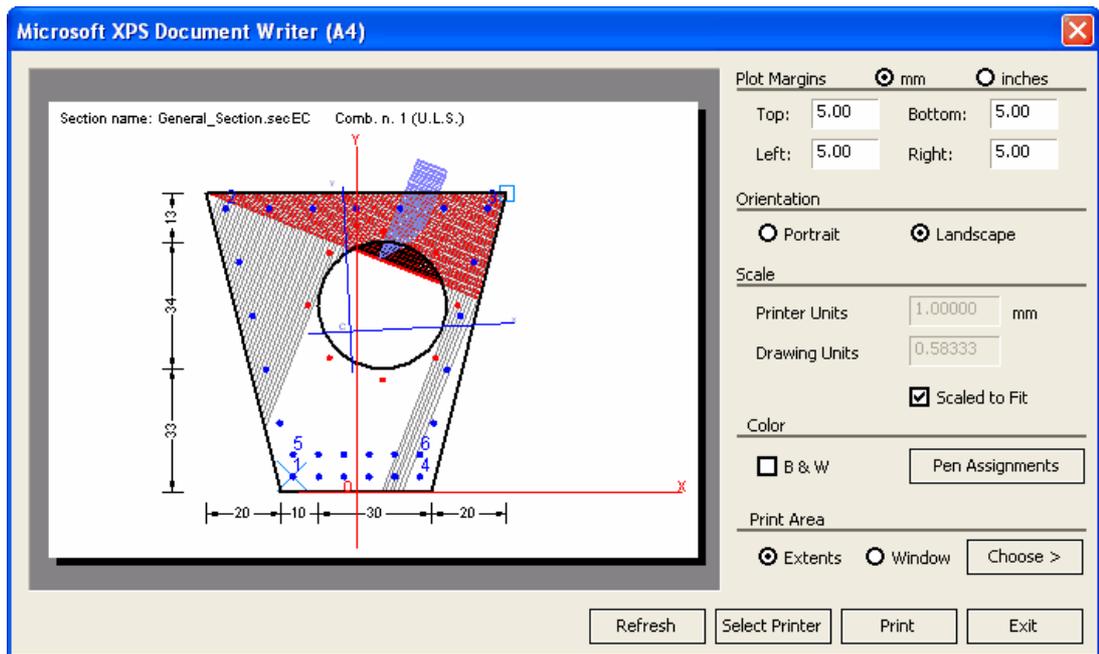
-   Print Checks. Print data input and all the input data and the checks in a special Text window allowing a comfortable viewing; in this window you can save its text to a file in *. doc, *. rtf, *. txt formats.

- 📋   Copy.   This command copy the image at moment displayed in the main window in the clipboard memory so you can paste it within any document you want

- 🔲   Export Dxf.   You can save in *.dxf file format the image at moment displayed in main window using a dialog box.

ULS INTERACTION Mx-My / N-Mx-Mx DIAGRAMs

If you select the ULS interaction Mx-My diagram option in Checks window program display the strength diagram Mx-My in the Main window:

STRENGTH DOMAIN Mx-My (ULTIMATE LIMIT STATE)

Section name: General_Section.secEC

File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\CALCOLI_SEZ\General_Section.secEC

Grid spacing moments Mx, My = 122 kNm

The corresponding Information window shows:

In N, Mx, My text boxes shall appear the values of calculated points of the domain boundary selected on the graph by the mouse. Drawing spacings may be modified (rounding its values). Display options can modify the graphical aspect of the diagram.

If you select the ULS N-Mx-My diagram option program display the interaction tridimensional diagram N-Mx-My in the Main window:
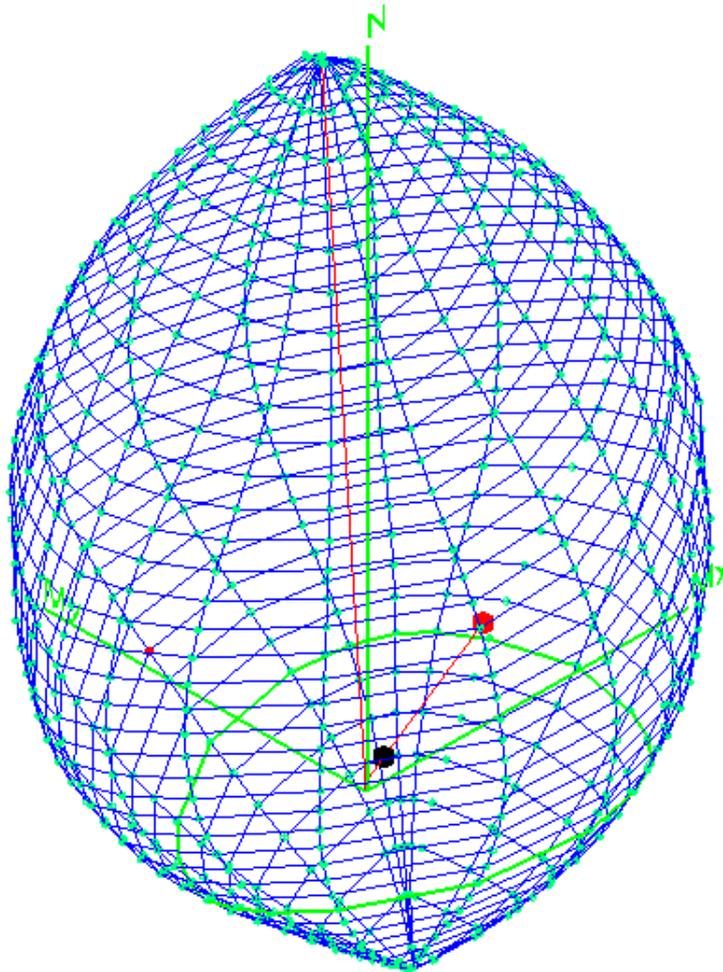
2.3.3 Stirrups

After performing calculation (by "Checks" Tab) you can display and/or change stirrups in the "Stirrups-Ductility" Tab window. In the below figure is shown an example of a column section designed for seismic zone with three hoops and two cross-ties as transverse reinforcement.
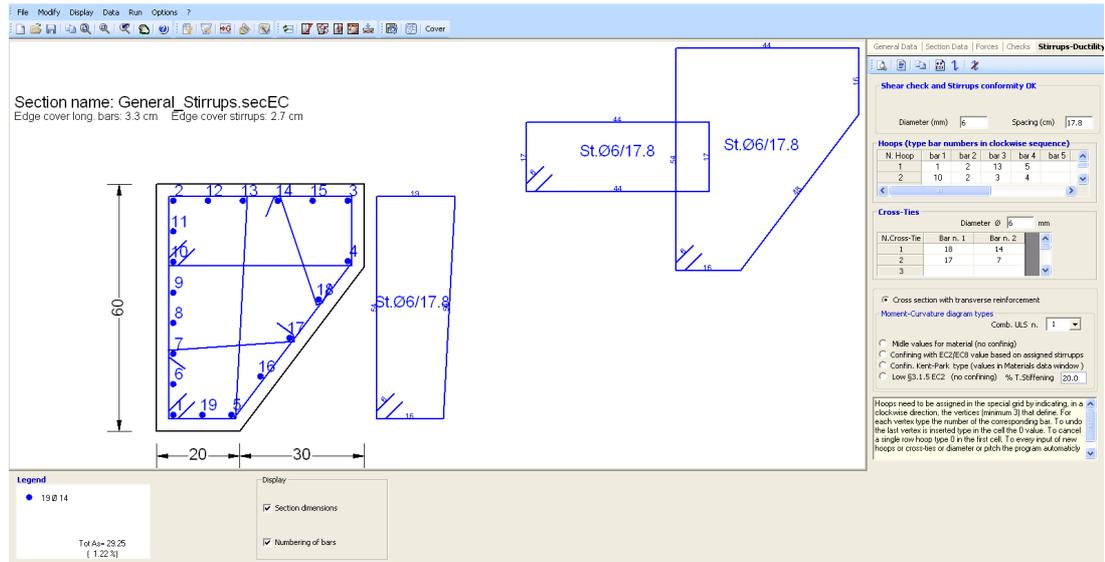
Options, text boxes and graphic commands are provided in the dialog box to change the initial proposal for hoops and cross-ties. For each change that you make the program performs an immediate shear check showing in the upper frame caption if it is Ok or Not:
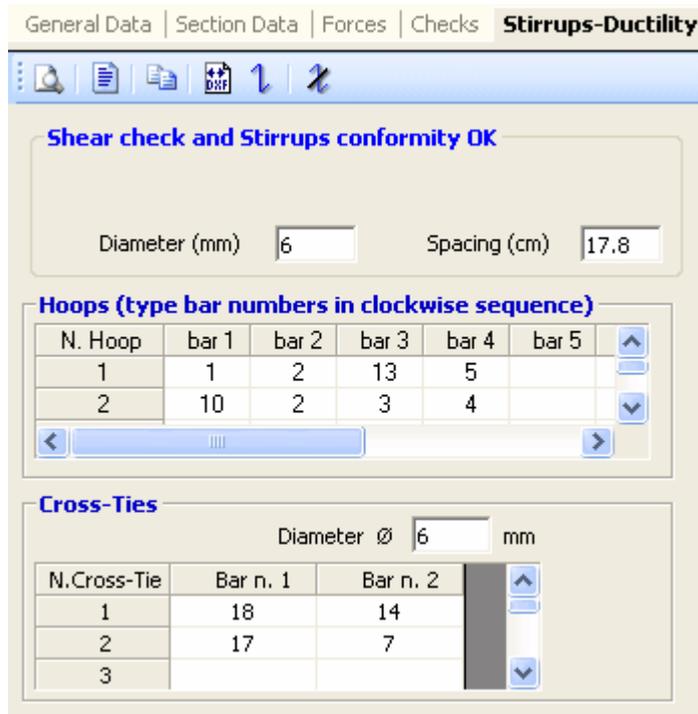
The three hoops have been generated typing in the upper grid of this dialog window, for each hoop, the long. bar numbers that define vertices. <u>To erase whole hoop type 0</u> in the first column of the grid. The cross-ties can be generated with graphical command or typing the number of

the two bars which define the single tie. Each cross-tie can be erased with the dedicated graphical ![icon] command or typing 0 in the grid.

The commands in the Tab bar are (from left to rigth):

- ![icon]   Print preview. Capture the image present in the main window and paste it in the Microsoft XPS Document Writer (useful to print the diagram directly on paper):

- ![icon]   Print numerical results for ductility check. Not active in the present display of stirrups.

- ![icon]   Copy. This command copy the image at moment displayed in the main window in the clipboard memory so you can paste it in any document you want.

- ![icon]   This command generate a dxf file of the image at moment displayed in the main window.

- ![icon]   Start the graphical generation of a new cross-tie by clicking with mouse on any two longitudinal bars

- ![icon]   Erase a cross-tie by clicking on it with mouse

Diameter and pitch of hoops and diameter of cross-ties can be changed in the corresponding text-boxes.

Cross-ties can be also generated typing in the grid (or type 0 to erase).

2.3.4   Ductility check

Especially for sections of seismic structures may be useful a direct calculation of the curvature ductility factor [eq. (5.4),(5.5)Ec8] by means the calculation of moment-curvature diagram.

Program is able to generate moment-curvature in biaxial bending too and for any section shape. For section with only one concrete region is also possible to obtain diagrams taking in account the confining of concrete by stirrups.

In the figure below is sown such a diagram generated for a confined rectangular column in seismic zone. Diagram can be saved as a dxf file, numeric results can be printed on the screen or on paper.

Ductility checks concern the following positions of sections of structures in seismic zones:

- within critical regions at the ends of all the beams (not secondary beams)
- within critical regions at the base of columns in DCM ductility class. (are excluded all the other critical areas of the columns on the upper floors on the first)
  - within all critical region of columns in DCH ductility class.
- within critical regions of ductile walls

To respect the local ductility in the above critical regions the curvature ductility factor μ_Φ must comply with the rule in §5.2.3.4 EC8:

$$\mu_\Phi \geq 1,5 \ (2q_0 - 1) \qquad \qquad \text{if} \qquad T_1 \geq T_C$$
(5.4) EC8

$$\mu_\Phi \geq 1,5 \ [1+2(q_0 - 1)] \ T_C / T_1 \qquad \text{if} \qquad T_1 < T_C$$
(5.5) EC8

where:

q_0 basic value of the behaviour factor

T_1 fundamental period of the building

T_C period at the upper limit of the constant acceleration of the spectrum

For beams EC8 allows to replace (5.4) above control with the equations (5.11), (5.12).
For columns EC8 allows to replace (5.4) above control with the relation (5.15).

Building a moment-curvature diagram of the cross-section it is possible to evaluate μ_Φ so as to allow a direct check of the above (5.4)-(5.5).

In the figure below is shown such a diagram generated for a general section of a column in seismic zone.

The moment-curvature diagrams can be of the following types:

- The first option generate a diagram using for concrete and steel the same stress-strain relationship used for design check but with the substitution of the design values with the mean ones.

- The second option refers to a diagram that take in account the confining of concrete core by stirrups and the spalling of concrete cover at strain strength. (See Ductility Checks 107)

- The third type refers to a concrete stress-strain relation parabola-trapeze (Kent-Park Type) whose parameters are assigned in the Materials Library 67.

- Last type generate a diagram base on the relation shown in §3.1.5 EC2. You can take into account a certain percentage of tension stiffening.

2.4   Rectangular sections of columns

This dialog window is opened if in General Data 9 Tab you have selected the typology "Rectangular Section of column".

For rectangular sections of the columns (the most common shape for columns in current applications) has been prepared a special type of calculation that, starting from a very quickly input (compared to that of the general sections 26), allows both the verification that the design of the reinforcement under biaxial bending and shear. It was also the possibility to perform the verification of the ultimate limit state of instability according to the simplified method of "nominal curvature 47" (§ 5.8.8 EC2).
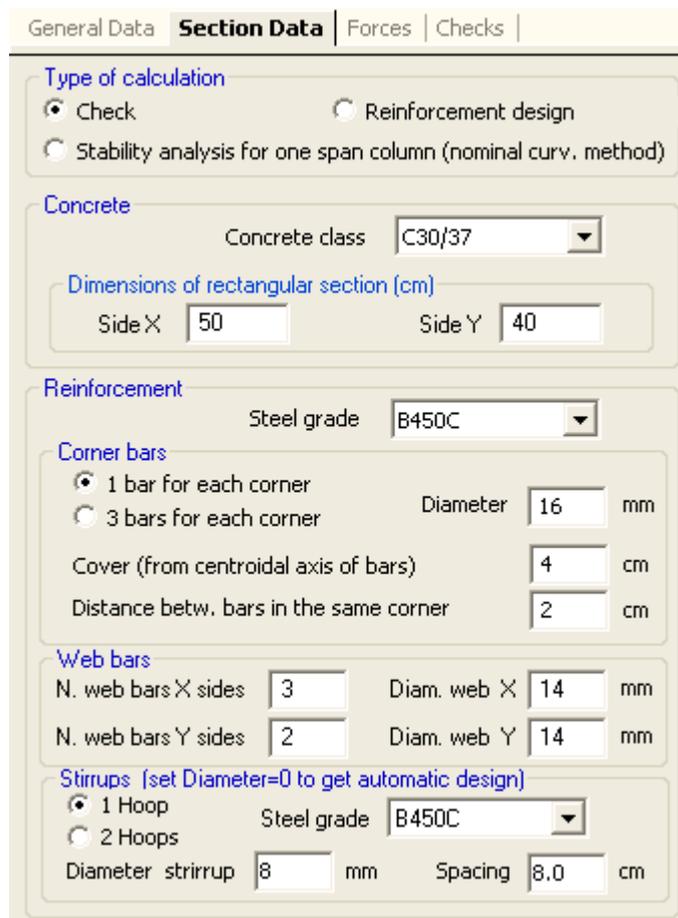
The simplifying assumptions (but also the most common in practice) relate to the dialog window shown in the above figure are:

- Rectangular section refers to the axes X, Y coincident with x, y principal axes of inertia of concrete section;
- Longitudinal reinforcement is doubly symmetrical with respect to the principal axes of inertia x, y;

- Corner bars all have the same diameter and concrete cover (measured from the centre of the bars).
- Web bars arranged along X direction have the same diameter that can be different from the remaining bars; the same for the web bars arranged along Y direction.
Below are listed each data to be entered in the above input window for check type of calculation:

• Concrete

in this frame can be assigned:
 - concrete class: to choose in the corresponding drop-down menu between the classes assigned in the Materials Library 67.
 - X side section: Measures in cm of the side parallel to the axis X.
 - Y side section: Measures in cm of the side parallel to the Y axis

• Reinforcement

 - steel grade: to choose in the corresponding drop-down menu between the classes assigned in the Materials Library 67.
 - corner bars: number of bar per corner for each vertex of the section (one or three) and diameter [mm] of the bars to be used.
 - cover: measured from centre of bars. No conformity check to EC2 rules - exposure class, strength class, etc. - is done by the program on these values.
 - distance between bars in the same corner: min distance measured between perimeter of bars (only if you have select three bars per corner)
 - web bars: number and diameter of intermediate bars between corners bars (along X and Y sides)
 - Stirrups: choose the number, the diameter and the pitch of initial hoops and the steel grade of stirrup bars. After the first calculation you can modify the pitch or the number of hoops and cross-ties in the Stirrups-Ductility 43 Tab. If you set the null value to the diameter (or to the pitch) the program performs the automatic design of stirrups 40 reinforcement using the user default diameters assigned in code and reinforcement options 70.

When you first open this dialog window, the default option is that of a single bar for each vertex. You can then select the other option that take three bars for each vertex also staring at the net distance between the three bars.

2.4.1   Reinforcement design

Given dimensions and strength class of the concrete section and the steel grade of longitudinal bars and stirrups you can require to the program a first design of longitudinal bars and stirrups selecting the design option in Section Data dialog window (Type of calculation frame). For a rectangular column sections such dialog appears as below:

Diameters of bars (longitudinal and for stirrups) used in design are those stored in Materials Library ⌐67⌐ (that you can modify as you want).

Longitudinal bars provided after the first run ("Checks" Tab selection) by present design calculation are distributed along the perimeter of the concrete regions. You can easily modify their number and diameter and run again for new checks.

After the first run in design mode the program provide only one hoop and the strictly number of cross-ties in seismic conditions ("DCM" or "DCH" ductility class). In the Stirrups-Ductility ⌐43⌐ Tab window you can modify the number of hoops (one or two) and the number and the definitions of cross-ties.

2.4.2 Results and diagram Mx-My

Selecting "Checks" Tab the program calculates and shows for each assigned combinations (ULS and SLS):

- [*Main window*] the graphical data (with eventual dimensions) of section with the concrete compressive area (red coloured) and the corresponding stress diagram (blue coloured); the black parallel line (orthogonal to neutral axis) represent the active strips shear resisting.

- [*Checks Tab window*] the principal results for Bending and Shear (capacity checks for ULS) and for Bending (stress control and crack widths for SLS).

- [*Information window*] the legend of longitudinal bars, the display options and the stress and strain of bars and of concrete vertices selected by mouse.

Command button of the above Checks Tab bar (from left to right):

-     Print preview. Capture the image present in the main window and paste it in the Microsoft XPS Document Writer (useful to print the diagram directly on paper):

-    Print Checks. Print data input and all the checks in a special Text window allowing a comfortable viewing; you can save its text to a file with *. doc, *. rtf, *. txt formats.

-    Copy.   This command copy the image at moment displayed in the main window in the clipboard memory so you can paste it in any document you want

-    Export Dxf. You can save in *.dxf file format the image at moment displayed in main window using a dialog box.

ULS INTERACTION Mx-My / N-Mx-Mx DIAGRAMs

If you select the ULS interaction Mx-My diagram option in Checks window program display the strength diagram Mx-My in the Main window:

STRENGTH DOMAIN Mx-My (ULTIMATE LIMIT STATE)

Section name: Column 13.secEC

File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\CALCOLI_SEZ\Column 13.secEC

N = 850 [kN] = Const.

My [kNm]

Mx [kNm]
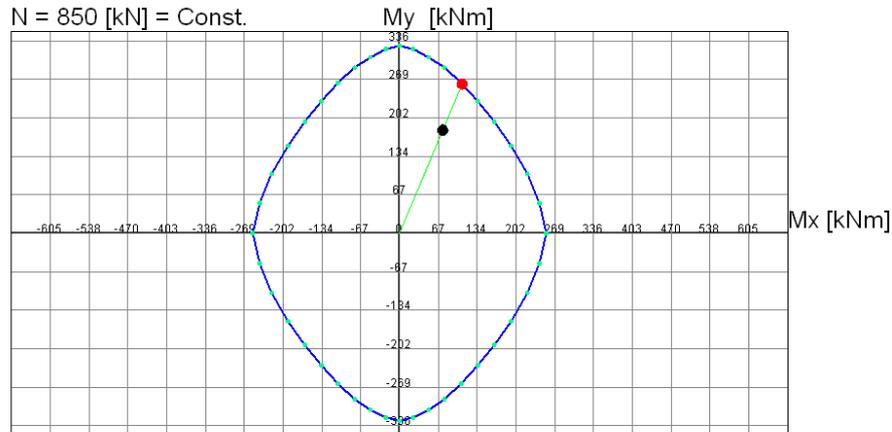
Grid spacing moments Mx, My = 67 kNm

The corresponding Information window shows:

Legend	N [kN]			Display	
• Calculated points	Mx [kNm]		N. interaction points	☐ Caluated points	
— Wireframe line	My [kNm]		76	☑ Grid	
— Reference axes	Drawing scale and grid spacing			☑ Forces assigned	
● Assigned forces				☐ Points numbering	
● Capacity forces			Spac. M(kNm) 122.3		

In N, Mx, My text boxes shall appear the values of calculated points of the domain boundary selected on the graph by mouse. Drawing spacings may be modified. Display options can modify the graphical aspect of the diagram.

If you select the ULS N-Mx-My diagram option program display the interaction tridimensional diagram N-Mx-My in the Main window.

2.4.3 Stirrups

After performing calculation (Checks Tab) you can display and/or change stirrups in the Stirrups-Ductility Tab window. In the below figure is shown an example of a column section designed for seismic zone with two hoops and two cross-ties as transverse reinforcement.
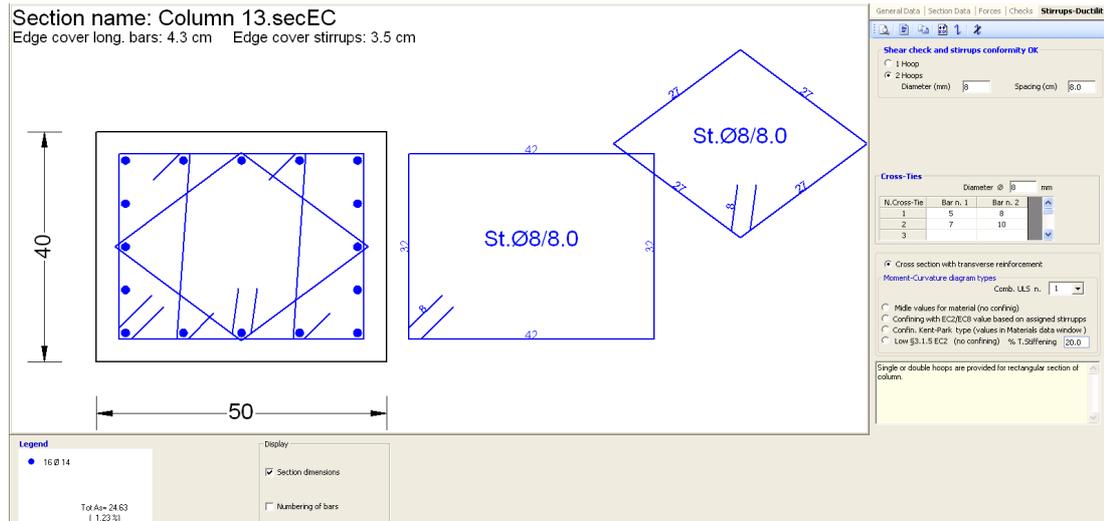
Options, text boxes and graphic commands are provided in the dialog box to change the initial proposal for hoops and cross-ties. For each change you make program performs an immediate shear check showing in the upper frame if it is Ok or Not:
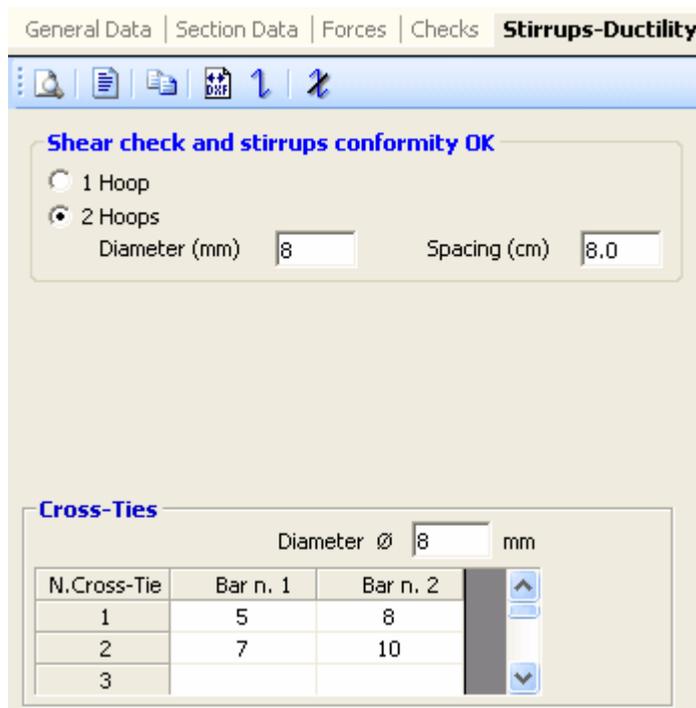
The commands in Tab bar are (from left to rigth):

- Print preview. Capture the image present in the main window and paste it in the Microsoft XPS Document Writer (useful to print the diagram directly on paper):

- Print numerical results for ductility check. Not active in the present stirrups display.

- 📋 Copy. This command copy the image at moment displayed in the main window in the clipboard memory so you can paste it in any document you want.

- 🗎 This command generate a dxf file of the image at moment displayed in the main window.

- ↥ Start the graphical generation of a new cross-tie by clicking with mouse on two longitudinal bars

- ✗ Erase a cross-tie by clicking on it with mouse

Diameter and pitch of hoops and diameter of cross-ties can be changed in the corresponding text-boxes.

Cross-ties can be also generated typing in the grid (or type 0 to erase).

2.4.4   Ductility checks

Especially for sections of seismic structures may be useful a direct calculation of the curvature ductility factor [eq. (5.4),(5.5)Ec8] by means the calculation of moment-curvature diagram.

Program is able to generate moment-curvature in biaxial bending too and for any section shape. For section with only one concrete region is also possible to obtain diagrams taking in account the confining of concrete by stirrups.

In the figure below is sown such a diagram generated for a confined rectangular column in seismic zone. Diagram can be saved as a dxf file, numeric results can be printed on the screen or on paper.

Ductility checks concern the following positions of sections of structures in seismic zones:

- within critical regions at the ends of all the beams (not secondary beams)
- within critical regions at the base of columns in DCM ductility class. (are excluded all the other critical areas of the columns on the upper floors on the first)
- within all critical region of columns in DCH ductility class.
- within critical regions of ductile walls

To respect the local ductility in the above critical regions the curvature ductility factor μ_Φ must comply with the rule in §5.2.3.4 EC8:

$$\mu_\Phi \geq 1,5\ (2q_0 - 1) \qquad\qquad \text{if} \qquad T_1 \geq T_C$$
(5.4) EC8

$$\mu_\Phi \geq 1,5\ [1+2(q_0 - 1)]\ T_C / T_1 \qquad\qquad \text{if} \qquad T_1 < T_C$$
(5.5) EC8

where:

q_0 basic value of the behaviour factor

T_1 fundamental period of the building

T_C period at the upper limit of the constant acceleration of the spectrum

For beams EC8 allows to replace (5.4) above control with the equations (5.11), (5.12).
For columns EC8 allows to replace (5.4) above control with the relation (5.15).
Building a moment-curvature diagram of the cross-section it is possible to evaluate μ_ϕ so as to allow a direct check of the above (5.4)-(5.5).

In the figure below is shown such a diagram generated for a rectangular section of a column in seismic zone (with confinement for core section).
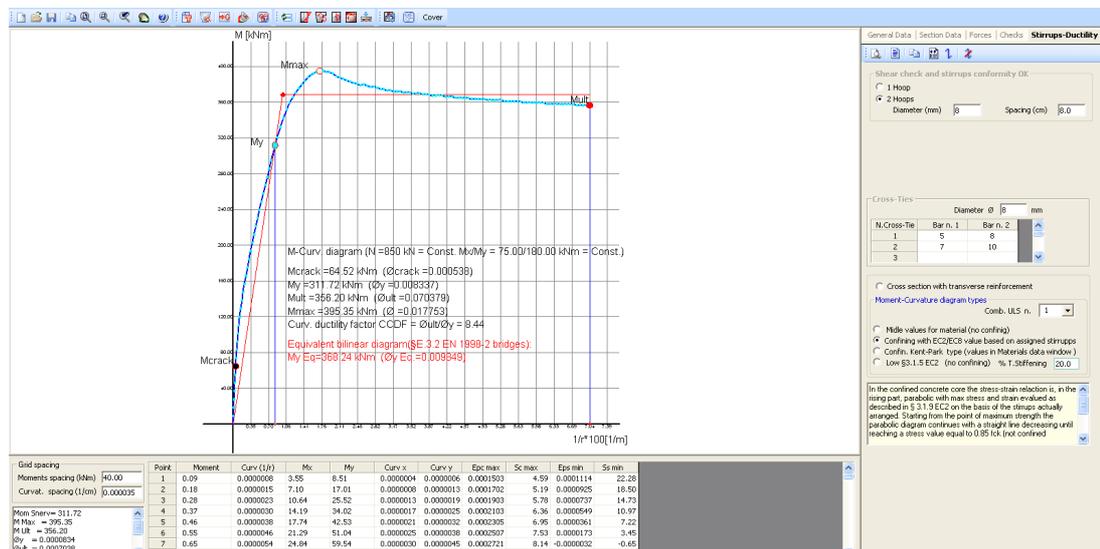
The moment-curvature diagrams can be of the following types:
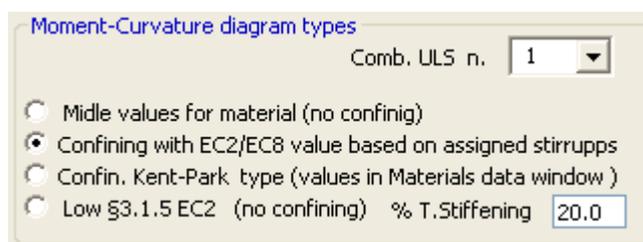
- The first option generate a diagram using for concrete and steel the same stress-strain relationship used for design check but with the substitution of the design values with the mean ones.
- The second option refers to a diagram that, only for one concrete region section, take in account the confining of concrete core by stirrups

and the spalling of concrete cover at strain strength. (See Ductility Checks[107])

- The third type refers to a concrete stress-strain relation parabola-trapeze (Kent-Park Type) whose parameters are assigned in the Materials Library[67].

- Last type generate a diagram base on the relation shown in §3.1.5 EC2. You can take into account a certain percentage of tension stiffening.

2.4.5   Instability check

In the present typology "rectangular section of column" (selected in General data[9] Tab) you can check the instability of the column with nominal curvature method[105] (§5.8.8 EC2). This method is primarily suitable for isolated member with constant normal force and defined length l_0 (to be pre-calculated by the user according to § 5.8.3.2 EC2).

The input windows for this calculation is as follows:

General Data | **Section Data** | Checks |

Type of calculation
- ○ Check
- ○ Reinforcement design
- ● Stability analysis for one span column (nominal curv. method)

Concrete

Concrete class | C30/37 ▼

Dimensions of rectangular section (cm)

Side X | 50   Side Y | 40

Reinforcement

Steel grade | B450C ▼

Corner bars
- ● 1 bar for each corner
- ○ 3 bars for each corner

Diameter | 16 | mm

Cover (from centroidal axis of bars) | 4 | cm

Distance betw. bars in the same corner | 2 | cm

Web bars

N. web bars X sides | 3   Diam. web X | 14 | mm

N. web bars Y sides | 2   Diam. web Y | 14 | mm

Stability data (Nominal curvature method §5.8.8 EC2)

Calculation with uniaxial or biaxial curvature 1/r
- ○ With uniaxial eq.(5.34)EC2   ● With biaxial M-Curv.diagram

Design constant axial force Nd | 500.00 | kN

Effective creep ratio (according eq.(5.19)) | 2.00

Data for Y axis stability direction

First order moment MXod (without imperfection) | 50.00 | kNm

Eccentricity eiY (=loY/400 see §5.2(7)) | 2.3 | cm

Effective length loY (according to §5.8.3.2) | 900.0 | cm

Data for Y axis stability direction

First order moment MYod (without imperfection) | 100.00 | kNm

Eccentricity eiX (=loX/400 see §5.2(7)) | 2.0 | cm

Effective length loX (according to §5.8.3.2) | 700.0 | cm

Longitudinal Bars
- ● 4 Ø 16
- ● 10 Ø 14

Tot As= 23.44
(1.17 %)

Geometrical data of concrete section
Axes X,Y of reference
Area = 2000 cm²
X centroid = 0.000 cm
Y centroid = 0.000 cm
SX = 0 cm³
SY = 0 cm³
JX = 266667 cm^4
JY = 416667 cm^4

Display
- ☑ Reference axes
- ☑ Section dimensions
- ☐ Concrete vertex numbering
- ☑ Numbering of bars
- ☑ Reference grid

Grid spacing (cm) | 10.0
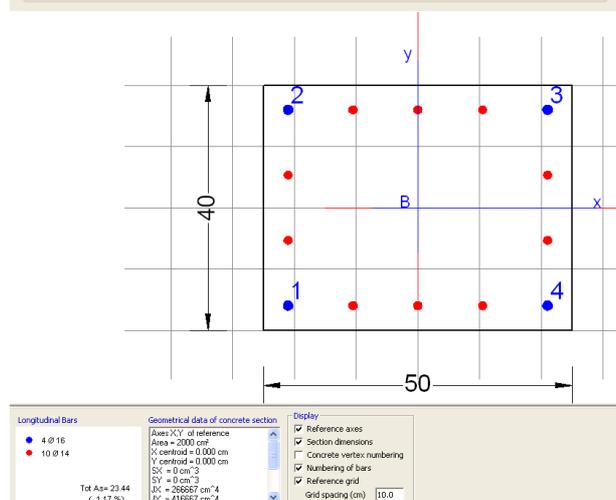
The relevant input data in this window are:

- Dimension of side X and Y of the concrete rectangular section
- Concrete Class and Steel grade
- Number and diameter and cover of corner bars and web bars
- Design constant axial force N_d
- Effective creep ratio according to eq. (5.19) EC2: $\varphi_{ef} = \varphi_{(\infty,t0)}$ MX_{0qp}/MX_{0d} where MX_{0qp} is the first order moment in SLS quasi-permanent combination
- First order bending moment in ULS combination of interest: MX_{0d}
- Eccentricity eY: $= l_{0Y}/400$ to take into account geometric imperfections according to simplified alternative in § 5.2(9) EC2.
- Effective length l_{0Y}: to evaluate in advance according to § 5.8.3.2 EC2

The curvature $1/r_0$ in eq (5.34) can be assessed by the program (see in above figure the option "Mode of calculation of curvature 1/r") with the direct formula in § 5.8.8.3 EC2 or by means the moment-curvature diagram as in "model column" method. In biaxial bending it is difficult to use the direct formula (5.34) of § 5.8.8.3 EC2 as it is hard to meet the conditions of applicability (5.38a) and (5.38b) EC2. The biaxial moment-curvature diagram provided without problems by the program make always possible to check biaxial instability. In the below results such a diagram is shown together with the windows to print numerical results. The graphic of diagram can be save as dxf file or printed directly on paper.

MOMENTS-CURVATURES DIAGRAM (Nd = 500 daN)

Section name: Rectangular_Column_Instability.secEC

File path: C:\SORGENTI_FVGS-RC-SEZ-EUROCODE\CALCOLI_SEZ\Rectangular_Column_Instability.secEC

General Data | Section Data | **Checks**

COLUMN STABILITY - Nominal Curvat

COLUMN STABILITY WITH NOMINAL CURVATURE METHOD [§5.8.8EC2]
[Curvature 1/r derived from biaxial Mom-Curv. diagram (see 'Model Column' method)]

INTERNAL FORCES AND RESULTS

Constant design axial load Nd:	500.00	kN
Effective creep ratio [according to eq.(5.19)] Kf:	1.35	
Bending first order moment around X axis MoX':	50.00	kNm
Effective lenght of column in Y direction (§5.8.3.2) loY:	900.0	cm

Slendemess ratio with respect to Y dir. LY = loY/iY:	77.9	
Min Slendemess ratio (eq.5.13) Lmin:	32.4	
Eccentricity of Nd by imperfections in Y direction eiY:	2.30	cm
Total first order moment around X: MoX=MoX'+ Nd*eiY:	**61.50**	**kNm**
Bending first order moment around X axis MoY':	100.00	kNm
Effective lenght of column in X direction (§5.8.3.2) loX:	700.0	cm
Slendemess ratio with respect to X dir. LX = loX/iX:	48.5	
Min Slendemess ratio (eq.5.13) Lmin:	32.4	
Eccentricity of Nd by imperfections in X direction eiX:	2.00	cm
Total first order moment around Y: MoY=MoY'+ Nd*eiX	**110.00**	**kNm**
Vectorial total first order moment Md:	**126.02**	**kNm**

M 1^ord. available=170.95 kNm

M 2^ord. unavailable=58.98 kNm

1/r [1/c

Moments grid spacing = 20 kNm   Curvatures spacing = 0.000002397 1/cm

ble Moments   [kNm]

vailable Mom. [kNm]

1/r [1/cm]

Display
☑ Grid reference
☑ Calculated points

Section1/1 | Page3/3 | Line24 | Col0 | 100 % | NUM

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2.5   Rectangular section of wall

The rectangular sections of the walls must have an aspect ratio greater than 4 and for seismic design (cantilever behavior with plastic hinge at base and bending moments without sign reversal at least the first two floors) shall have a minimum cross-dimension 1.5 meters in the case of low-rise buildings and at least 2.0 meters tall buildings. The seismic wall sections are characterized by confinement, with special hoops at the end zones in order to confine the vertical reinforcement that resists to flexural strength. The central area of the section (web) is reinforced with horizontal and vertical bars mainly intended to absorb the shear stresses. There are two main types for the seismic walls (see EC8 in /2/₁₇₃):

- Ductile walls: with a strong degree of interlocking the base and sufficiently slender to be able to develop a plastic hinge at the base (the remaining part of the wall must instead remain in the elastic range). The design can be done either in DCM or in DCH.

- Lightly reinforced walls: whose considerable transverse dimension (> 4 m or > 2/3 height of the wall) makes impossible the formation of a plastic hinge and can not prevent the rotation at the base. The design is expected only in DCM.

- Wall outside the critical area: the sections located above the critical zone not strictly require the confinement with special stirrups of the end zones, however it is recommended in any case its prediction even if they do not follow the rules of transverse

reinforcement detail of EC8 code in the critical zone. Even in the absence of stirrups of confinement (or for non seismic wall) we recommend to define the length of the critical area for the sole purpose of the positioning of vertical reinforcement of the extremities.

Section Data dialog window is the following:

Principal data to enter in the above dialog window:

Type of wall

- Ductile wall: in the General Data⌐9¬ window you must preliminary enter the position of the current section into the wall: within or out the height of the critical region [see §5.4.3.4.2(1)]. In General Data should also entered the ductility class (DCM or DCH). In CDH the conformity check is much more difficult then in CDM. In DCH if the ratio between the height to length is not greater then 2.0 the wall is "squat": this fact is to be entered in the pertinent check-box.
- Large lightly reinforced wall: boundary region must be always confined also out of the critical region. Ductility class can be only CDM.

Concrete section data

For this type of section is expected only checks (no design of reinforcement).

Are to be entered the concrete class, the sizes of the rectangular section and the length L_c of the confined boundary elements measured by the

distance between the axes of the confining hoops (as you see in below figure). Main vertical bars for bending will be placed within the width L_c. In the case in which the amount of vertical bars to be placed in the boundary elements is insufficient, the measure L_c previously assigned will be increased. The same in the case when the compressed area (due to bending deformation) with strain exceeding 0.0035 should fall outside of L_c (that is if the strain ε_{cu2} in the figure below exceeds 0.0035).   As clarification of the adopted symbols, see the graph below:
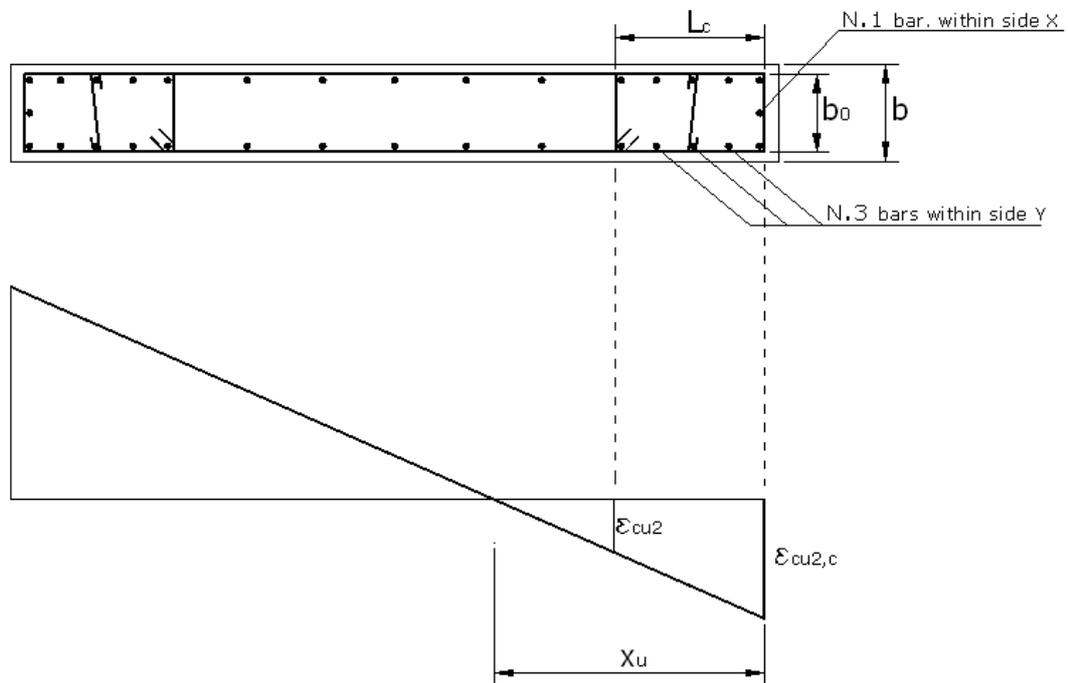
Reinforcement of each confined boundary region

Must be assigned the steel grade, the number, diameter and cover of intermediate bars (over the fixed 4 corner bars) placed along sides parallel to X axis and along sides parallel to Y axis. The four corner bars are not to be entered as they are considered always present in the boundary elements. Diameter and pitch of confining hoops complete the input data of this reinforcement.

Reinforcement of not confined region (web)

In the web region must be entered the diameter and the number of vertical and horizontal bars assegnati placed along each two side of the web.

Calculation data

In the case of large lightly reinforced wall must be entered only the behaviour factor q. For ductile wall in CDM must be entered the basic behaviour factor q_0 , T_1 (fundamental period of vibration of the building) and T_c (upper limit period of the constant spectral acceleration region of the spectrum). For ductile wall in CDH are also necessary the spectral values $Se(T_1)$ and $Se(T_c)$.

2.5.1   Stirrups

After performing calculation ("Checks" Tab) you can display and/or change stirrups in the "Stirrups-Ductility" Tab window. In the below figure is shown an example of a section within critical region of a ductile wall designed for seismic zone. Note the two hoops in the boundary regions and the cross-ties in the same regions. In the web region of the wall (out of the boundary confined regions) are provided n.9/m² cross-ties but them are not displayed in the graph but only indicated in the text.
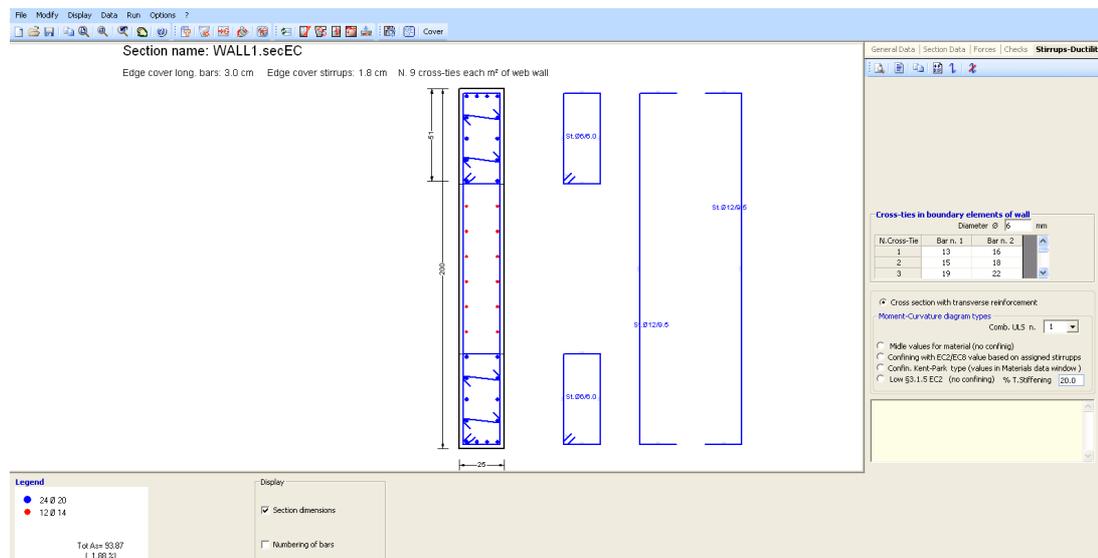
Only cross-ties can be added or erased in the grid. After the eventual changes you can check again the section clicking the "Check" Tab.

2.6 Polygonal sections in Torsion

The dialog Tab window for polygonal section (hollow or not) with only torsion force acting is:
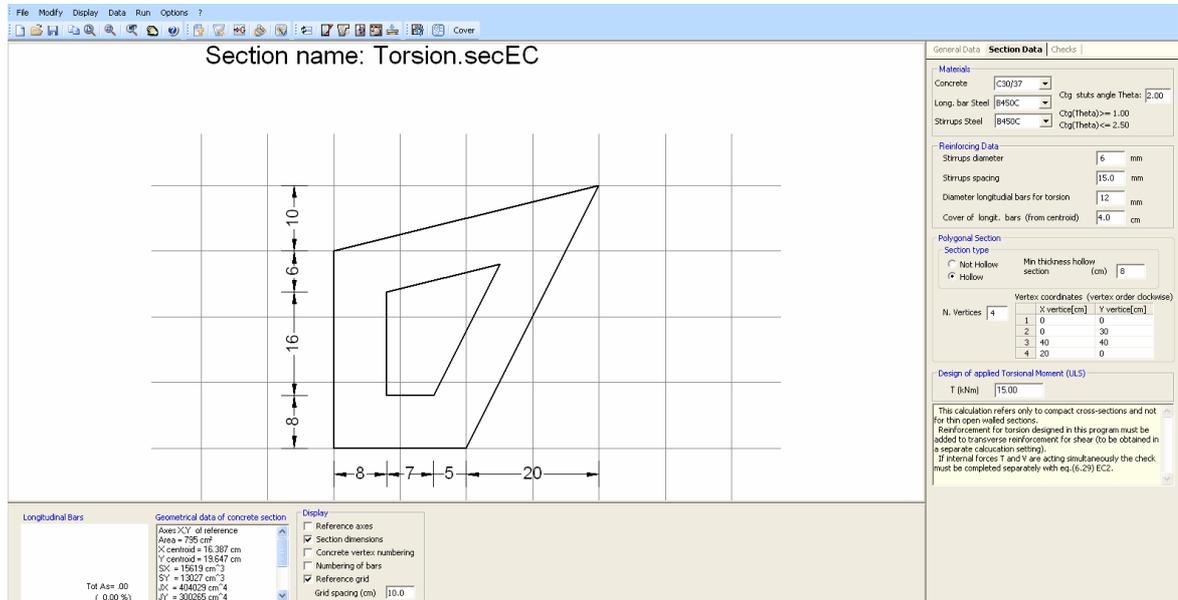
As usual in the data input dialog are to be assigned geometrical data of the concrete region (hollow or not), materials data and steel bar (stirrups and longitudinal for torsion only) details (diameter, pitch).

An important input data is the Cot(Theta) where Theta is the angle ϑ of inclined struts in the web (§6.2.3(2) EC2). Cot(Theta) is limited by values assigned in Code and reinforcement options⌐70⌐ window (National Annex). If the section is also subjected to shear force a rule of standards requires that you must use (in the two separate check) the same value for Cot(Theta); the designed stirrups areas for the two forces (shear and torsion) can be summed and should be checked (manually by the user) with the following interaction relationship about concrete resistance [eq.(6.31) EC2]:

$$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max} <= 1.00$$

For predefined section the above check is, instead, automatically performed by the program.

2.6.1 Results

The output window is (the position of bars is only indicative):
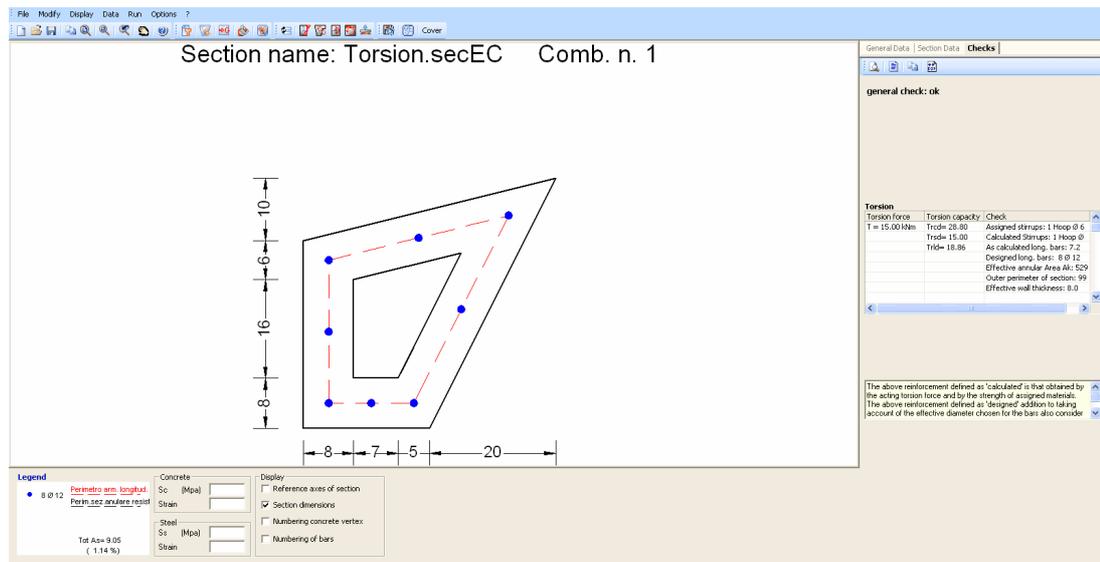
In the output Check windows is present the usual Tab Bar with its options.

The text printing output (for checks results only) for the showed example is:

CHECKS RESULTS

Design value of applied torsional moment Tsdu:	15.00	kNm
Torsional resistance by concrete stuts Trdu:	28.80	kNm
Torsional resistance developed by assigned stirrups Trwsd_A:	15.62	kNm
Torsional resistance developed by minimum quantity of stirrups Trwsd:	15.00	kNm
Assigned Spacing of stirrups :	15.0	cm
Spacing of stirrups of assigned diameter strictly necessary for torsion:	15.6	cm
Torsional resistance developed by the yielding of longitudinal bars Trwld:	18.86	kNm
Design Torsional cracking moment:	3.49	kNm

Torsion check OK

Area enclosed by the external perimeter of section Ac =	1000	cm²
The perimeter of concrete section uc =	136	cm
Effective wall thickness teff =	8.0	cm
Area enclosed by the center line of walls Ak =	529	cm²
The perimeter of area Ak uk =	99	cm

Assigned stirrups	1 Stirrup Ø 6 /15.0	(1.9 cm²/m)
Stirrups strictly necessary:	1 Stirrup Ø 6 /15.6	(1.8 cm²/m)
Area calculated long. bars for torsion Asl =	7.2	cm²
Designed longitudinal bars for torsion:	8 Ø 12	(9.0 cm²)

Note   The above designed reinforcement refer to the assigned diameters and to the detailing of code [§9.2.3 EC2]. The number of designed longitudinal bars is such as to allow their spacing to be not greater than 35 cm.

2.7   SLS deflections in single beam

Last typology that you can select in the General Data⌐9⌐ window is the SLS deflection calculation of a single isostatic beam for the verification of limit state of deformation.

The beam can be hyperstatic as belonging to a frame, but solving the frame, for the SLS combination of interest (usually quasi-permanent), we make the beam isostatic applying two bending moments at the end (see main window in below figure). Alternatively the beam may be a cantilever.

In general maximal sag of a beam must not exceeds span/250 if subject to a quasi-permanent loads. Deflection <u>after construction</u> that could damage adjacent part of the structure must be limited to span/500.

You can directly check (no check is performed by the program) the limit state of deformation comparing the above limit values (or the limit value that you choose) with a calculated deflection performed by the program according to the most rigorous method described in §7.4.3(7) EC2 by double integration of the differential equation of the bending and computing curvatures at frequent section along the member.

The calculation take in account cracking, creep, aging and shrinkage as mentioned in the topic Limitation of Deformation⌐126⌐.
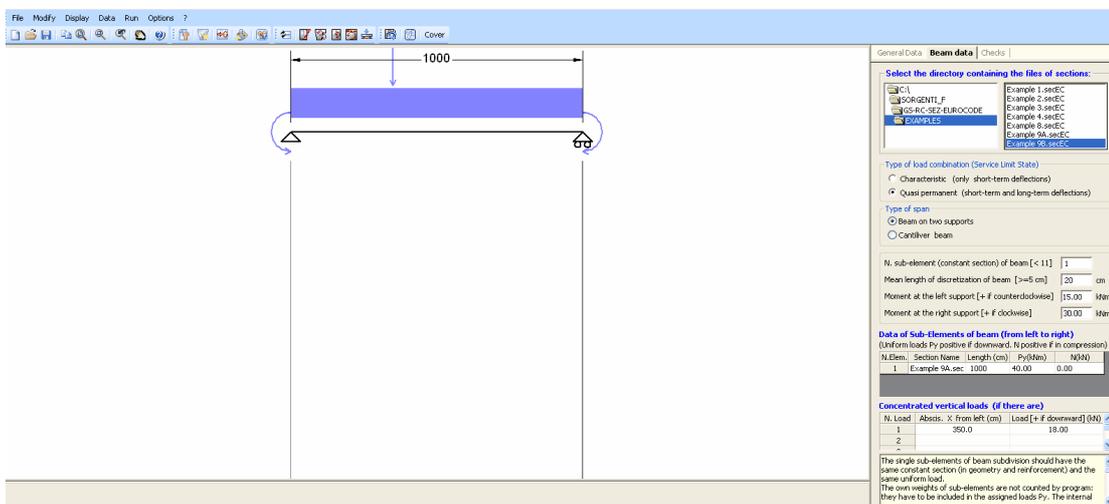
Beam can be assigned as consisting as one or more sub-elements: each sub-element is characterized by a constant section (as geometry and reinforcement) and a distributed load acting on it. Two sub-element may be different only for different placing of longitudinal bars or for different distributed loads. Before of this input it is necessary to input ad save with a different name the above section that define the different sub-element (even a single). The section to assign may be predefined or general but with uniaxial bending forces and must be saved in a single directory. Creep

and shrinkage coefficients assigned (in Materials library 67) to each section are taken in account in the deflections calculation of the beam.

The input begin with the selection of the directory (no selection of file) in the above dialog box. Then you must define the type of load combination (Characteristic only for short term deflections or quasi-permanent for short and long term deflections). The selection of the type of span complete this first part of input.

The data of each sub-element are (in this example there is only one):

- Section Name: to select in the first column of the grid (in this column are present all the file *.secEU within the directory selected before)

- Lengh [cm]: of each sub-element (the sum of all sub-elements is the span of the beam)
- Py [kN/m]: if the uniformly distributed load <u>included self-weight</u>.
- N [kN]: Axial force (acting in the centroid of all sections). May be useful for evaluation of viscous shortening of columns in a multi-storey building.

Eventual concentrated vertical loads can be assigned (as in above grid) with their abscissa [cm] from left support and value [kN].

RESULTS
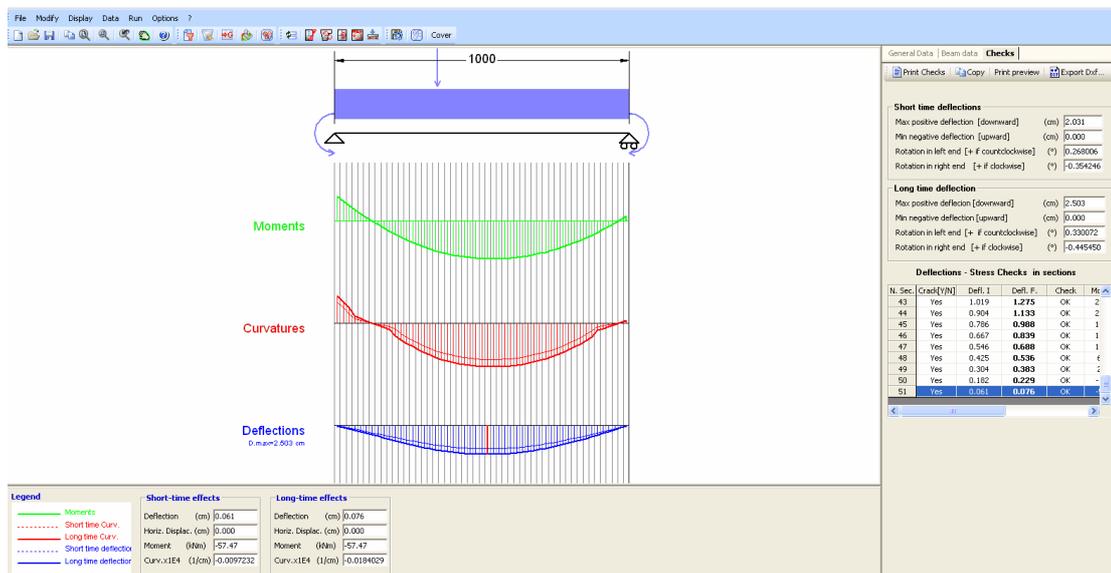
Moving the mouse along the beam length you can read the single section results in the information window (bottom of the screen).

Deflection results are distinct in short time (only cracking if it occurs and no rheological coefficient applied) and long time deflection. Stress check is also performed by the program.

The above command Tab bar contains the usual tools seen in the other typology.

After Print command the printing output is:

File Modifica Formato

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DEFLECTIONS - CURVATURES - AXIAL DISPLACEMENTS OF BEAM SECTIONS

N.Sec:	Progressive order number of the section (from beam discretizzation) from left to right
X sec:	Distance of sections from left end of the beam
Sec.Name:	File name of section at X distance
Check:	OK/NotOK = conformity of max stresses to the critical stresses of materials
Crack:	Y/N = section cracked or not cracked (concrete stresses < tensile strength)
Df. I:	Short time deflection [cm] [+ if downward] in the current section
Cur. I:	Short time curvature [1/cm] in current the section
A.D. I:	Short time axial displacement [cm] of section (+ if form right to left)
Df. F:	Long time deflection [cm] of current section [+ F downward]
Cur. F:	Long time curvature [1/cm] of current section
A.D. F:	Long time axial displacement [cm] of section

N.Sec	X	Sec. Name	Check	Crack	Df. I	Cur. I	A.D. I	Df. F	Cur. F	A.D. F
1	9.8	Example 9A.secEC	OK	S	0.046	-0.0000127	0.000	0.056	-0.0000165	0.000
2	29.4	Example 9A.secEC	OK	S	0.142	-0.0000092	0.000	0.176	-0.0000130	0.000
3	49.0	Example 9A.secEC	OK	S	0.243	-0.0000055	0.000	0.300	-0.0000095	0.000
4	68.6	Example 9A.secEC	OK	N	0.345	-0.0000023	0.000	0.428	-0.0000043	0.000
5	88.2	Example 9A.secEC	OK	N	0.448	-0.0000015	0.000	0.558	-0.0000028	0.000
6	107.8	Example 9A.secEC	OK	N	0.552	-0.0000007	0.000	0.688	-0.0000013	0.000
7	127.5	Example 9A.secEC	OK	N	0.656	0.0000001	0.000	0.820	0.0000001	0.000
8	147.1	Example 9A.secEC	OK	N	0.760	0.0000008	0.000	0.951	0.0000015	0.000
9	166.7	Example 9A.secEC	OK	N	0.863	0.0000015	0.000	1.081	0.0000028	0.000
10	186.3	Example 9A.secEC	OK	N	0.966	0.0000021	0.000	1.211	0.0000040	0.000
11	205.9	Example 9A.secEC	OK	S	1.069	0.0000037	0.000	1.339	0.0000078	0.000
12	225.5	Example 9A.secEC	OK	N	1.170	0.0000064	0.000	1.464	0.0000103	0.000

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effects

(cm)

ec. (cm)

(kNm)

2.8 Forces

Selecting "Forces" Tab the program shows the underlying dialog windows that make possible the input of load combination forces (up to 60 for ULS and SLS).
This window automatically varies according to the typology of the section.
The two figures below show the windows for a predefined section (the left one) and for a general section (the right one).

General Data | Section Data | **Forces** | Checks |
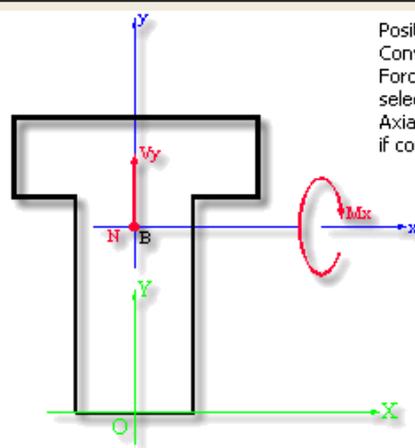
Positive Sign
Conventions of
Forces in the
selected reference.
Axial force N positive
if compressive

Forces reference system (N always centroidal)
⦿ Axes x,y // axes X,Y of reference. N applied at centroid

Type of bending
⦿ Uniaxial (neutral axis always // axis X of coord. reference)
○ Biaxial (exact calculation based on N, Mx, My values)

Combination type

ULS Ultimate limit states: n. load combinations	4
SLS Serviceability limit states: Characteristic load comb.	0
SLS Serviceability limit states: Frequent load comb.	0
SLS Serviceability limit states: Quasi permanent load com	0

Internal forces   Import forces from file .txt

	N [kN]	Mx [kNm]	Vy [kN]	MTor[kNm]
1	600.00	100.00	0.00	0.00
2	2000.00	100.00	0.00	0.00
3	5000.00	100.00	0.00	0.00
4	10000.00	100.00	0.00	0.00
5				
6				
7				

Force reference system (N is always applied at the centroid of the concrete section)

For general sections [26] (right figure) are provided two reference systems (x,y axes principal of inertia or axes X',Y' parallels to X,Y coordinates axes); type of bending and shear can be uniaxial (X' axis is always parallel to X) or biaxial. Internal forces MX,MY are bending moments, VX,VY are shear forces, N is axial force: sign conventions are showed in the scheme placed in the upper part of the figure.

For predefined sections [12] (left figure) reference system can be only that parallel to X,Y coordinate axes and bending moments and shear forces cam be only uniaxial. Internal force Mx is the bending moment, Vy is the shear force, N is axial force and T is the torsional force: sign conventions are showed in the scheme placed in the upper part of the figure.

For rectangular sections of columns[38] and for rectangular sections of walls general coordinates system overlaps with local system x,y (axes principal of inertia)

Type of bending

For predefined sections the type of bending is only uniaxial. For rectangular sections of columns and walls is only biaxial.

Only for general section it is possible the choose between uniaxial (neutral axis forced to be parallel o X axis) or biaxial bending (and shear).

Combination type

Selecting in this box one of the four type of load combinations (the first relates to ultimate limit states the subsequent three to serviceability limit states) are shown, in the underlying grid, the corresponding internal forces assigned or to assign. It is required to assign at least one ULS combination.

Internal forces

In this grid you can enter, in the pertinent columns, the internal forces acting on the section. The type of forces vary depending on the type of section and of the reference system assumed above.

You can assign forces importing their values by a text file previously prepared (see Import forces from text file[65]).

You can also assign forces pasting with the mouse columns of values from a different grid (excel etc).

2.9   Import general section from dxf file

From the drop-down menu "File" you can select the command Import general section from dxf file. So the below importation window allows you to import one or more polygonal concrete regions and (optionally) any number of bars with their own diameters. The imported section will be considered by program a general section 26 typology.
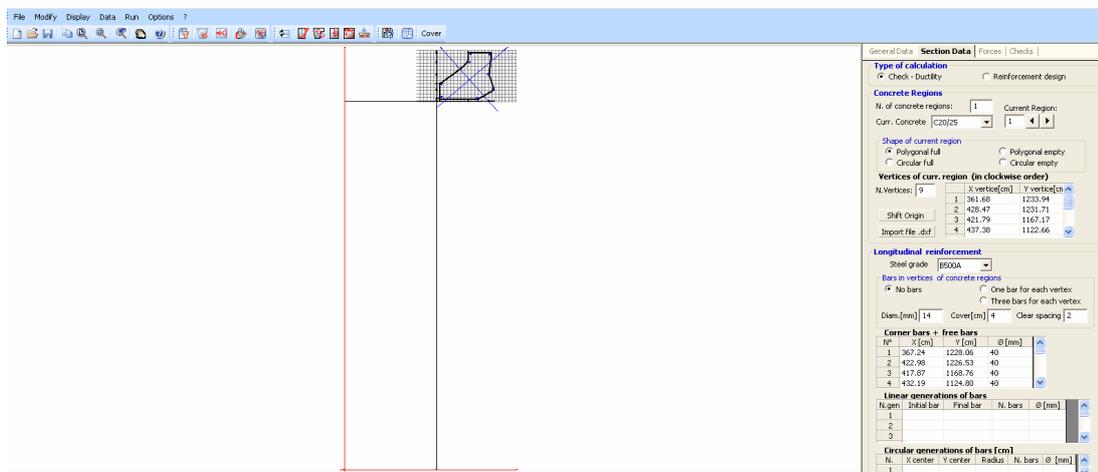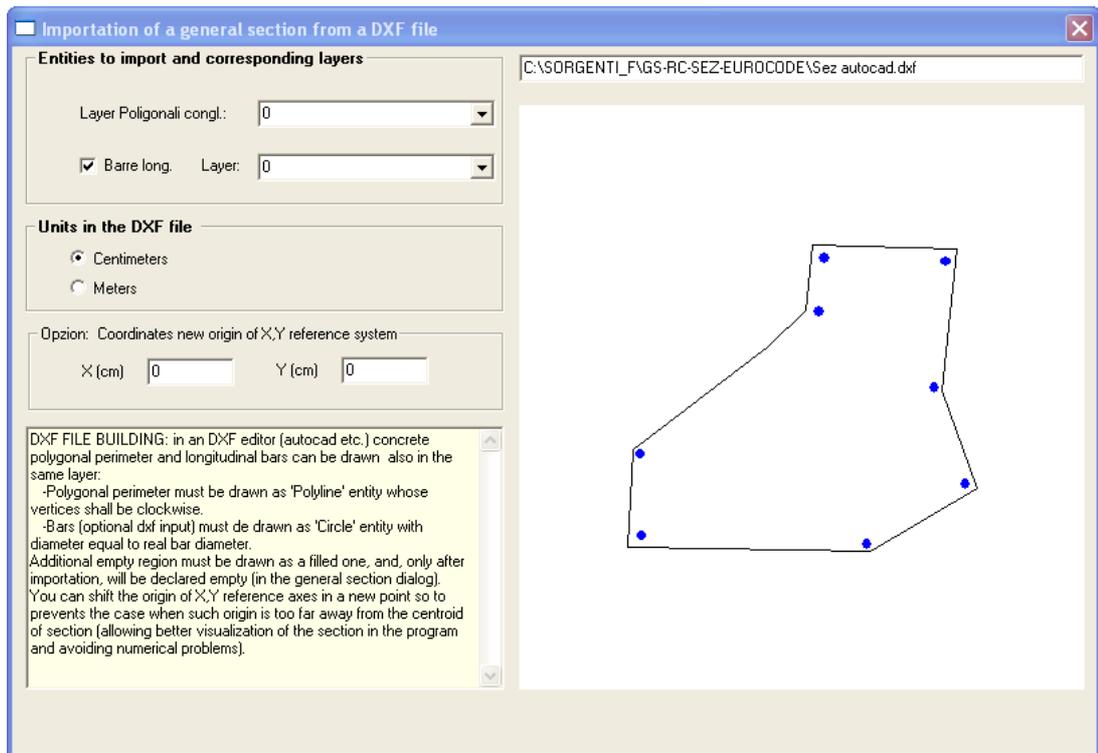
The procedure to build the dxf file is specified in the information text box of the above importation window. If after the importation of file the

section results very far from X,Y axes origin (as in the present example) you can shift the centroid to the origin X,Y axes by means of Shift Origin command in the general section window.

The dxf file of the simple polygonal section shown in the above figure is saved in the "\Examples" directory with the name "Sec autocad.dxf".

2.10   Import forces from text file

If the section has a lot of load combination forces to check can be usefull to buid a text file containing such forces in a format defined in the following.

Once defined any section typology from the drop-down menu file should be selected the command *Import forces from text file.* After opening of below window you must select in the upper text box, the file name (already built and saved) to import. Select also the unit of force written in the text file. Reversal also eventual signs of forces for different sign conventions.

The numbers in the text file must contain the point (not a comma) as the decimal separator. The numbers of the same row must be separated by a comma or a space. The rows are separated by a carriage return (enter).

The first row should contain the number of the ordered combinations ULS, SLS characteristic, SLS frequent, SLS quasi-permanent, following subsequent rows. Forces to enter into individual rows must comply with the format shown in this dialog import. For example, if you want to import two comb. ULS and one comb. SLS quasi-permanent the row to type is: 2, 0, 0, 1

All subsequent rows have the same format consists of six terms per row: N, Mx, My, Vx, Vy, T for all the forces that both ULS and SLS

Obviously, if the section is subject only to My Vx forces, the remaining ULS forces will always be null, while in the SLS combinations SLS forces will be null My, Vx, Vy, T. So each row should always contain 6 numbers (although some of them are necessarily zero).

As the units of the imported forces may be different from those used by the program must be assigned (in the appropriate options) the units on forces to import. A similar assignment is made for signs of forces to import

In order to reverse (or keep) the correct sign of the forces in the file, please note that the program considers positive:

    - Axial force N if compressive.

    - The shear forces Vx, Vy if bound as the positive direction of the reference axes.

    - Bending moment Mx if it compresses concrete fibers having max value of y-coordinate.

    - Bending moment My if it compresses concrete fibers having max value of x-coordinate.

Below is an example of a text file that corresponds to a generic section with two comb. ULS and one comb. SLS quasi-permanent:

2,0,0,1
50.00, 60.00,25.00,80.00,10.00,0
0, 80.00, 45.00,20.00,15.00,0
70.00,46.00,15.30,0,0,0

This text file named "FORCES_FROM_FILE.txt" is saved for proof in the "Example" folder of the program.

2.11 Materials Library

This is the button in the Tab bar to open the library of the materials data (the same selecting "Materials Library" from the drop-down RUN menu). In this window all tension are expressed in Mpa (N/mm²).

At starting of the program or of a new calculation this window show the default library which may be amended or extended. To save the library as default library for the next calculations save it with the command "Set as default". To save the current library (after modifications) only for current section save it with the command "Set for current calc." (this current library is saved together with the data of the section.

Material Library

Concrete

N°	Class	Values based on fck	Ecm	fcm	fck	fcd	fctd	fctm	Creep coefficient	Shinkage coefficient	Aging coefficient	n coeff.
1	C20/25	✔	29962	28.0	20.0	11.3	1.0	2.2	2.5	.00025	0.8	
2	C25/30	✔	31476	33.0	25.0	14.2	1.2	2.6	2.5	.00025	0.8	
3	C30/37	✔	32837	38.0	30.0	17.0	1.4	2.9	2.5	.00025	0.8	
4	C35/45	✔	34077	43.0	35.0	19.8	1.5	3.2	2.5	.00025	0.8	

Reinforcing Steel

N°	Grade	Es	fym	fyk	fyd	ftk	k =(ftk/fyk)	e_uk	e_ud	k=(ftd/fyd) for design
1	B500A	200000.0	530.0	500.0	434.8	525.0	1.05	0.025	0.0225	1.00
2	B500B	200000.0	530.0	500.0	434.8	540.0	1.08	0.05	0.0450	1.00
3	B500C	200000.0	530.0	500.0	434.8	575.0	1.15	0.075	0.0675	1.00
4	B550A	200000.0	580.0	550.0	478.3	580.0	1.05	0.025	0.0225	1.00

Serviceability Limit States (max stresses - max crack width)

Exposition Class	Combination Type	Stress concr. % fck	Stress reinf. % fyk	Crack width mm
XC0, XC1	Charact., Frequent	60.0%	80.0%	----
XC0, XC1	Quasi-permanent	45.0%	80.0%	0.40
XC2,XC3,XC4,XD1,XD1,XD2,XD3,XS1,XS2,XS3,XF,XA	Charact., Frequent	60.0%	80.0%	----
XC2,XC3,XC4,XD1,XD1,XD2,XD3,XS1,XS2,XS3,XF,XA	Quasi-permanent	45.0%	80.0%	0.30

B550B · fyd ftd 0.045
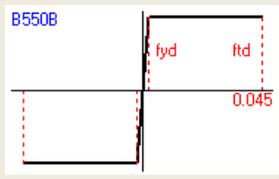
[Set as default] [Set for current calc.] [Cancel and Exit]

CONCRETE

Class: Name of the strength class of the concrete by means a string of no more than 10 alphanumeric characters. The normalized classes to edit are those present in the table 3.1 EC2 on the basis of the characteristic cylindrical strength f_{ck} and cubic R_{ck}.

☑ Values based on fck: if this box is checked Ecm, fcm, fcd, fctd, fctm (column with dark background colour) are automatically calculated by program typing the only fck value. Their values are defined on the basis of formulas in table 3.1 EC2, of partial safety factor $_c$ in table 2.1N EC2 and of code setting adopted by user (National Annexes) in the window dialog "Code and reinforcement options[70]". If the box is not checked you can freely change all the above values.

Ecm: Mean elastic E_{cm} modulus (Mpa). In table 3.1 EC2 Ecm = 22000 $[(f_{cm}/100]^{0.3}$ Mpa

fcm: Mean compressive cylindrical strength $f_{cm} = f_{ck} + 8$

fck: Characteristic compressive cylindrical strength f_{ck}

fcd: Design compressive strength $f_{cd} = a_{cc} f_{ck} / _c$ with $_c = 1.5$ and a_{cc} = coefficient taking account of long term effects defined in the window Code options 70 (recommended value 1.0)

fctd: Design tensile strength $f_{ctd} = 0,7 f_{ctm} / _c$

fctm: Design mean tensile strength $f_{ctm} = 0.3 (f_{ck})^{2/3}$ if $f_{ck} \leq$ 50 Mpa; $f_{ctm} = 2.12 \ln[1 + (f_{cm}/10)]$ if $f_{ck} > 50$ Mpa

Creep coeff.: Coefficient (t,t0) [eq.(B.1) in B.1 Annex B EC2] used in SLS stress determinations based on AAEM and EM method and in SLS deflections calculation 56. Default value are to be changed in base to the real case study.

Shrinkage coeff.: Coefficient e_{cs} [eq. (3.8) EC2]. Same considerations in above creep coeff.

Aging coeff.: Coefficient (t,t0) used in SLS stress determinations based on AAEM method and in SLS deflections calculation 56. For t=∞ use χ=0.8. In EM method χ=1.0

n omog.: E_S/E_C used in SLS stress simplified calculation (n = 15 as common default value).

fc max: max strength of confined concrete in a stress-strain relation Kent-Park type (for ductility check with moment-curvature diagrams)

fc ult: strength of confined concrete at max compressive confined strain in a stress-strain relation Kent-Park type (for ductility check with moment-curvature diagrams)

ec 0: compressive strain e_{c0} at max confined stress in a stress-strain relation Kent-Park type (for ductility check with moment-curvature diagrams)

ec ult: compressive strain e_{cu} in correspondence of fc ult in a stress-strain relation Kent-Park type (for ductility check with moment-curvature diagrams)

STEEL

Name: Name of steel grade (string of no more than 10 alphanumeric characters)

Es: Elastic modulus E_S. ($E_S = 200000$ is the recommended value in EC2)

fym: Mean yielding stress f_{ym} (no indication in EC2). Used in ductility check

fyk: Characteristic (nominal) yielding stress f_{yk}

fyd: Design yielding stress $f_{yd} = f_{yk} / \gamma_s$ with $\gamma_s = 1.16$; this value is not editable as derived automatically from the value assigned to fyk

ftk: Characteristic nominal tensile strength f_{tk}. <u>This value is never used in program calculation</u> (but only for description). The ratio f_{tk}/f_{ty} is regulated for each class (A-B-C) of steel in Annex C EC2.
ftk/fyk: This ratio f_{tk}/f_{ty} , important for steel ductility, is regulated for each class of steel in Annex C EC2. <u>This value is never used in program calculation</u> (but only for description)
e_uk: Characteristic strain at max force e_{uk} (for A class steel ≥ 0.025; for B class steel ≥ 0.025 ; for C class steel ≥ 0.075). It is important to edit this value in accordance with the steel grade
e_ud: Design ultimate strain $e_{ud} = e_{uk} * k_u$ with k_u = reduction factor assigned in Code and reinforcement options (0.9 if the recommended for k_u value in EC2). This value is not editable as derived automatically from the values assigned to e_uk and k_u

k: Design ratio f_{td}/f_{yd} ; if you set <u>k=1</u> program assume in the design stress-strain diagram an horizontal top branch without strain limit (with a very little difference in results program assumes e_{ud} as strain limit); if you set <u>1<k≤1.35</u> program assume in design stress-strain diagram (see fig. 3.8 EC2) an inclined branch with strain limit e_{ud} and max stress $k \cdot f_{yd}$. Assigned k value must be consistent with nominal ratio f_{tk}/f_{ty}

2.12 Code and reinforcement options

This is the button in the Tab bar for quick opening of the following important dialog window (you can open this window also from from drop-down menu Options).

At starting of the program or of a new section this window show the default values which may be amended or extended. To save the options as default for the next calculations save it with the command "Save code and options as default". To save the current options (after modifications) only for current section save it with the command "Exit" (current options are saved together with the data of the section.

Code setting and reinforcement options

Code setting

National Annex values: EC2 code

| 2 EC2 | 3 EC2 | 6 EC2 | 7 EC2 | 9 EC2 | 5 EC8 |

§ in Eurocode	EC Values		My value
3.1.6(1) A cc	1.00	☐	0.85
3.1.6(2) A ct	1.00	☑	
3.2.7(2) Eud/Euk	0.90	☑	

Options for calculations

Diameter long. reinf.

1st Diam. mm `14`
2nd Diam. mm `16`

Diameter stirrups

1st Diam. mm `6`
2nd Diam. mm `8`

Diameter edge bars for torsion

Diameter `10` mm

Diameter cross-tie for columns/walls

Diameter `6` mm

A's/As min. (predefined sections) `.5`

Min. net distance for long. reinf. [§8.2(2)] `2` cm

Max. dist. between long. bars(design only) `25` cm

Default cover (from center of bars) `4` cm

Minimum long. distance between stirrups `5` cm

Ctg of angle for stuts in torsion (1<=Ctg<=2.5) `2`

Max personal Ctg angle of stuts in shear `2.5` <2.5

Concrete sections discretization (cm)

Default Step for discretization `0.500` <= 2 cm

⦿ Ribbed bars surface   ○ Smooth bars

[Info]   [Save Code and Options as default]

[Exit (Code and Options for current section)]

Code setting

This frame concerns only those parameters (used for r.c. section) that EC2/EC8 left open for national choice in National Annex.

The first drop-down combo box option is the standard EC2 with its recommended parameters. Subsequent options relate to some national annex here assumed.

Code dependent parameters are listed in order according to articles (§) in code. Placing the mouse on a line with code will display some information about the parameters.

In each option selected you may change the single value clearing the pertinent check box: so you can implement your country (or your particular) values also selecting EC2 as annex option.

In the above Tabs you can choose the parameter of interest selecting the number of the EC2/EC8 chapter where the parameter is present.

Reinforcement options

Reinforcement diameter in this window are used by the program to design calculation of the reinforcement. Only the first diameter is used for initial default values in input windows.

- Af'/Af: min ratio between compressive longitudinal bar area and the tension one. In seismic zone EC8 fixed precise values for thi ratio depending on the position of the section in the member
- Min net distance for long.reinf.: Min distance between edge bars. For the correct assignment of this value see §8.2 EC2
- Default cover: distance between centroid of the bar neatest to the concrete edge and the same concrete edge. For the correct assignment of this value see §4 EC2 and the sub-program Concrete cover 75
- Minimum long. distance between stirrups: minimum pitch in stirrups design for the first diameter in above options; if for such diameter the pitch is not sufficient program assume the second diameter
- Cot of angle of struts in torsion: this cot value is assumed as default only for the typology Polygonal section in Torsion. Limiting value are those assigned in above Code setting
- Max personal Cot angle of struts in shear: Program assume this value as max limit value for cot value for a much conservative amount of stirrups than EC2 cot recommended value 2.50

- Barre ad aderenza migliorata o barre lisce: questa opzione viene utilizzata solo nel calcolo dell'apertura delle fessure e nel calcolo delle deformazioni in esercizio.
- Default step of discretization: for very large sections (i.e. bridge piers) we recommend to set this value ≥ 1 cm for a much quick calculation.

2.13   Concrete rheological coefficients

This is the button in the Tab bar for quick opening of the following dialog window (you can open this window also from from drop-down menu Options).

This is a service sub-program (to select in the principal Tab bar) 7 that calculate the rheological coefficients (creep and shrinkage) with formulas mentioned in § 3.1.4 EC2. These coefficients can be entered in the Materials Library 67 in order to be used by program if AAEM (Age Adjusted Effective Modulus) or EM (Effective Modulus) are selected in General data 9 input for SLS verifications. Creep and shrinkage are, however, always used by program in the SLS deflection 56 calculating of beams in SLS.

Aging coefficient from t_0 to $t_1 = \infty$ can be assumed, with good approximation, always equal to 0.8 (with $t_0 > 15$ days). EM method wants ageing coefficient set to 1. If time t_1 is finite (t, t_0) is calculated by the following semi-empirical relaxation function (Bazant and Kim, 1979):

$$\chi(t, t_0) = [1 - R(t, t_0)/E_c(t_0)]^{-1} - 1/\varphi(t, t_0)$$

$$R(t, t_0) = (1 - 0.008) / \Phi(t, t_0) - 0.115 [\Phi(t - t, t_0)/\Phi(t, t_0 + \Delta) - 1] / \Phi(t, t - 1)$$

in cui:
$$\Delta = (t - t_0)/2$$
$$\Phi(t, t_0) = [1 + \varphi(t, t_0)]/E_c(t_0)$$

2.14 Concrete Cover

The concrete cover is the distance between the surface of the reinforcement closest the nearest concrete surface (including link and stirrups and surface reinforcement where relevant) and the nearest concrete surface. The nominal cover c_{nom} shall be specified on the design drawings. It is defined a minimum cover, c_{min} (see §4.4.1.2), plus an allowance in design for tolerance, c_{dev} (see §4.4.1.3):

$$c_{nom} = c_{min} + c_{dev}$$

The evaluation of the nominal cover is preliminary to the checks of any section once established concrete data, the environmental conditions and the diameter of longitudinal bars and stirrups.

The cover to assign to longitudinal bars in the input windows of this program is the above nominal cover value plus half diameter of bars.

The calculation of the concrete cover to be performed two times: first with reference to the stirrups, the second to that of the longitudinal bars. The concrete cover more restrictive of the two affects the value of the other concrete cover.

In the Example 4.1 in ref./4/ are given:

- 20   as diameter of longitudinal bars
- 8   as diameter of stirrups
- XC1   as exposure class
- $d_s = 20$ mm   as max aggregate size (<32 mm)
- 50 years   as design working life of structure
- S4   as Structural class
- 10 mm   as c_{dev}
- Normal quality control is put in place.

For stirrups the calculation leads to: $c_{min} = 15$ mm   then $c_{nom} = c_{min} + c_{dev} = 15 + 10 = 25$ mm

For long. bars the calculation leads to: c_{min} = 20 mm   then $c_{nom} = c_{min}$ + c_{dev} = 20+10 = 35 mm

The concrete cover for the stirrups is "dominant". In this case, the concrete cover for longitudinal bars is increased to: 25+8 = 33 mm

The same result is shown in the below window that you can open from "Run" drop-down menu or directly from the pertinent command (the last one) in the principal Tab bar of the program. Note that the last result is the cover value to assign in the input windows of the present program. Results can also be printed on paper.

Concrete Cover acc. to §4.4.1 EC2-1

Parameters for Definition of Concrete Cover

Diameter of reinforcement (for bond requirementl)

Max diameter (or equivalent diam.) of longitudinal bars	20	mm
Max Diameter od Stirrups (set 0 in absence of them)	8	mm

Concrete data and Environmental conditions

Exposure Class acc. to §4.4.1.2(5)	XC1	
Concrete Class acc. to Appendix E EC2-1	C25/30	
Design Working Life acc. to § 4.4.1.2(5)	50 years (S4)	

☐ Nominal maximum Aggregate Size Greater than 32mm acc.to table 4.2

☐ Special Quality Control of Concrete Production acc. to table 4.3N

Variations

☐ Reduction Cmin,dur for slab members §4.4.1.2 table 4.3N

☐ Increase for abrasion §4.4.1.2 (13)   [0] mm

☐ Reduction for Add. Prot. §4.4.1.2(7)(8)   dCdur,st+dCdur,add   [0] mm

☐ Additive Safety Element acc. to §4.4.1.2(6)   dCdur,g   [0] mm

  Additive Allowance for Deviation acc. to §4.4.1.3(1) dCdev   [10] mm

Cover Results

Min edge Cover for stirrups [eq. (4.2)]	Cmin	15	mm
Min edge Cover for longitudinal bars [eq. (4.2)]	Cmin	23	mm
Nominal edge Cover for stirrups [eq. (4.1)]	Cnom	**25**	mm
Nominal edge Cover for longitudinal bars [eq. (4.1)]	Cnom	**33**	mm
Nominal Cover to Longitudinal Bars Centroidal Axis		**43**	mm

[Print] [Exit]

The correspondences between the symbols used in this sub-program and those in §4.4.1 EC2 are:

Cmin,dur = $c_{min,dur}$ minimum cover due to environmental conditions

dCdur,st = $c_{dur,st}$ reduction of minimum cover for use of stainless steel

dCdur,add= $c_{dur,add}$ reduction of minimum cover for use of additional protection

dCdur,g = $c_{dur,}$ additive safety element

2.15   Text height - Printing options

From drop-down Options menu you can select this dialog window to set the following entities:

- text height [cm] of text within the graphs (text height is instead automatic for the interaction domains and moment-curvature diagrams);
- drawing scale of the stress diagram in the screen graphs;
- type of printing (compact or extended) in the results report (i.e. printing or not stress in all vertices of section and in all bars).

2.16   Colors in the graphs
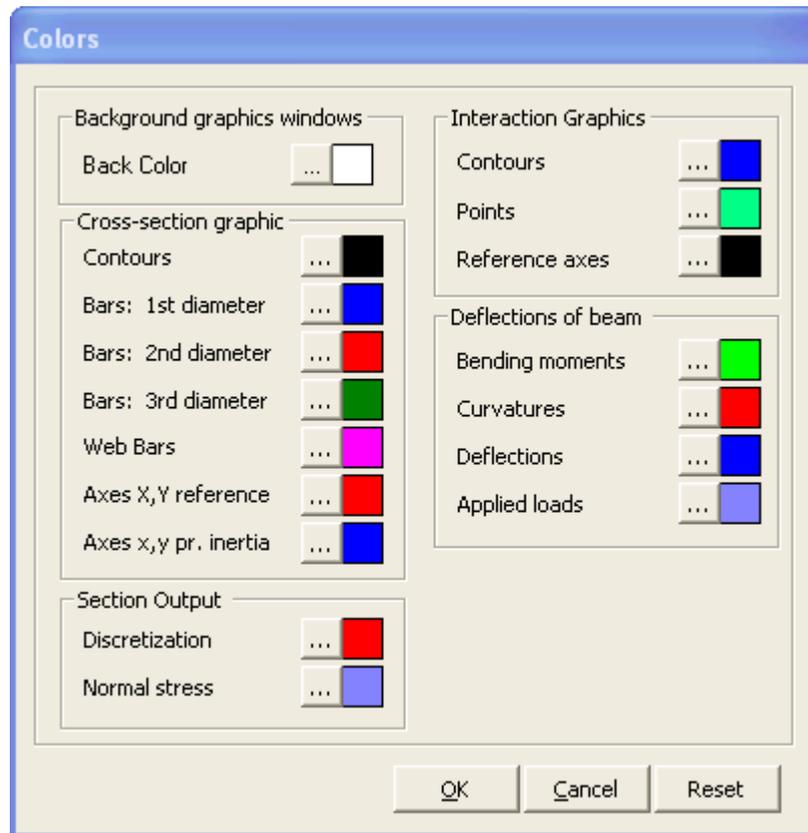
From drop-down Options menu you can select this colors dialog window
to set the principal graphical entities of the program.

3 Theoretical Background

3.1 ULS Bending and Axial Force

3.1.1 Assumptions

The calculation of the ultimate moment resistance M_{Rd} of rc cross-section is based on the following assumptions (§6.1 EC2):

- plane sections remain plane
- the strain in bonded reinforcement is the same as that in the surrounding concrete
- tensile stresses of concrete are ignored (no strength in tension)
- the stresses in the concrete in compression are derived from the design stress-strain relationship illustrated in the following
- the stresses in the reinforcing steel are derived from the design curves showed in the following

Stress-strain relation for the design of concrete

For the design of cross-sections program uses the parabola-rectangle stress-strain relationship according to §3.1.7 EC2 depicted in the follow figure.
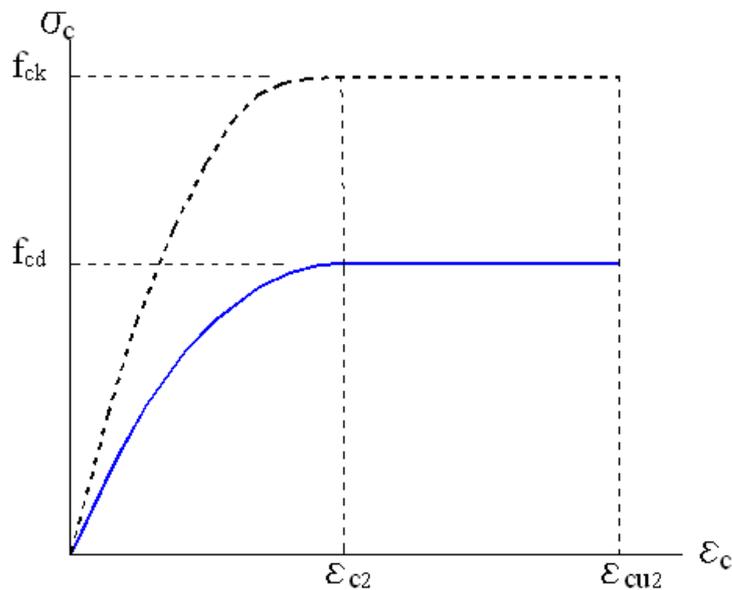
The value of the design compressive strength f_{cd} is:

$$f_{cd} = \alpha_{cc}\, f_{ck} / \gamma_c \qquad\qquad (3.15)\text{EC2}$$

where:

$\gamma_c = 1.5$ is the partial safety factor for concrete

α_{cc} is a coefficient taking account of long term effect… (recommended value = 1.0 but editable in Code and reinforcement options [70])

The parabola-rectangle in figure is described by the following function:

$$\sigma_c = f_{cd} \left[1 - \left(1 - \varepsilon_c / \varepsilon_{c2} \right)^n \right] \quad \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \qquad (3.17)EC2$$

$$\sigma_c = f_{cd} \qquad\qquad \text{for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \qquad (3.17)EC2$$

ε_{c2} = is the strain at reaching the maximum strength = 0.0020 for $f_{ck} \leq 50$ Mpa

= 0.0020 + 0.000085$(f_{ck}-50)^{0.53}$ for $f_{ck} \geq 50$ Mpa

ε_{cu2} = is the ultimate strength = 0.0035 for $f_{ck} \leq 50$ Mpa

= 0.0026 + 0.00035$[(90-f_{ck})/100]^{0.53}$ for $f_{ck} \geq 50$ Mpa

n = 2.0 for $f_{ck} \leq 50$ Mpa

= 1.4+23.4$[(90-f_{ck})/100]^4$ for $f_{ck} \geq 50$ Mpa

Stress-strain relation for the design of steel

For the design of cross-sections program uses one the bilinear stress-strain relationships according to §3.2.7 EC2 depicted in the follow figure.
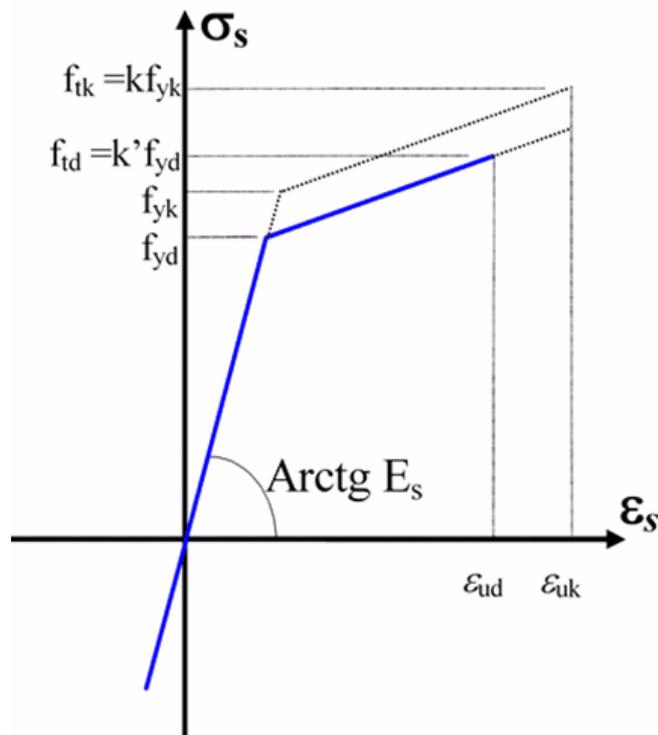
The inclined top branch has a strain design limit $\varepsilon_{ud} = k_u \cdot \varepsilon_{uk}$ a   yield stress $f_{yd} = f_{yk}/_s$ and a maximum stress $f_{yd\,max} = k\,f_{yk}/_s$

where:

ε_{uk} = ultimate characteristic strain (ductility)

k_u = reduction factor for ε_{uk} (= 0.9 recommended in EC2; but you can edit a different value according to National Annex in Code and Reinforcement options)

k = f_{yk}/f_{yk} this ratio can vary from 0 to 1.35

In §3.2.7(2) EC2 is allowed to set k=1.0 (horizontal top branch without strain limit) independently of the actual value of f_{yk}/f_{yk} .

The actual min or/and max value of f_{yk}/f_{yk} vary according to the class of steel (i.e. for class C the ratio $1.15 \le f_{yk}/f_{yk} \le 1.35$) (see Annex C EC2)

In the program you can set (in the Materials Library [67]) any k ≥ 1.0 for design checks independently of the actual value of f_{yk}/f_{yk} (but pay attention to respect the min and max actual value of f_{yk}/f_{yk})

In the program you can set freely (in the Materials Library [67]) the value ε_{uk} but you must pay attention to respect the actual ultimate strain (see Annex C EC2)

3.1.2   Uniaxial bending

Given a cross-section referred to O,x,y axes, the bending is defined uniaxial when the bending is around x axis and the corresponding neutral axis is parallel to x.

This condition, strictly speaking, is true only if the section is symmetrical with respect to y axis. But in many circumstances (beam within rigid floor diaphragms, foundation beams) the   lateral constrains make sufficiently approximate the above hypothesis.

To evaluate internal forces Mx, N for each position of the neutral axis program works with the following procedure of integration of stresses.
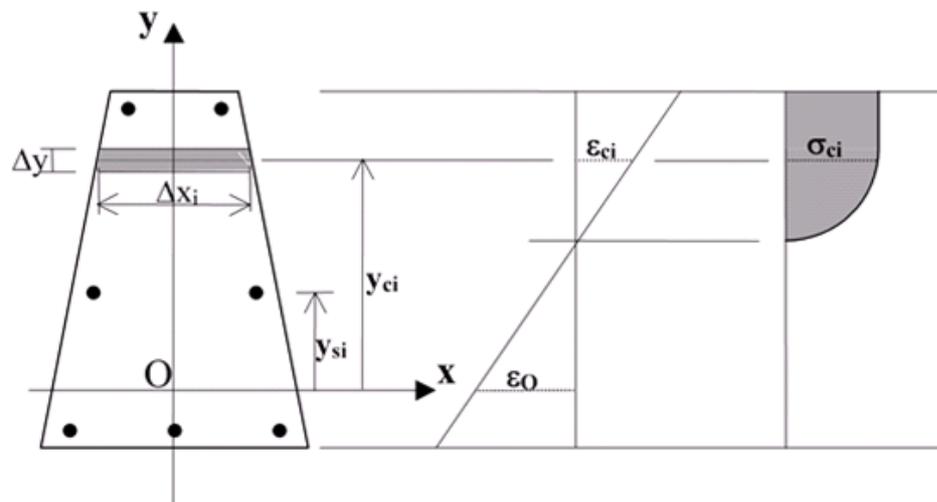
Discretization of cross-section and generic stain and stress distribution

The cross section is first discretizzed into n strips parallel to x axis (and then to neutral axis) having all the same thickness   y (step) that you can change (from 0.2 to 2.0 cm) in the dialog window Code and reinforcement options 70 . The width of   x$_i$ each strip is given (see figure above) by the intersection of the line passing in the middle of the step   y. The area of each strip is then given by:

$$Ac_i =   x_i .   y$$

The internal forces N , Mx for any position of the straight line of deformation of the cross-section are given by the following equilibrium equations referred to x axis:

$$N = \int_{A_c} \sigma_c \cdot dA_c + \Sigma A_{si} \, \sigma_{si} \cong \Sigma_n A_{ci} \, \sigma_{ci} + \Sigma A_{si} \, \sigma_{si}$$

$$Mx = \int_{A_c} \sigma_c \cdot y_c \cdot dA_c + \Sigma A_{si} \, \sigma_{si} \, y_{fi} \cong \Sigma_n A_{ci} \, \sigma_{ci} \, y_{ci} + \Sigma A_{si} \, \sigma_{si} y_{fi}$$

where:

σ_{ci} is the stress, assumed constant in the entire strip, in the A_{c_i} area

ε_{ci} is the strain, assumed constant, in the A_{c_i} area

σ_{si} is the stress in the centroid of area A_{si} of the generic bar i

ε_{si} is the strain in the centroid of area A_{si} of the generic bar i

In above summations the normal force N is supposed to be applied in the origin O; to compare the internal moment Mx with the external one (design moment) with N placed in the centroid of concrete section, it is necessary (after summation) to add the moment $- N \, y_c$ to the above Mx (Y_c = y-coordinate of centroid).

Every possible strain distribution in ultimate limit state is shown in the figure below.
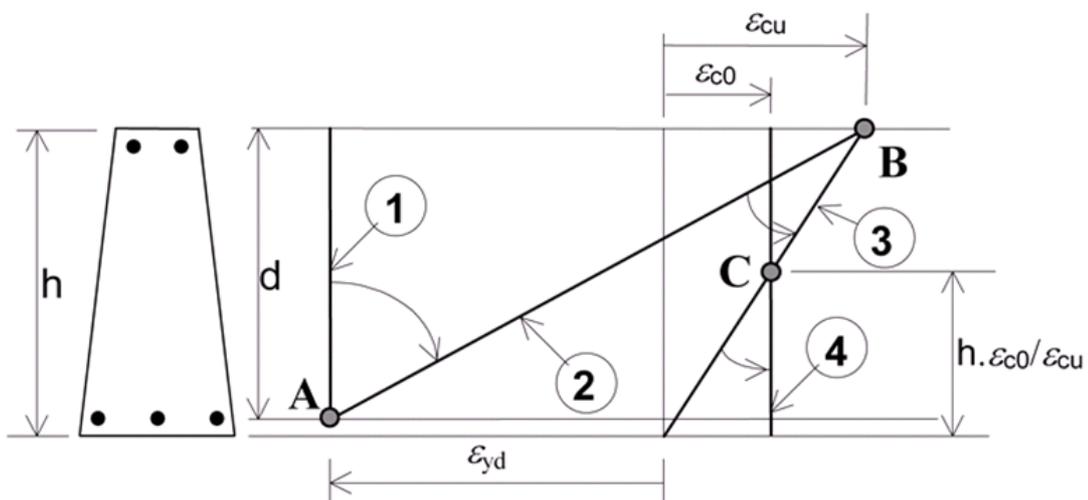
All possible strain distribution at ultimate limit state (with strength areas between 1-2-3-4 characteristic points)

For the assumption of conservation of planar sections and the described limitations imposed to the ultimate strain of concrete and steel, the deformed configurations of the section, corresponding to the various possible ultimate states, all pass at least one of the points labeled A, B, C of the figure. In the same figure are also represented the positions 1-2-3-4 (for positive curvatures) of the strain configurations corresponding

to transition from one type of failure to another.

The strength area from 1 to 2 is obtained by rotating the section around the point A (pivot A). The second area from 2 to 3, by rotating the section around B. The third field, finally, from 3 to 4 around the pivot C.

SAFETY CHECK
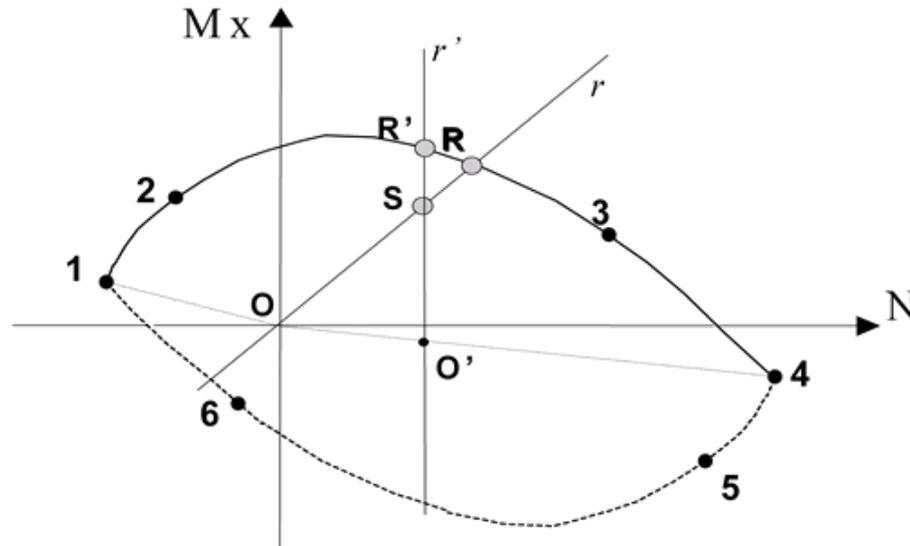
Strength domain (interaction N-Mx domain) for ultimate limit state in uniaxial bending

The above figure is a generic interaction region of the strength internal forces N, Mx (the moment is referred to the centroid of concrete).

The boundary curve of region is the border of strength domain and each of its points corresponds to one of strain distribution (with ε_c or ε_s at their strength value) shown in previous paragraph.

In that border 1-2-3-4 points are the same points seen in the previous topic with only reference to the positive (2-3) or null (1-4)curvatures. Points 5-6 refers to negative curvatures.

To calculate the forces N, Mx corresponding to those characteristic points (from 1 to 6) just executes the integration described in the topic uniaxial bending[83] assuming the following analytical expression of the straight line of deformation:

$$\varepsilon(y) = b\ y\ + c \qquad (1)$$

where :

$$b\ = c_x \qquad = \qquad \text{curvature around x axis}$$
$$c\ = e_o \qquad = \qquad \text{strain in the origin O of reference axes}$$

x,y

coefficients b, c of line (1), are derivate by the strains ε_P , ε_Q of any two points P,Q of the section with y-coordinates y_P, y_Q :

$$b = (\varepsilon_P - \varepsilon_Q) / (y_P - y_Q)$$
$$c = \varepsilon_Q - b\, y_Q$$

Known b, c it is easy to calculate with the eq. (1) any point {N,Mx} of the safety border. For example if you want an intermediate point between characteristic points 2 and 3 of which are known the curvatures b2, b3 referred to pivot B (in previous topic):
assign a generic curvature b between b2 and b3, indicate with y_B, ε_B the y-coordinate and the known strain of pivot B, you can get the unknown value of coeff. c in eq. (1):

$$c = \varepsilon_B - b\, y_B \qquad\qquad [\,b2 < b < b3\,]$$

The integration of stress provides, finally, the pair of N, Mx values of point with curvature b.

$$c = \varepsilon_B - b\, y_B \qquad\qquad [\,b2 < b < b3\,]$$

The same procedure you can do for every b curvature for the other characteristic couple of points so to built the entire border of the strength domain.

Given a generic couple of design forces {N_S,Mx_S} , uniaxial bending check is positive if its representative point S (see previous figure) is not external to strength domain (as in depicted case). This condition that we define measure of safety can be numerically expressed in multiple ways. This program allows the following two way (see previous figure):

1) At constant eccentricity e=Mx/N: program calculate {N_R,Mx_R} of point R as intersection of border line with the stress path line r (with Mx/N = e for all of them). The measure of safety is given by the segment ratio OR/OS (**Safety Factor**). If such ratio is ≥ 1.0 the bending conformity is positive for {N_S,Mx_S} design forces.

2) At constant normal force N=const.:   program calculate {$N_{R'}$,$Mx_{R'}$} of point R' as intersection of border line with the stress path line r'(with N = const for all of them). The measure of safety is given by the segment ratio O'R'/O'S (**Safety Factor**). If such ratio is ≥ 1.0 the bending conformity is positive (check is OK) for {N_S,Mx_S} design acting forces.

DIRECT SAFETY CHECK

Program perform the uniaxial bending check (to obtain the safety factor) in the following way.

Generic equation of straight line r (or r') in the previous figure has the form:

$$l \cdot N + m \cdot Mx + p = 0 \qquad (2)$$

where l, m coefficients are immediately given from condition that r passes through the design point $\{N_s, Mx_s\}$ and the origin point O (or O')

In the case depicted in the previous figure the intersection point $\{N_R, Mx_R\}$ R is placed between characteristic point 2-3 then in the second area strength then we know the pivot B:
the only unknown is then the curvature b that must be: b2 ≤ b ≤ b3.
So b can be easily calculated with iterative procedure of bisection.

3.1.2.1 Design of reinforcement

This type of caclulation is a option available in section data[12] dialog window.
Given the geometrical and mechanical data of predefined concrete section and the steel grade and position of the two row of bars, program search the minimum areas of row bars which correspond to a strength domain in which all acting forces $\{N_s, Mx_s\}$ fall (S on the border or within the border if code detailing requires more steel area).
For beams bars their design procedure is based on the respect on a limit value for x/d (max value set = 0.45) and on the minimum ratio A_s'/A_s between compressive and tensile area of bars (assegned in the window Code and reinforcement options[70]).
For columns bars design provedure is based only on the A_s'/A_s assumed in options.

3.1.2.2 Detailing cecks

Strength check based on bending resistance is not sufficient to state the positive conformity of a cross-section.
It is also necessary to check the conformity to the detailing of reinforcement (§8 EC2; §5 EC8) and the detailing with references to member (§9 EC2; §5 EC8) to which section belongs.
Program performs also these checks using the Eurocodes rules and the options assigned in Code and reinforcement options[70] window.

3.1.3 Biaxial bending and axial force

Biaxial bending with axial force refers to general section[26] and rectangular section of columns[38].

A general section (as that depicted in the below figure) consistes in one or more concrete polygonal regions and any number of bar anyway arranged. Concrete geometrical data and bars have a general reference system O,X,Y. Positive bending moment are showed in the figure. N assigned normal force (positive if compressive) is applied in the centroid of concrete regions. Bending design moment can be refered to O,X,Y axes or C,x,y principal inertia axes.

In calculations performed by the program internal normal force N is applied in the origin O and MX,MY to X,Y axes.
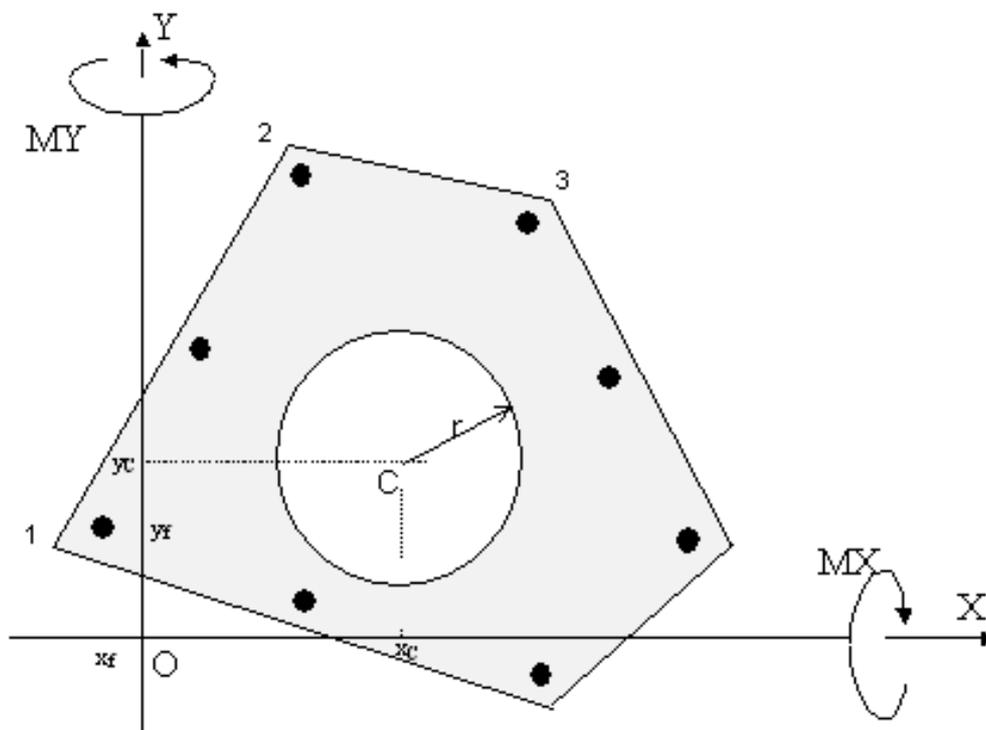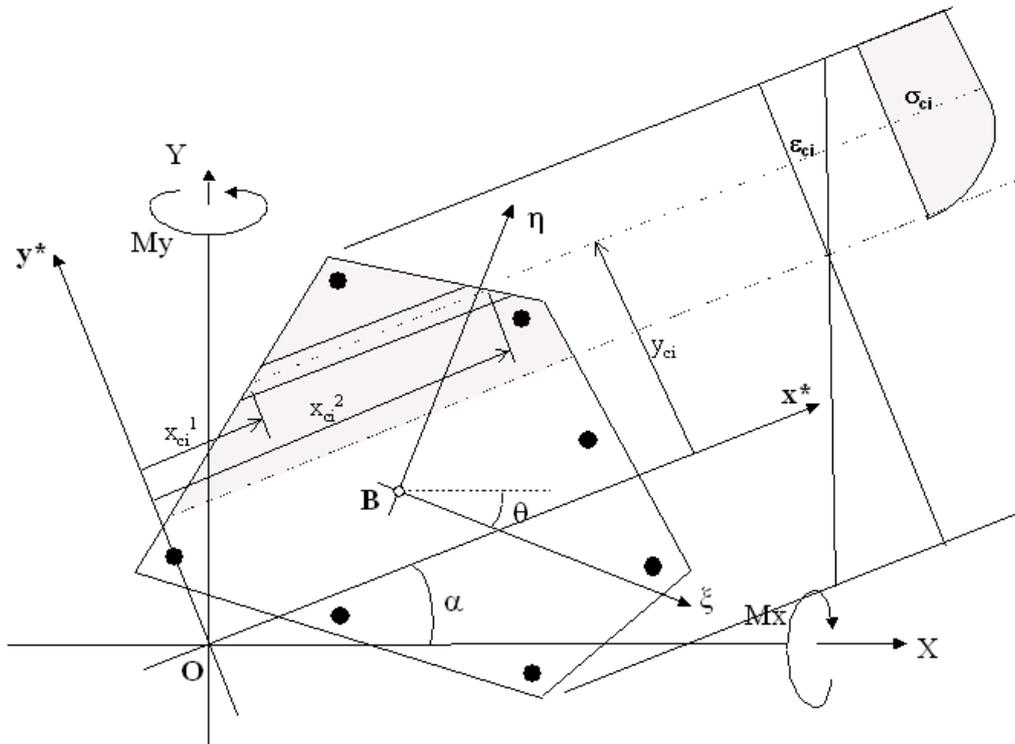
Scheme of integration of stesses in biaxial bending

In the above figure is showed the scheme of integration of stresses referred to a generic position of neutral axis with the same inclination α of x* axes which belongs to a rotated reference system O,x*,y*. The numerical integration is performed with:

$$N \quad = \quad \int_{A_c} \sigma_c \cdot dA_c + \Sigma A_{si} \cdot \sigma_{si} \qquad \cong \Sigma_n A_{ci} \, \sigma_{ci} + \Sigma A_{si} \, \sigma_{si}$$

$$Mx^* \quad = \quad \int_{A_c} \sigma_c \cdot y_c \cdot dA_c + \Sigma A_{si} \cdot \sigma_{si} \cdot y_{fi} \qquad \cong \Sigma_n A_{ci} \, \sigma_{ci} \, y_{ci} + \Sigma A_{si} \, \sigma_{si} \, y_{fi}$$

$$My^* \quad = \quad \int_{A_c} \sigma_c \cdot (-x_c) \cdot dA_c - \Sigma A_{si} \cdot \sigma_{si} \cdot x_{fi} \qquad \cong -\Sigma_n A_{ci} \, \sigma_{ci} \, x_{ci} - \Sigma A_{si} \, \sigma_{si} \, x_{fi}$$

where, with reference to the generic discrete strip i of the figure:

$$A_{ci} = (x_{ci}^2 - x_{ci}^1) \cdot \Delta y$$

$$x_{ci} = (x_{ci}^2 + x_{ci}^1) / 2$$

The integration results Mx*, My* are then referred to B, ξ, ,η principal inertia system (to compare the internal forced with the design ones):

$$M\xi = Mx^* \, cos \, \theta + My^* \, sin \, \theta + N \; y_R$$
$$M\eta = -Mx^* \, sin \, \theta + My^* \, cos \, \theta - N \; x_R$$

where x_R, y_R are the coordinates of origin O referred to principal axes of inertia.
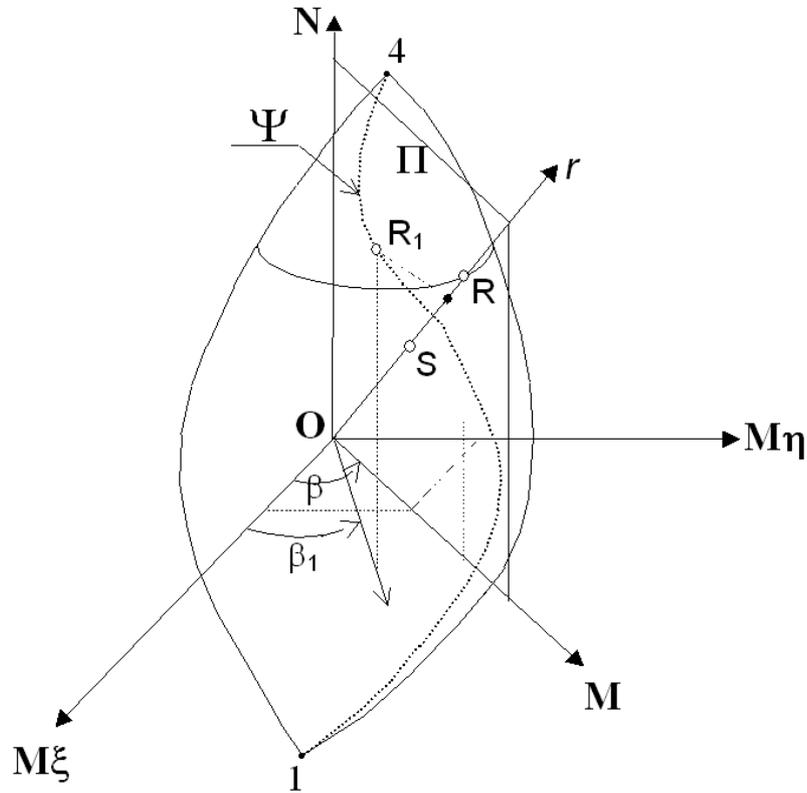
Three-dimensional interaction domain N-M -M

As you can see in the above figure, the generic design forces $S\{N_{Sd}, Mn_{Sd}, M\xi_{Sd}\}$ has three components to compare, in a definite measure direction r, with the correspondent strength forces $R\{N_{Rd}, Mn_{Rd}, M\xi_{Rd}\}$ obtained by the intersection of the straight line r with the surface of interaction (border of 3-d domain). To each point of this surface correspond a strain configuration of the section with a failure of concrete or steel. In particular the cusp points 1 correspond to the max tension strain (ε_{ud}) at null curvature and 4 to the min compressive strain (ε_{cd}) at null curvature.

For each direction α of the neutral axis we can obtain the same possible strain distribution at ultimate limit state seen in uniaxial bending with characteristic intermediate point 2-3. The ultimate border line 1-2-3-4 seen in uniaxial bending here become (performing the previous numerical integrations) a three-dimansional line ψ (from 1-4 in above figure). Varying the angle α (i.e. neutral axis direction) from 0 to 2 π the set of all the curves ψ constitute the entire interaction surface.

3.1.3.1   Safety check with constant M/N
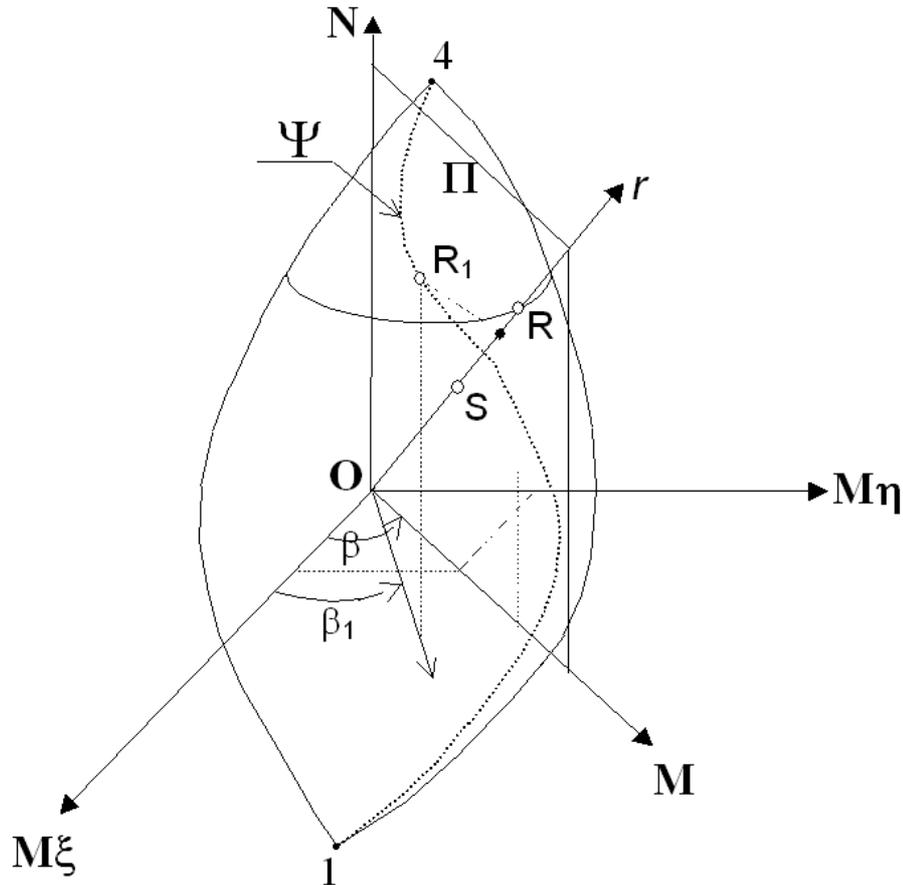
The measure of safety with constant M/N is performed in perfect analogy to that developed for uniaxial bending[83]. Point S is a representative point of the external design forces $\{N_{Sd}, M\xi_{Sd}, Mn_{Sd}\}$, the measure of safety is given (see figure) from the ratio between the length of segments OR and **OS** measured along the straight line r belonging to plane Π passing through the axis N and forming a constant angle   with axis M_ξ:

$$\beta = arctan\ (Mn_{Sd}\ /\ M\xi_{Sd}\ )$$

The check consists to find the particular direction α of neutral axis whose corresponding spatial curve of interaction Ψ intersects the plane of interaction Π just in the point **R** (intersection of line r and the interaction surface. As the point **R** belongs to Π plane , its components Mn_{Rd}, $M\xi_{Rd}$ form   , the, the same β angle.

3.1.3.2 Safety check with constant N

In this case, the straight line r of measure lies in the plane orthogonal to N axis and all where all points are then characterized by $N = N_{Ed}$ (design axial force). To be sure that the origin **O'** of the measure line r is internal to the interaction surface, **O'** is assumed as the intersection of the straight line O-4 with the plane $N = N_{Ed}$. So the safety measure is the segment ratio **O'R** / **O'S.**
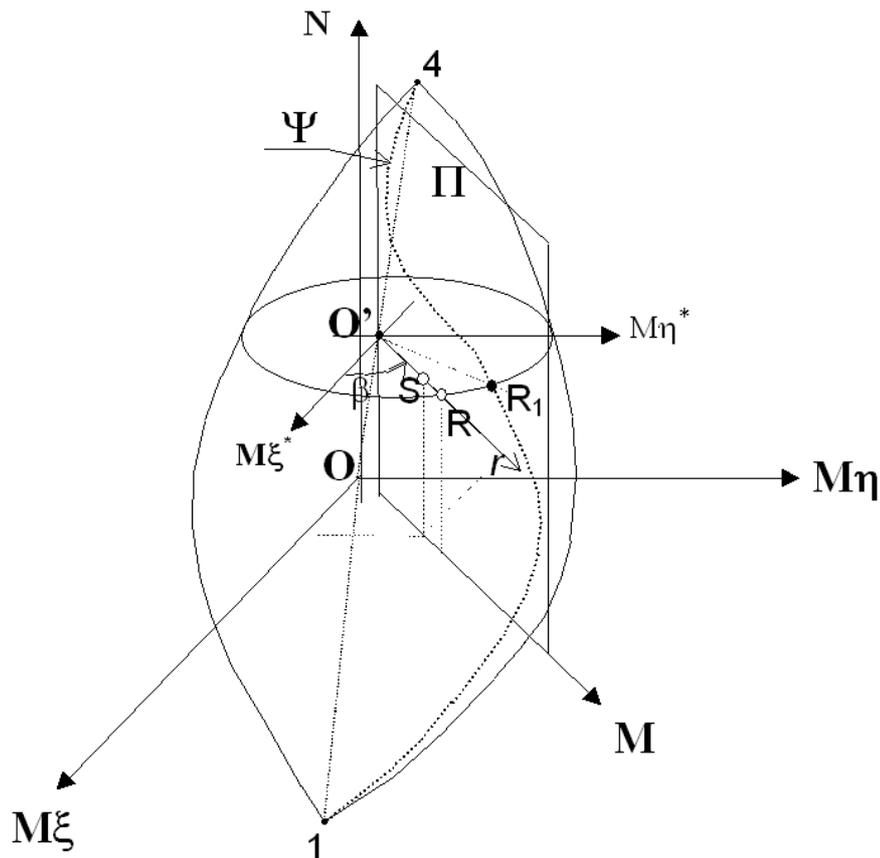
3.2 ULS Shear

3.2.1 Uniaxial check (predefined section)

The design for shear force resistance is only performed in the ultimate limit state. The analyzed rc sections belong to beam or to columns with transverse reinforcement that consist only of stirrups placed orthogonally to the axis of member. Do not use this shear check with opened thin walled sections (section do not stay plane with bending-shear actions).

The general design requirement according to EC2 (EN 1992-1-1) is the following:

$$V_{Ed} \quad \leq \quad V_{Rd}$$

where

   V_{Ed} design value of applied shear force

   V_{Rd} design value of shear force resistance

Depending on the failure mechanism, the design value of the shear force resistance is determined by one of the following three values:

   $V_{Rd,c}$ design shear resistance without shear reinforcement

   $V_{Rd,s}$ design shear resistance limited by yiel shear of shear reinforcement

   $V_{Rd,max}$ design shear resistance limited by strength of concrete compression struts

 Design shear resistance without shear reinforcement

The design value for the design shear resistance $V_{Rd,c}$ may be determined by:

$$V_{Rd,c} = [C_{Rd,c} \ k \ (100 \ _l \ f_{ck})^{1/3} - k1 \ \sigma_{cp} \ ] \ b_w \ d \qquad (6.2a) \ EC2 \quad \sigma$$

For the symbol's meaning please see EC2 /1⌐173⌐/.
Program provide in the general data⌐9⌐ window   a special member characteristic defined "Beam without stirrups" that you can choose if no stirrup is is to use.

Design shear resistance with shear reinforcement

Program provide only shear reinforcement perpendicular to the member's axis ($\alpha = 90°$), then the following is applied:

$$V_{Rd,s} = A_{sw}/s \ \cdot z \cdot f_{ywd} \ \cdot cot\vartheta \qquad (6.8)EC2$$

where:

   A_{sw} cross-sectional area of shear reinforcement

   s   longitudinal spacing between stirrups (pitch)

   z   level arm of internal force of bending and axial force acting in the same combination

           (program do not assume the simplified value $z = 0.9 \cdot d$)

   f_{ywd} yield strength of shear reinforcement

   ϑ inclination of concrete compression struts

The inclination ϑ of struts must comply with the following limit values recommended by EC2:

$$1 \leq \cot \vartheta \leq 2.5 \qquad\qquad\qquad (6.7)EC2$$

Those limiting value can be different in the National Annex (see Code and reinforcement options[70])
The design shear resistance of concrete compression struts ($\alpha = 90°$ always in this) is:

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (1 + \cot^2 \vartheta)$$

where

α_{cw} = 1   factor for considering stress conditions in compression chord

b_w minimum width of cross section (predefined sections)

z lever arm of internal forces

v_1 reduction factor for concrete strength (National Annex can change this parameter. See Code and reinforcement options)[70]

f_{cd} design value of concrete strength

ϑ inclination of concrete struts

The procedure followed by the program for shear check is:

If for $\cot \vartheta = 1$ $V_{Rcd} < V_{Ed}$ then concrete section dimensions are not sufficient for shear resistance then it is necessary to resize the section.
If for $\cot \vartheta = 1$ $V_{Rcd} \geq V_{Ed}$ then equating design shear V_{Ed} to shear resistance (1) V_{Rcd} we obtain ϑ value:

$$\vartheta = 0.5 \cdot \arcsin [(2\,V_{Ed})/(a_{cw} \cdot v_1 \cdot f_{cd} \cdot 0.9 \cdot d \cdot b_w)]$$

if $\cot \vartheta$ satisfyes eq. (6.7)EC2 the strictly necessary stirrups area A_{sw} is given by eq. (6.8). The ckeck is OK if the comparison between A_{sw} is less than the actually present area.
If the section was defined (in general data[91]) as internal to a critical region of a beam in seismic zone and its ductility class is DCH then for EC8 program sets $\cot \vartheta = 1$.
If $\cot \vartheta > 2.5$ we assume $\cot \vartheta = 2.5$.
In Code and reinforcement options[70] windows you can optionally assign to $\cot \vartheta$ a max value less than 2.5.

In shear checks program controls all the stirrups detailing specified in § 8 and § 9 of EC2 and in § 5 of EC8.

3.2.2   Biaxial check

Eurocode 2 rules (§6.2) for the shear, applied in the previous paragraph [92] to the predefined sections, shall apply only to the uniaxial stress perpendicular to the neutral axis of a section of the beam subject to bending also uniaxial.

In current design applications, however, the checks of reinforced concrete columns must always be conducted in biaxial bending with axial load and in the presence of both components of the shear referring, in general, the principal axes of inertia of the section of concrete only. Capacity design of columns in seismic zones requires a considerable increase of the shear strength that can become a critical element in the design.

While for biaxial bending Eurocodes provide two types of approximate solutions in the case of biaxial shear is not given any indication, simplified or rigorous as it is, on the possible mode of calculation. It is therefore proposed the following procedure.
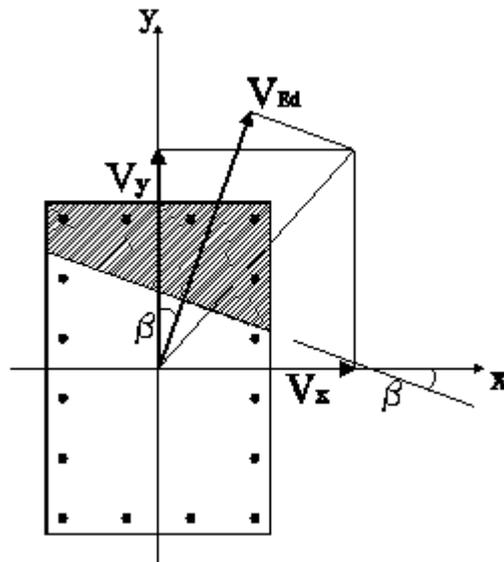
Assuming known the neutral axis at ultimate limit state, the resulting shear vector $V_x + V_y$ is not generally orthogonal to that line. Even for the simple rectangular section of the figure it is evident that the calculation of the shear resistance conducted separately for the two acting components V_x and V_y , in addition to lead to results to the detriment of safety is conceptually incorrect because it admits the decoupling of the components V_x, V_y does not actually present in the same way as that of the components M_x, M_y bending moment. It therefore seems more appropriate to assume, as plane truss resistant model to shear, the plane orthogonal to the direction of the neutral axis of the section obtained from the calculation of biaxial bending in ULS. Consequently the

value of the acting shear force will be constituted by the resultant V_{Ed} of the sum of the projections of the components V_x, V_y on the direction orthogonal to that of the neutral axis. Indicated with β the angle formed from the neutral axis with the x axis of inertia, the value of effective shear force to assign in the following calculation is then:

$$V_{Ed} = V_x \sin \beta + V_y \cos \beta$$

In this way biaxial shear problem has become an uniaxial shear calculation. Sections reported below are oriented so as to highlight the orthogonally between the neutral axis (in biaxial bending) and the direction of the acting shear V_{Ed} just defined.
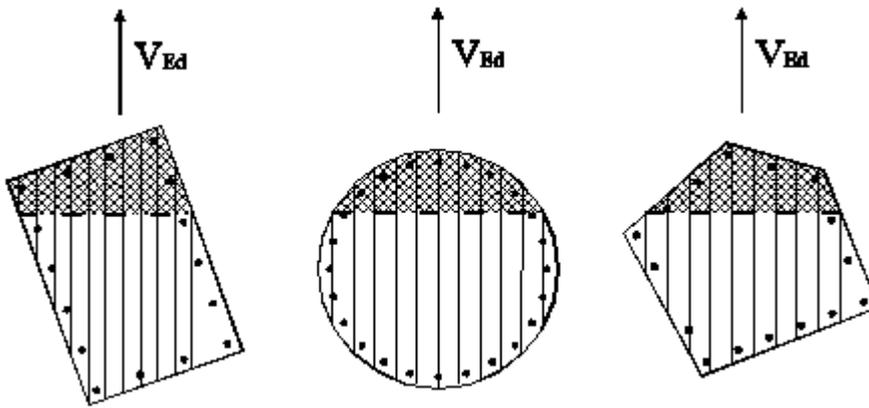
A rational approach to the problem of the evaluation of shear resistance is to divide the sections in a sufficient number of elementary strips (as shown above) plotted parallel to the direction of the shear V_{Ed} and consider the shear strength as the sum of the individual elementary strips. Each strip is considered in the same way as a rectangular section whose resistance can be evaluated based with the relationships (6.8)-(6.9) EC2 below reformulated with reference to the generic strip and assuming the stirrups always orthogonal to the beam axis ($\alpha = 90$ °):

$$V_{Rsd\,i} = z_i \cdot A_{sw\,i} /s \cdot f_{ywd} \cot\theta \qquad\qquad (1)$$

$$V_{Rcd\,i} = z_i \cdot b_i \cdot a_c \cdot f'_{cd} \sin\theta \cdot \cos\theta \qquad\qquad (2)$$

where:

z_i effective inner lever arm of strip i   (to each z_i corresponds a d_i value)
$A_{sw\,i}$ portion of the area of the stirrups engaged in the single strip i

s   pitch of stirrups

f_{ywd} yield strength of shear reinforcement

f'_{cd} reduced compressive strength of concrete ($v_1 \cdot f_{cd}$);

a_{cw} = 1   factor for considering stress conditions in compression chord

θ inclination of concrete struts (the same for all strips).

So that each strip could represent the projection (on the cross section) of an elementary isostatic truss is necessary that its two extremes fall respectively in the current compressed strut (compressed area of the section due to the bending) and in the tensile current formed by the longitudinal bar placed along the perimeter. To meet the first condition we exclude strips that do not intersect neutral axis. The second condition may be satisfied if the longitudinal bars are sufficiently distributed along the perimeter of the section (tubular reinforcement). For columns in seismic zones you can consider this condition always satisfied because of the high values of the minimum reinforcement ($\rho \geq 1\%$) which involve small distances between the bars (EC8 provides, also, a distance ≤ 15 cm on high ductility DCH and ≤ 20 cm on mean ductility DCM). In not seismic zones the designer can meet this requirement with an appropriate distribution of the longitudinal bars.
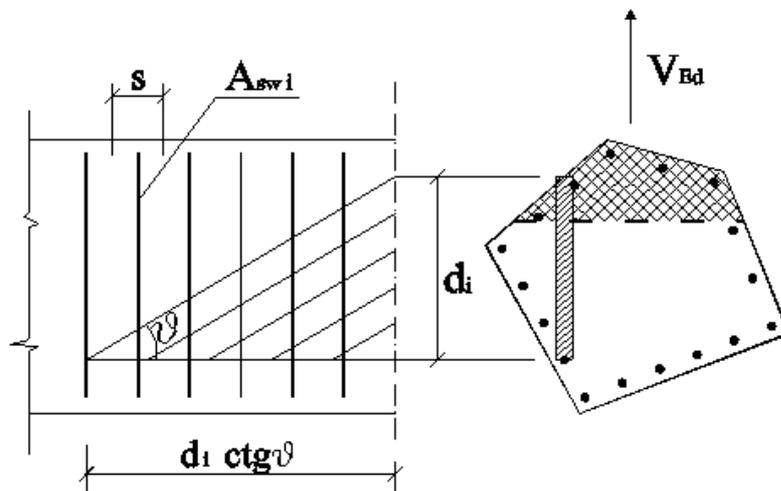
On the basis of the above assumptions the present figure shows the longitudinal section of the truss corresponding to the generic elementary resistant strut with an effective inner lever arm equal to z_i and with an effective depth equal to d_i. In order to fully assess the shear resistance expressed by (1) remains to be defined $A_{sw,i}$ ie the cross sectional area of the stirrups pertaining to the elementary strut. In (1) it is evident that such resistance is directly proportional to the height of the effective inner lever arm z_i, then it is logical to assume:

$$A_{sw\,i} = z_i / Sz_i \cdot A_{sw,i}$$

As a result the shear resistances, this time referring to the entire section, become:

$$V_{Rsd} = (Sz_i^2 / Sz_i) \, A_{sw} /s \cdot f_{yd} \cdot \cot\theta \qquad (3)$$

$$V_{Rcd} = S \, z_i \cdot b_i) \cdot a_c \cdot f'_{cd} \sin q \cdot \cos\theta \qquad (4)$$

§6.2.3(3) EC2 states that the shear resistance should be taken as the lesser of the two.

Equations (3) and (4) are equivalent to the corresponding EC2 formulas (6.8) and (6.9) provided to set:

$$z_{eff} = \qquad Sz_i^2 / Sz_i$$
$$d_{eff} \cdot b_{w, eff} = S \, (d_i \cdot b_i) = A_{eff}$$
$$b_{w, eff} = \qquad A_{eff} / d_{eff}$$

The effective values $b_{w \, eff}$, z_{eff}, d_{eff} thus obtained define, for any form of section, width and depth of an equivalent rectangular section for the purposes of shear resistance. These dimensions allow, i.e., <u>to use the same shear verification check illustrated for the predefined sections in uniaxial shear:</u>

$$V_{Rd, s} = \qquad z_{eff} \cdot A_{sw} /s \cdot f_{ywd} \cdot \cot\theta$$
$$V_{Rd, max} = \qquad z_{eff} \cdot b_{w, eff} \cdot \alpha_{cw} \cdot f'_{cd} \cdot \cot\theta / (1 + \cot^2 \theta)$$

3.2.2.1   Effective stirrups area

Given a high demand for biaxial shear (as in seismic zones) we try in the following, to assess the effective area resistant to shear provided not only by hoops, but also by cross-ties actually arranged in the section with the same pitch of the brackets. It is assumed here that the variation of the direction of the shear force (as normally occurs in various combinations provided) results in a consequent change in the effective resistant area to shear provided by hoops and cross-ties.

To fix ideas, consider the generic section of a column, represented in the following figure, reinforced with two hoops marked A, B and two cross-ties a, b.
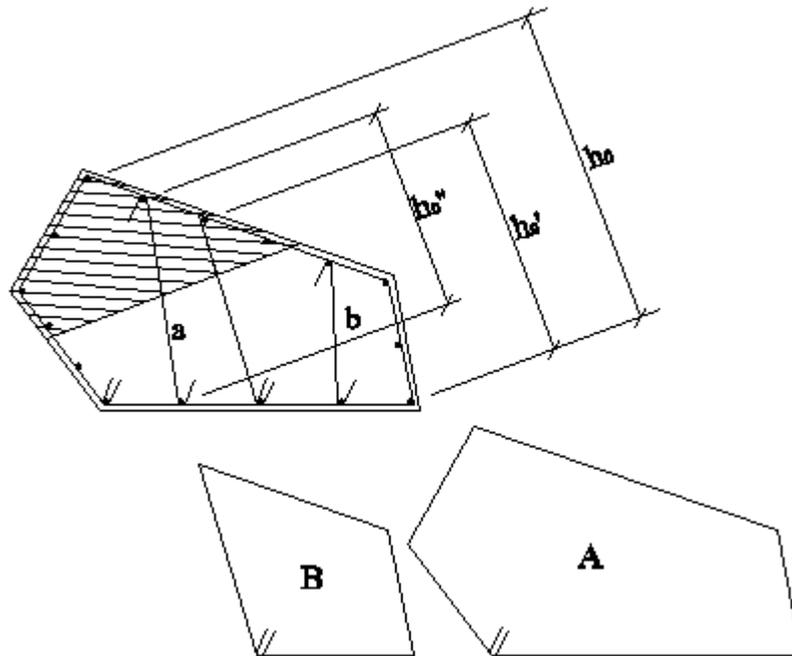
With reference to the perimetric hoop A we can say that, regardless of the direction of the shear, its contribution can always be assumed to be two times the area of the cross section of the hoop itself. The effectiveness of this hoop is indeed always maximal because it envelops in each case, the strut of the resistant truss at maximum depth for any direction of shear forces. Said maximum effectiveness may be commensurate to the maximum distance between the longitudinal bars (connected by hoop A) on the measured direction of the shear force (length h_0 in the figure). For every shear direction it is always possible to identify two resistant legs of the hoop which develop an effective shear resistant area equal to two times the area of the section of the hoop.

The effective area of inner hoops (such as B) , can be evaluated by reducing the area of the two legs by means of a factor equal to the ratio h_0'/ h_0 in which h_0' is the maximum distance between the bars enclosed by the inner hoop (always measured on the direction of the shear here assumed orthogonal to neutral axis).

With similar reasoning can be considered the effective area of the single cross-tie reduced to the ratio h_0'' / h_0 where h_0'' is the projection of the length of the tie on the shear force direction. On the safe side we may exclude (in the calculation of effective area) the cross-ties in which at least one of the two ends does not fall in the compression zone of the section.

Consider, for example, the below square section transversely reinforced with two hoops, as well square, which steel bars have the same cross-sectional area A_{br}. With reference to a shear force inclined of 45° to the

sides of the section, the effective area of the external perimeter hoop is equal to that of two legs: $2 A_{br}$. As said above the effective area of the inner bracket can be assessed equal to:

$2 A_{br} h_0 '/ h_0 = A_{br}$.

In total, therefore, the effective area resistant to a shear vector angle of 45 ° is equal to $3 A_{br}$.

In the case, instead, where the shear is parallel to the sides of the section, resulting $h_0 '= h_0$, we can assume as effective area $4 A_{br}$. For all intermediate inclinations, therefore, the effective area of the transverse reinforcement to be taken in checks of ductility capacity and of shear resistance ranges from $3A_{br}$ to $4 A_{br}$.
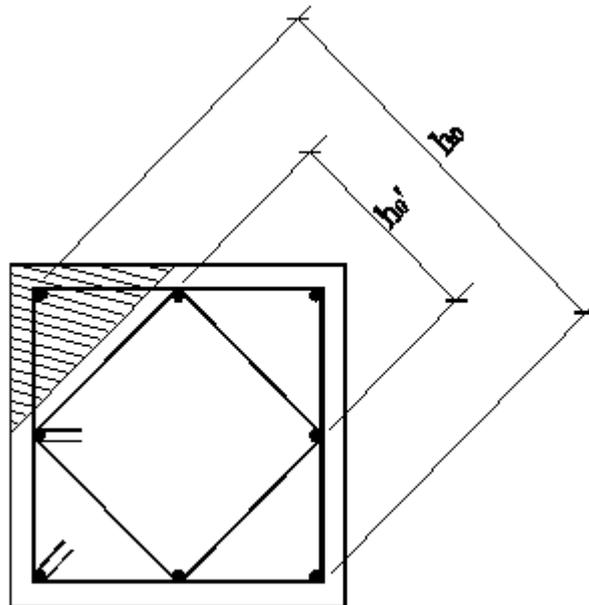
3.2.3 Shear check of walls

The check of seismic wall shear force (amplified automatically by the program) are performed according to EC2/EC8 rules assuming as inner lever arm equal to 80% of the width of the section, except in the case of the walls in ductility class DCH for which the shear strength compression $V_{Rd,max}$ should be reduced by 60% and assuming $\cot\vartheta = 1$ (what makes difficult the conformity of shear check for these walls in high ductility class. We recommend for seismic walls to adopt the ductility class DCM.
Sliding shear check is provided in EC8 only for walls in ductile class DCH (§ 5.5.3.4.4). The program does not provide for the presence of inclined bars at the base of the wall in the last mentioned check.
In ductility class DCH wall should also be verified for diagonal tension failure (§ 5.5.3.4.3 EC8).

The considerable number of checks and parameters entails, for ductile walls, the need to iterate the verifications by changing from time to time the reinforcement. To this end, we recommend to print at each step the full results (also on the screen) in which they are exposed eventual non-conforming values with reference to the standards requirements.

3.3 ULS Torsion

3.3.1 Torsion check

A full design for torsion shall be made only when the static design of a structure depends on torsional resistance of elements. In statically indeterminate structures where torsion arises from compatibility only, it will be normally unnecessary to consider torsion at ultimate limit states.

The program applies the method given in § 6.3 EC2 to calculate torsional resistance. Such method refers to thin walled closed sections in which equilibrium is satisfied by a closed shear flow. Solid sections may be modelled by equivalent thin-walled sections. Complex shapes, such T or flanged sections may be divided into more sub-sections, each of which is modelled by equivalent thin-walled sections, and the total resistance taken as the sum of the capacities of individual elements.

The method of calculation assumes an annular flow of shear stresses and, consequently, one can assume for resistance a tubular truss in which tensile stresses are balanced by the longitudinal and transverse reinforcement therein and the oblique compression forces by the struts of outer concrete.

Are explicitly excluded from this discussion (and from the program) sections that consist in open thin-walled profiles.

SLU check

EC2 (§6.3.2) define $t_{ef} = A/u$ as the effective wall thickness given by the ratio between the concrete area (including inner hollow area) and the outer perimeter u. The thickness t_{ef} need not be taken as less than twice the distance between edge and centre of longitudinal bars. For hollow sections the real thickness is an upper limit.

Torsion check in a polygonal equivalent thin-walled section is satisfied if the acting torsion force T_{Ed} is not greater of the following three torsional resistances provided by concrete struts and by the transverse (hoops) and longitudinal reinforcement:

a) Concrete:

$$T_{Rd,max} = 2 A_k \cdot t_{ef} \cdot f'_{cd} \cdot \cot \vartheta / (1 + \cot^2 \vartheta)$$

b) Hoops:

$$T_{Rsd} = 2 \cdot A_k \cdot A_s / s \cdot f_{ywd} \cdot \cot \vartheta$$

c) Longitudinal bars:

$$T_{Rld} = 2 \cdot A_k \cdot \Sigma A_{sl} / u_k \cdot f_{yd} / \cot \vartheta$$

where:

A_k area enclosed by the centre-lines of the connecting walls, including inner hollow areas

u_k perimeter of area A_k

A_s area of cross-section of the hoop bar (one bar)

s pitch of hoops

ΣA_{sl} cross-sectional area of longitudinal reinforcement for torsion

f'_{cd} $= v \cdot f_{cd}$ [see (6.9)EC2]

f_{ywd} design yield stress of the hoops

f_{ywd} design yield stress of the longitudinal bars

$\cot \vartheta$ $=$ cotangent of the angle ϑ of compression struts

The maximum resistance of a member subjected to torsion and shear is limited by the capacity of compression struts. In order not to exceed this capacity the following safety condition should be satisfied:

$$T_{Ed}/T_{Rd,max} \ + \ V_{Ed}/V_{Rd,max} \leq 1.0 \qquad\qquad (1)$$

This safety condition is named "safety factor" in the program output of predefined section.

When torsion and shear are simultaneously present the angle ϑ assumed in the checks must be the same.

For light action effects the absence of cracking is expressed by:

$$T_{Ed}/T_{Rd,c} \ + \ V_{Ed}/V_{Rd,c} \leq 1.0 \qquad\qquad (2)$$

where:

$$T_{Rd,c} = 2 \, f_{ctd} \cdot t_{ef} \cdot A_k$$

Predefined sections [12]

For this typology the program performs (with the above formulas) both the calculation of verification (check) that the design of the reinforcement. If combinations with torsion and shear force acting simultaneously interaction check is carried out with the relationship (1).

As default value of $\cot \vartheta$ is assumed that assigned in Code and reinforcement options [70] window. In presence of bending moment a portion of the upper and lower area of longitudinal bars for bending is reserved to torsion resistance (proportionally to the length of sides of the section). The area of web bars completes the longitudinal reinforcement for torsion. All the longitudinal area of bars resisting to torsion are non included in the resisting are for bending.

Polygonal sections in torsion[54]

Because of their generality for General section[26] typology is not expected torsion check in interaction with bending and shear. So for this section typology program performs only the torsion verification for the assigned reinforcement; also shown in output are the areas of stirrups and longitudinal bars strictly necessary to balance torsion force. Torsional concrete struts resistance $T_{Rd,max}$ is also calculated in order to manually control the above relationship (1). The torsional reinforcement areas may be, at last, added to those already calculated for bending and shear and so to get the final design of the section. You must assign for Cot ϑ the same value assumed for the shear force acting in the same combination.

The program proposes as results a disposition of the reinforcement that respect the detailing of standards (§9.2.3 EC2):

- The longitudinal bars spacing of the torsion link should not exceed u/8 or the requirement in 9.2.2(6) or the lesser dimension of the beam corss-section.

- The longitudinal bars should be so arranged that there is at least one bar at each corner, the others being distributed uniformly around the inner periphery of the links, with a spacing not greater than 350 mm.

3.4 ULS Instability

3.4.1 Nominal curvature method

The analysis of slender isolated columns may be performed with simplified methods such as that based on "nominal curvature method" used in the present program for the typology "rectangular section of columns[47]" and "predefined sections[23]" in the case of circular section of columns.

The nominal curvature method (§5.8.8 EC2) is primarily suitable for isolated member with constant normal force N_{Ed} and defined length l_0 (pre-calculated according to § 5.8.3.2 EC2). The purpose of this method is to give a nominal second order moment M_2 (based on effective length l_0 and an estimated max curvature) to add to the first order moment M_{0Ed} (inclusive of the effect of imperfections $e_0 = l_0/400$) so to have a final design moment M_{Ed}:

$$M_{Ed} = M_{0Ed} + M_2 \qquad\qquad (5.31)EC2$$

$$M_2 = N_{Ed} \cdot e_2 = N_{Ed} \, (1/r) \cdot l_0^2/c$$

where:

$$1/r = K_r \cdot K_\varphi \cdot 1/r_0 = K_r \cdot K_\varphi \cdot \varepsilon_{yd}/(0.45 \cdot d) \qquad (5.34)EC2$$

$c = 10$ for constant cross sections (or less if the first order moment is constant - 8 is a lower limit)
program use 10 always

$d = h/2 + i_s$ this value for effective depth is to be used if reinforcement is not concentrated on opposite sides

i_s radius of gyration of the total reinforcement area

K_r correction factor depending on axial load (see (5.36) EC2) : if $K_r > 1$ then $K_r = 1$

K_φ correction factor for taking account of creep (see (5.37)EC2) : if $K_\varphi < 1$ then $K_\varphi = 1$

The curvature $1/r_0$ in (5.34) § 5.8.8.3 EC2 can be assessed by the program with the direct formula in § 5.8.8.3 EC2 or by means the

moment-curvature diagram as in "model column" method (see ENV). In biaxial bending it is difficult to use the direct formula (5.34) as it is hard to meet the conditions of applicability in (5.38a) and (5.38b). These conditions are always checked by program. The biaxial moment-curvature diagram provided by the program make always possible to check biaxial instability. If curvature $1/r_0$ is assessed with moment curvature diagram (uniaxial or biaxial), only K_φ is used to correct the curvature as K_r is implicitly taken in account in the diagram.

Slenderness is defined as follows:

$$\lambda = l_0 / i$$

where:

l_0 is the effective length of the isolated member as defined in §5.8.3.2(2) EC2

i is the radius of gyration of the uncracked concrete section = $\sqrt{(J/A)}$

Second order effects may be ignored if the slenderness λ is below λ_{lim}:

$$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n} \qquad (5.13)$$

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

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

$C = 1.7 - r_m$ (if r_m is not known $C = 0.7$ may be used)

$\varphi_{ef} = \varphi_{ef} \cdot M_{0Eqp}/M_{0Ed}$ effective creep ratio [see (5.19)EC2]

$\omega = A_s f_{yd} / (A_c F_{cd})$ where As is the total area of long. bars

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

Program applies (5.13) setting $C = 0.7$ always (r_m is not assigned) and φ_{ef} as the input data assigned in the dialog window. The other parameter are directly calculated.

If $\lambda \leq \lambda_{lim}$ program stops calculations with a message.

3.5 Ductility checks

3.5.1 Local ductility for beams and columns

Ductility checks concern the following sections for structures in seismic zones:

- within critical regions at the ends of all the beams (DCM, DCH). [Not secondary beams]
- within critical regions at the base of columns in DCM
- within all critical regions of columns in DCH
- within critical regions of ductile walls (DCM, DCH)

To respect the local ductility in the above critical regions the curvature ductility factor μ_Φ must comply with the rule in §5.2.3.4 EC8:

$$\mu_\Phi \geq 1,5 \, (2q_0 - 1) \qquad \text{if} \qquad T_1 \geq T_C \qquad (5.4) \text{ EC8}$$

$$\mu_\Phi \geq 1,5 \, [1 + 2(q_0 - 1)] \, T_C / T_1 \qquad \text{if} \qquad T_1 < T_C \qquad (5.5) \text{ EC8}$$

where:

q_0 basic value of the behaviour factor

T_1 fundamental period of the building

T_C period at the upper limit of the constant acceleration of the spectrum

For beams EC8 allows to replace (5.4) above control with the equations (5.11), (5.12).
For columns EC8 allows to replace (5.4) above control with the relation (5.15).

For beams ductility check is normally performed by the program applying the eq. (5.11),(5.12) EC8 that that is, controlling the maximum and minimum tension reinforcement ratio of longitudinal bars.
For columns ductility check is normally performed by the program controlling the stirrups confining and applying the following disequation:

$$\alpha \, w_{wd} \geq 30 \, m_\Phi \, n_d \, \epsilon_{sy} \, b_c / b_0 - 0.035 \qquad (5.15)$$

EC8

where:

$\alpha \quad = \epsilon_s \cdot \epsilon_n = $ confinement effectiveness factor

w_{wd} = (volume of confining hoops/volume of concrete core) · f_{yd}/f_{cd} = $(A_{st,x}/(s \cdot b_0) + A_{st,y}/(s \cdot h_0)) \cdot f_{yd}/f_{cd}$ = $w_x + w_y$

m_f = curvature ductility facto.

n_d = $N_{Ed}/(A_c f_{cd})$ = normalized design axial force

$_{sy}$ = design value of tension steel strain at yield

b_c = gross cross-sectional width

b_0 = width of confined core

Eq. (5.15) should be applied in both directions x ed y (in a rectangular section):

$$\alpha \, w_{wd} \geq 30 \, m_\Phi \, n_d \,\,\,_{sy} \, b_c/b_0 - 0.035$$
$$\alpha \, w_{wd} \geq 30 \, m_\Phi \, n_d \,\,\,_{sy} \, h_c/h_0 - 0.035$$

Note that care must be taken to ensure the same ratio of transverse reinforcement that is to be approximately $w_x = w_y$.

As the program operates in biaxial bending and biaxial shear the last two formulas (5.15) EC8 are redefined as a function of only the actual y-direction inclined (as w_{wd} is the sum of both the amount of shear reinforcement in the two directions perpendicular of the section) of neutral axis and the relative amount of transverse reinforcement effective in only one direction y:

$$\alpha \, w_y \geq (30 \, m_\Phi \, n_d \,\,\,_{sy} \, d_c/d_0 - 0.035) / 2$$

where d_c e d_0 are equivalent to the transverse dimension (b_c, h_c) and the corresponding dimension of the confined core (b_0, h_0) obtained from the biaxial shear check. Then the control of the local ductility of the section is performed only once by the program for each of the seismic combinations and not twice (separately for the directions of the axes x and y) for each combination.

In the case of the sections belonging to existing structures calculated with non-linear method (pushover) the ductility in bending can no longer be calculated with the previous reports simplified (valid only in the case of calculation with behaviour structure factor q), but in its place must be assessed the capacity of rotation (of ductile mechanisms) from the ultimate curvature and the yield strength which, at the same time, define their relationship with the ductility in bending. The best way to figure these relations is to track the diagram bending moments-curvatures to which is dedicated the continuation of this paragraph. It is recalled that the maximum ductility demand for the columns is required in sections of attack with the foundations in which the model of seismic behavior provides for the formation of plastic hinges; in other critical areas of columns the ductility demand is lower (should not form plastic hinges thanks to the respect of the capacity design rules) and, for these areas, the EC8 does not require specific checks in DCM (for DCH also requires verification of ductility within the other critical regions of the columns but with reduced demand).

Curvature ductility factor μ_Φ of the critical regions is defined as the ratio of the post-ultimate strength curvature 85%- of the moment of

resistance, to the curvature at yield, provided the limiting strains of of concrete and steel $_{cu}$ and $_{su,k}$ are not exceeded.

EC8 does not provide details on both the meaning of the above definition or operational modalities with which to conduct the direct calculation of the said ductility factor. In the following we will refer to the specialized literature and to the first draft of Eurocode 8.

For a comprehensive evaluation of the ductile behavior of a section subjected to a constant normal force N is necessary to trace the diagram moment-curvature using the relationship stress-strain based upon the mean values of resistance (not the ones used for the design resistance check) of the concrete and steel.

To fix the ideas consider the below section (35x35 cm) of a column (reinforced with 8Φ18 longitudinal bars and with Φ6/12cm hoops and cross-ties) and its diagram Moment-Curvature (for N = 400 kN) obtained on the basis of the mean strength value for conglomerate (parabola-rectangle f_{ck} = 30 Mpa, f_{cm} = 38 Mpa and $_{cu}$ = 0.0035) and steel B450C (f_{ym} = 480 Mpa, f_{sum} = k·f_{ym} with k=f_{tk}/f_{yk} and $_{suk}$ = 0.0075). The ratio between the ultimate curvature (0.046600 1/cm) and the yield strength curvature (0.013044 1/cm) provides the following value of the ductility factor m_{Φ} = 3.57. The same section is studied in the example "Ductility check[155]".

M-Curv. diagram (N =400 kN = Const. Mx/My = 100.00/0.00 kNm = Const.)

Mcrack =27.71 kNm (Øcrack =0.000552)
My =176.14 kNm (Øy =0.013044)
Mult =206.51 kNm (Øult =0.046600)
Mmax =206.51 kNm (Ø =0.046600)
Curv. ductility factor CCDF = Øult/Øy = 3.57

Equivalent bilinear diagram(§E.3.2 EN 1998-2 bridges):
My Eq=198.60 kNm (Øy Eq.=0.014707)
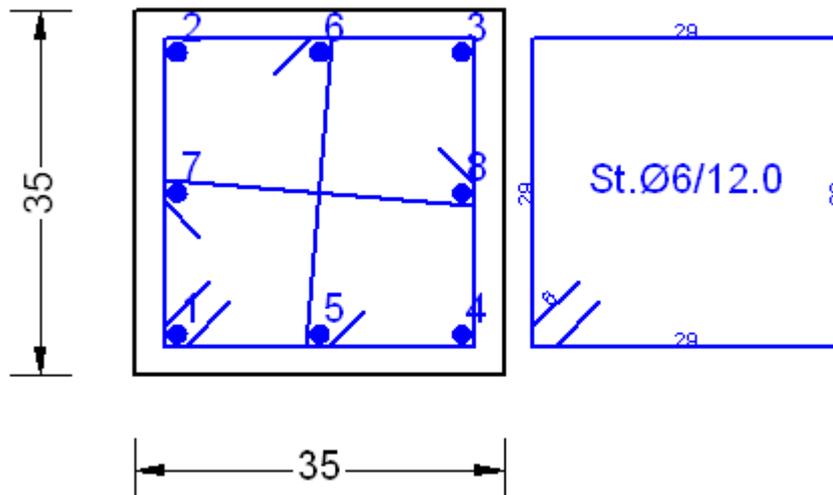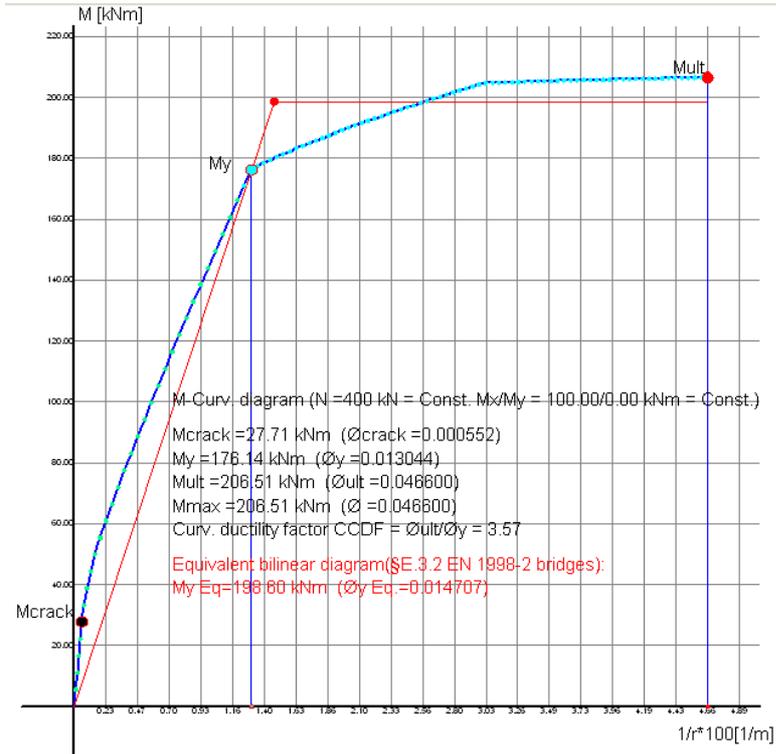
The low ductility factor obtained in this way is clearly unsatisfactory for a structure in a seismic zone calculated for example with a behaviour factor q_0 = 3.5, because for the above (5.4) it is required a ductility factor $\mu_\Phi \geq$ 2 × 3.5 - 1 = 6.0. The only factor that can improve the ductility is the increase of the ultimate strain concrete, which can be achieved by means of a dense stirrup confinement. In fact the increase of the confining pressure exerted by the stirrups on the concrete core produces an increase of the resistance f_{ck} (to $f_{ck,c}$) and of the strains ε_{c0}

ed ε_{cu} (to $\varepsilon_{c0,c}$ and $\varepsilon_{cu,c}$) as you may see in the below stress-strain relationship diagram.
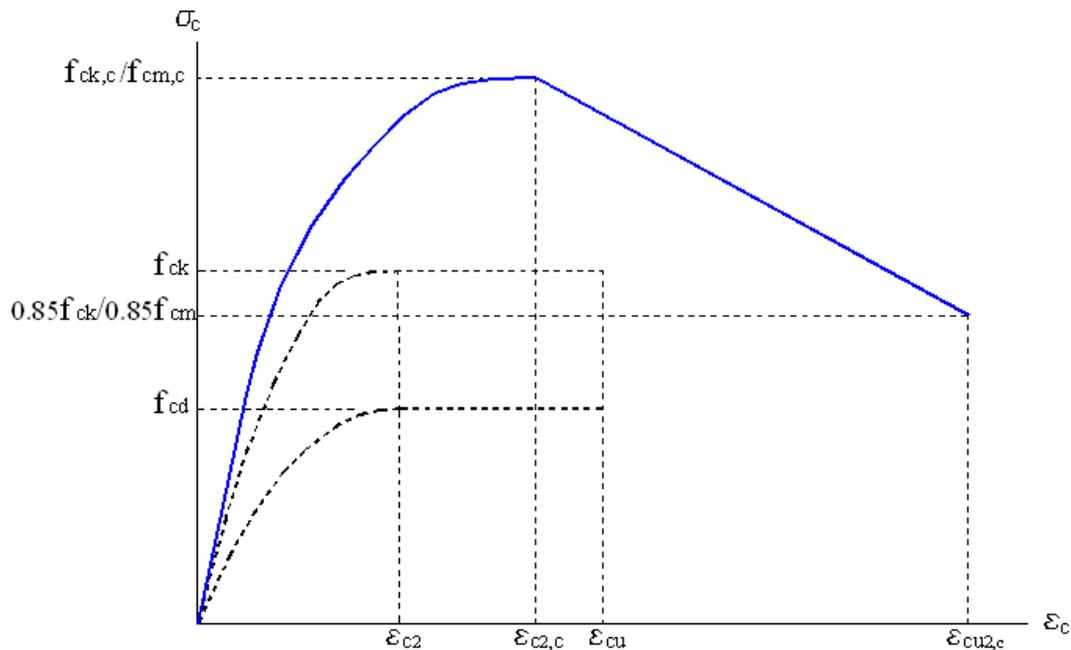
Blue line show the confined stress-strain relation of the confined concrete core of the section: the ascending branch has a parabolic form unlike the descending straight branch (softening effect) that is similar to the trend of a Kent-Park relationship type but here it is conventional and do not depends by the quantity of stirrups. Mean value of the confined strength ($f_{cm,c}$) is most suitable than the the characteristic ($f_{ck,c}$) in ductility assessment. The main variables that determine the confinement effect are:

- the ratio between the volume of the stirrups and the volume of the corresponding confined concrete (high amount of steel match high confining pressures);

- the yield stress of the steel of the stirrups as it provides an upper limit to the confining pressure explicable;

- stirrup spacing increase which reduces the effect of the arc confining pressure;

- the ratio between the diameter of the bar of the stirrups and the length of the largest side of the stirrups, due to the flexural stiffness that the bar plays in its plan to develop the confining pressure (this problem is not present in circular columns while for rectangular columns is resolved by transverse ties that connect the longitudinal bars and at the same time the hoops).

For predefined section[12], rectangular section of columns[38] and for

general section[26] consisting of only one concrete region the program can perform a direct ductility check (moment-curvature analysis based) automatically generating the above confined stress-strain relation based on the actual assigned hoops and cross-ties.

For the same typology of sections it is also possible to perform the direct ductility check using the following specific values of confined concrete parameters assigned in the Materials Library[67] (in this case you may directly model the inclination of the decreasing branch of the above diagram) :

- fc max $= f_{cm,c}$ (max strength in the above graph)

- fc ult $= 0.85 \, f_{cm,c}$ (ultimate strength in the above graph)

- ec 0 $= \varepsilon_{c2,c}$ (strain at $f_c = f_{cm,c}$)

- e0 ult $= \varepsilon_{cu2,c}$ (ultimate strain)); if $\varepsilon_c > \varepsilon_{cu2,c}$ then $f_c = 0$

- fcm $= f_{cm}$ max compressive stress for concrete part outside the confined core: if $\varepsilon_c > \varepsilon_{cu}$ (usually 0.0035) then $f_c = 0$

Parameters used in the ductility check for the steel bars are:

- Es $= E_{sm}$

- fym $= f_{ym}$ mean yield stress of stirrups steel

- fu $= k \cdot f_{ym}$ (ultimate strength) where $k = f_{td}/f_{yd}$ assigned in material library[67] for the stirrups steel

Let's now suppose that the above square section is within the critical region at the base of a column to design in ductility class DCM. The Local ductility condition (5.4) EC8 can be checked (this is the usual way) by means of the (5.15) relation that we firstly apply only with reference to Y direction (m_f= 6.8 min value required):

$$w_Y \geq (30 \, m_\Phi \, n_{d \; sy} \, h_c/h_0 - 0.035) / (2 \cdot \alpha)$$

(furthermore $w_Y \geq 0.08/2 = 0.04$)

where first member is:

$$\mathbf{w}_Y = A_{st,y} /(s \cdot h_0) \cdot f_{yd} / f_{cd} = 0.84823 / (12.00 \cdot 29.4) \cdot 391.3 / 20.0 = 0.047$$

- $A_{st,y}$ $= 3 \cdot 0.28274 = 0.84823$ cm² (area of 3 legs Φ 6)

- s $= 12$ cm (pitch of stirrups)

- h_0 $= 35 - 1.8/2 - 0.6/2 = 29.4$ cm (dimension along dir. Y of the confined core section measured between centreline of hoop)

and second member is:

$(30 \, m_\Phi \, n_d \, {}_{sy} \, h_c/h_0 - 0.035) / (2 \cdot \alpha) = (30 \cdot 6.8 \cdot 0.1633 \cdot 0.001956 \cdot 1.1905 - 0.035) / (2 \cdot 0.4554) = 0.0467 < \mathbf{w}_Y = 0.047 > 0.04$ (A)

- $n_d = N_{Ed}/(A_c \cdot f_{cd}) = 400000/(350^2 \cdot 20) = 0.1633$ (normalized design axial force)
- ${}_{sy} = f_{yd}/E_s = 391.3/200000 = 0.001956$ (design value of tension steel stirrups strain at yield)
- $h_c/h_0 = 35/29.4 = 1.1905$ (ratio of gross sectional depth (Y direction) to width of confined core)
- $\alpha = \alpha_n \cdot \alpha_s = 0.7189 \cdot 0.6335 = 0.4554$ (confinement effectiveness factor)
- $\alpha_n = 1 - \sum b_i^2/(6 \times b_0 \times h_0) = 1 - 8 \cdot 13.5/(6 \cdot 29.4 \cdot 29.4) = 0.7189$ [from (5.16a) EC8]
- $\alpha_s = (1 - s/2b_0)(1 - s/2h_0) = (1 - 12/(2 \cdot 29.4))^2 = 0.6335$ [from (5.17a) EC8]

[For a generic polygonal section $\alpha_s = A_{co}/A_0$ where A_0 is the area enclosed by the centreline of the hoop perimeter and A_{co} is the area of the polygon interior to A_0 obtained with an offset of the sides of A_0 equal to s / 4. Always a polygonal cross section in the formula of the product an $b_0 \cdot h_0$ is replaced with the newly defined area A_0.]

In (A) we see that the ductility check (5.15) is satisfied in Y direction.
If we use the same eq. (5.15) with a 45° inclined direction and with the same value for m_Φ we have also a positive conformity for local ductility:

$\mathbf{w_{45°}} = 0.049 > 0.047$

Now we want to check local ductility by means the moment-curvature diagram with the above stress-strain relation for mean confined value of strength.
To evaluate confined parameters of concrete we calculate first the effective lateral compressive stress σ_2 (generated by the designed closed link and cross-ties) with the following experimental formula (see Model Code 1990):

$\sigma_2 = 0.5 \cdot f_{cm} \cdot \alpha \cdot \omega_w = 0.5 \cdot 38.0 \cdot 0.4554 \cdot 0.06074 = 0.526$

where:

ω_w = (volume of confining stirrups / volume of concrete core) $\cdot f_{ym}/f_{cm} = (6 \cdot 29.4 \cdot 0.28274 / (12 \cdot 29.4^2)) \, 480.0/38.0 = 0.0607$

Known σ_2 we can apply EC2 formulas (3.24), (3.25), (3.26), (3.27) to assess the parameters of confined concrete:

$$f_{cm,c} \quad = \quad f_{cm}(1{,}000 + 5{,}0\,\sigma_2/f_{cm}) \qquad \text{if} \qquad \sigma_2 \leq 0{,}05\,f_{cm}$$

$$f_{cm,c} \quad = \quad f_{cm}(1{,}125 + 2{,}5\,\sigma_2/f_{cm}) \qquad \text{if} \qquad \sigma_2 > 0{,}05\,f_{cm}$$

$$f_{cu,c} \quad = \quad 0{,}85 \cdot f_{cm}$$

$$\varepsilon_{c2,c} \quad = \quad \varepsilon_{c2}\,(f_{cm,c}/f_{cm})^2$$

$$\varepsilon_{cu2,c} \quad = \quad \varepsilon_{cu} + 0{,}1 \cdot \alpha \cdot \omega_w$$

In our case $\sigma_2 < 0{,}05\,f_{cm} = 1.9$ Mpa then:

$$f_{cm,c} = \quad 38\,(1.000 + 5.0 \cdot 0.526/38) = 40.6 \text{ Mpa}$$

$$f_{cu,c} = \quad 0.85 \cdot 38 = 32.3 \text{ Mpa}$$

$$\varepsilon_{c2,c} = \quad 0.002\,(40.6/38)^2 = 0.00228$$

$$\varepsilon_{cu2,c} = 0.0035 + 0.1 \cdot 0.4554 \cdot 0.06074 = 0.00626$$

Note the increase, thanks to the confining stirrups, of ultimate concrete strain $\varepsilon_{cu2,c}$ (almost 2 times the not confined concrete value 0.0035).

In the below figure is shown the moment-curvature diagrams with the new confined parameters and the corresponding stress-stain relation (blue line in the above graph).
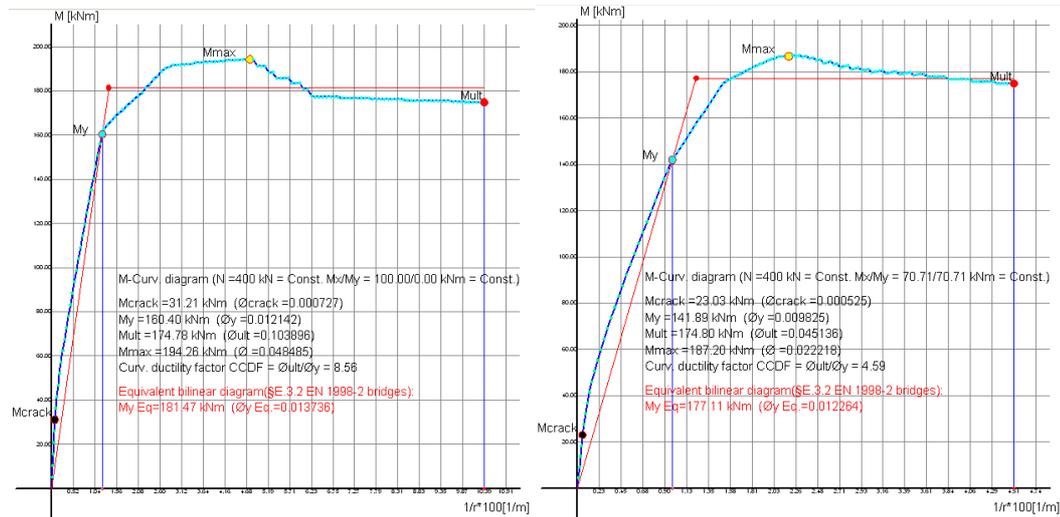
The diagram on the left refers to the uniaxial direction Y from which we obtain a good (conformity OK) ductility factor $m_\Phi = 8.56 > 6.8$ (min required value); note the sudden drop of the diagram for the crushing of the cover (not confined concrete).

The diagram on the right refers to the biaxial direction 45° inclined (in each point of the biaxial diagram the ratio Mx/My is held constant and equals to the ratio of ULS design moments) from which we do not obtain a sufficient (conformity Not Ok) ductility factor $m_\Phi = 4.59 < 6.8$ (min required value); in general for all inclined direction of the seismic forces ductility factor m_Φ is less than than the uniaxial directions X or Y.

Note that the above application of the (5.15)EC8 for the 45° inclination leads to a positive ductility check in contrast with the more realistic assessment obtained with the moment-curvature diagram.

An additional moment-curvature diagram generated by the program (lower option in the stirrups-ductility window) is based on the mean compressive stress-strain low described in § 3.1.5 EC2 for not linear analysis of frames.

For this last diagram it is also possible to assign the % of the mean concrete tension strength f_{ctm} to take in account in the tension region of the following assumed stress-strain diagram:
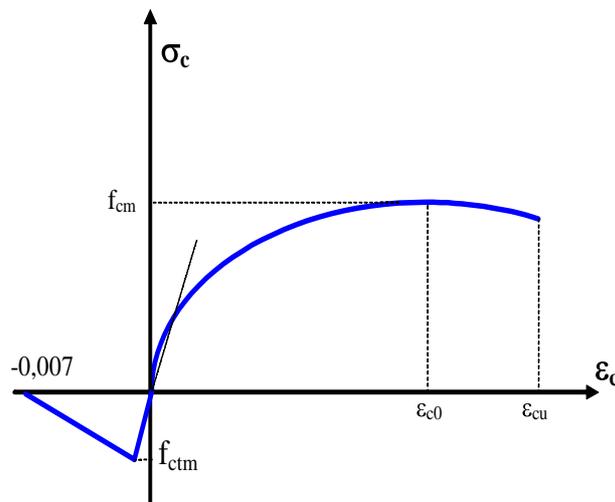
3.5.2 Local ductility of walls

For ductile wall Ec8 prescribes a lot of detailing rules in § 5.4.3.4.2.
In the critical region of the wall the width L_c of the confined boundary zones (see below figure) must be not lesser than 0.15 l_w and of 1.5 b, and the minimum longitudinal reinforcement ratio is 0.005.
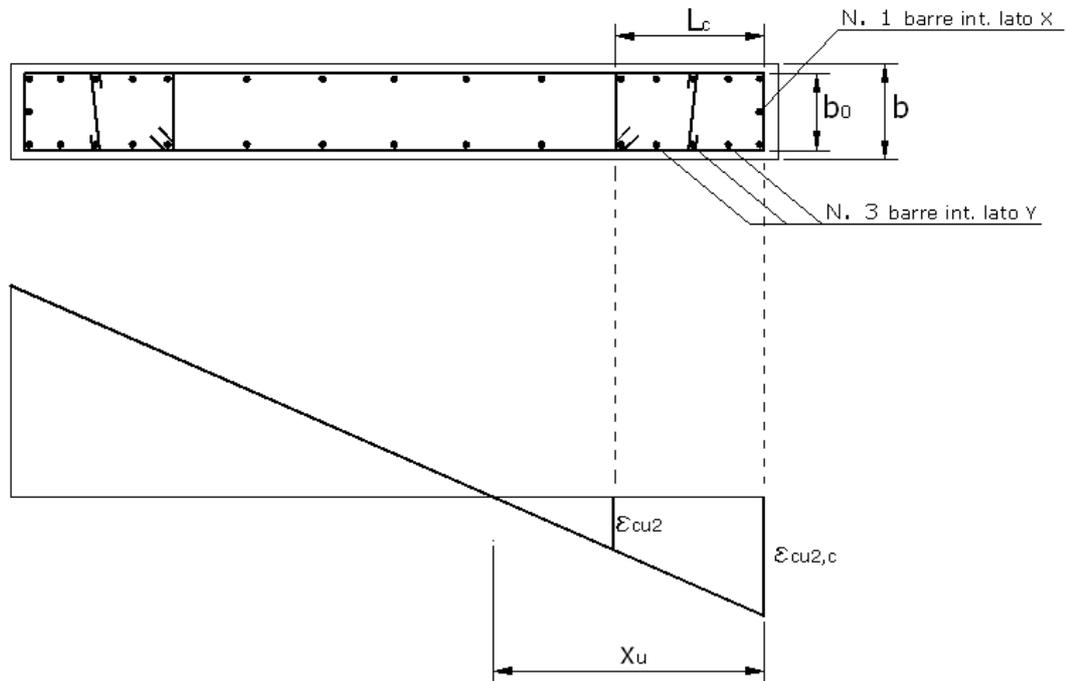
For large lightly reinforced walls the length L_c of the boundary zones should extend (in l_w direction) for a value not less than b or $2 \cdot b \cdot s_{cm}/f_{cd}$ where s_{cm} is the mean value of the compressive stress.

The local ductility check to perform for the ductile walls in the critical region is ensured by the following relation:

$$\alpha \, w_{wd} \geq 30 \, m_f \, (n_d + w_v) \, _{sy} \, b_c/b_0 - 0.035 \qquad (5.20) \text{ EC8}$$

in cui:

$\alpha \qquad = \alpha_s \cdot \alpha_n$ = confinement effectiveness factor in L_c end zoned (as in the in previous topic[107])

$w_{wd} \qquad = (A_{st,x}/(s \cdot b_0) + A_{st,y}/(s \cdot h_0)) \cdot f_{yd}/f_{cd} = w_x + w_y$

$m_f \qquad$ required curvature ductility factor to calculate with (5.4) or (5.5) EC8 where q_0 is amplified with the factor M_{Ed}/M_{Rd}

$w_v \qquad$ = mechanical ratio of vertical web reinforcement ($r_v \, f_{yd}/f_{cd}$).

$w_{wd} \geq 0.08$ in CDM class and 0.12 in CDH class.

It should also be checked for length L_c of the areas confined primarily considering the length of the compressed area x_u ULS (see figure) after spalling of the concrete cover:

$$x_u = (n_d + w_v) \, l_w \, b/b_0$$

where l_w is the width of the section and b_0 is the thickness of the confined end zones.

L_c must be $\geq$ L' length where concrete strain if greater than 0.0035 :

$$L_c \geq L' = x_u \, (1 - {}_{cu2}/{}_{cu2,c})$$

with:

${}_{cu2} = 0.0035$

${}_{cu2} = 0.0035 + 0.1 \, \alpha \, w_{wd}$

3.6 SLS checks

3.6.1 Limitation of stresses

With reference to SLS combinations compressive stresses in the concrete shall be limited in order to avoid longitudinal cracks, micro-cracks and high level of creep. Stresses in the reinforcement shall be limited in order to avoid unacceptable cracking or deformations.

In the calculation of stresses cross sections are assumed by the program uncracked if concrete flexural tensile stress does not exceed f_{ctm} and if the option "Crack width always in cracked state" (see General Data $\boxed{9}$ window) is not checked. But if in at least one combination among those assigned, is exceeded the value of f_{ctm}, then all combinations are checked as cracked. If the above option is checked then cross section is assumed cracked for all SLS combinations without tensile control.

For characteristic combination forces program assumes as critical value of concrete compressive stress: $k_1 \cdot f_{ck}$ ($k_1 = 0.60$ is the recommended value; to modify k_1 in according to national annex see Code and reinforcement options $\boxed{70}$).

For characteristic combination forces program assumes as critical value of steel tension: $k_3 \cdot f_{yk}$ ($k_3 = 0.80$ is the recommended value).

For quasi-permanent combination forces program assumes as critical value of concrete compressive stress: $k_2 \cdot f_{ck}$ ($k_2 = 0.45$ is the recommended value).

Program provides two calculation method for SLS stresses evaluation: Simplified method and AEEM (or EM) method.

Simplified method (n method)

The stress-strain relationship is always linear both for the concrete (for tension stress not exceeding f_{ctm}) that for the steel. Area of steel bar is amplified with the factor n = E_s/E_c. In general the value assumed for n is 15 to take in account creep effects (long time effects) on stresses redistribution. Anyhow you can set n value for each concrete class in Materials Library $\boxed{67}$ window (for characteristic combinations only may be better not to take in account creep and set n = E_{sm}/E_{cm}).

Calculation procedure:

for the hypothesis of conservation of plane sections the deformation of the cross section may be expressed by means of a plane $\varepsilon(x,y)$ the equation of which is:

$$\varepsilon(x,y) = a\,x + b\,y + c \qquad\qquad (1)$$

where *a, b* are the curvatures of the plane around x and y axes of refence assumed, and *c* is the strain at the origin of refence axes.

Now we express the neutral axis as the intersection of above plane (1) with the plane undeformed $\varepsilon(x,y) = 0$:

$$a \text{ x} + b \text{ y} + c = 0 \qquad\qquad (2)$$

The stresses in a generic point x,y of the concrete region j are:

$$\sigma_{cj}(x,y) = E_{cj} \ \varepsilon(x,y) = E_{cj} \ (a \text{ x} + b \text{ y} + c) \qquad \text{if } \varepsilon > 0$$
$$\tag{3}$$
$$\sigma_{cj}(x,y) = \ 0 \qquad\qquad\qquad\qquad\qquad \text{if } \varepsilon \le 0$$

where E_{cj} is the elastic modulus of concrete region j.

similarly, for the generic steel bar which centre has coordinates x_i, y_i:

$$\sigma_s (x_i, y_i) = E_s (a \text{ x}_i + b \text{ y}_i + c) \qquad\qquad (4)$$

indicating with A_j the effective area (which stress is greater then f_{ctm}) of region j and with A_i the area of the single bar i, the three equations of equilibrium are:

$$N = \int_{Aj} \sigma_c (x,y)\, dA + \sum \sigma_s(x_i,y_i)\, A_i$$

$$Mx = \int_{Aj} y\, \sigma_c(x,y)\, dA + \sum y_i\, \sigma_s(x_i,y_i)\, A_i$$

$$My = \int_{Aj} (-x)\, \sigma_c(x,y)\, dA + \sum (-x_i)\, \sigma_s(x_i,y_i)\, A_i$$

Areas, static moments and moments of the second order, can be written:

$$A = \sum_j E_{cj} \int_{Aj} dA + E_s \sum A_i$$

$$Sx = \sum_j E_{cj} \int_{Aj} y\, dA + E_s \sum y_i A_i$$

$$Sy = \sum_j E_{cj} \int_{Aj} x\, dA + E_s \sum x_i A_i$$

$$Jx = \sum_j E_{cj} \int_{Aj} y^2\, dA + E_s \sum y_i^2 A_i$$

$$Jy = \sum_j E_{cj} \int_{Aj} x^2\, dA + E_s \sum x_i^2 A_i$$

$$Jxy = \sum_j E_{cj} \int_{Aj} xy\, dA + E_s \sum x_i y_i A_i$$

Substituting (3) into (5) and taking into account last relations, we obtain the following non-linear system in the unknowns *a, b, c* :

$$\begin{vmatrix} N \\ Mx \\ My \end{vmatrix} = \begin{vmatrix} A & Sx & Sy \\ Sx & Jx & Jxy \\ Sy & Jxy & Jx \end{vmatrix} \begin{vmatrix} c \\ b \\ a \end{vmatrix} \quad (5)$$

The iterative solution of the above system provides the coefficients *a, b, c* and then the stresses by means of the relationships (3) and (4).

AAEM method

AAEM (Age Adjusted Effective Modulus) provides a more refined evaluation of stresses and deformations of cross sections taking into account the assigned values (in Materials Library[67])of creep $\varphi(t,t_0)$, shrinkage $\varepsilon(t,t_0)$ and of the time of load application (for cracked and uncracked sections). EC2 standards not provide any guidance on this method of calculation, but merely provide the rheological data of the concrete (see Concrete rheological coefficients[73]).

You can assume this method for SLS checks selecting the pertinent option in General data[9] window. Program use as default this method in SLS deformation checks[126].

AAEM method is based on a double calculation of the same section:

- evaluation of instantaneous stresses at time t_0 of first loading;

- evaluation of stresses changes due to creep and shrinkage from t_0 to the final time t. Program assume the simplifying assumption that, after t_0, bending and axial force remain constant up to time t (this is strictly true only for isostatic members).

For a compressed concrete sample not reinforced and without any constrains the creep coefficient $f(t,t_0)$ is defined at time t_0 as:

$$f(t,t_0) = s_0 / e_0 \qquad (6)$$

where $e_0 = s_0 / E_c(t_0)$ \qquad (7)

This theory is valid for $s_0 \leq 0.45\, f_{ck}(t_0)$.

If from (6) we express the strain e_0 in function of time we have:

$$e(t,t_0) = s_0 [1 + f(t,t_0)]$$

If after t_0 of first stress s_0 , we have a gradual reduction of stress ds, the superposition principle allows us to express the strain at time t, as follows (including shrinkage e_{cs} too):

$$e(t) = e_{cs}(t,t_0) + s_0 [1 + f(t,t_0)] / E_c(t_0) + E_c(\tau) [1+f(t,\tau)]\, ds \qquad (8)$$

To avoid the difficulties of integrations of differential relation (8) the method replaces the actual load history $\sigma(t)$ with a single stress increase $\Delta\sigma$ and a creep coefficient $f(t,t_0)$ reduced by means of the so-called coefficient of aging $c = c(t,t_0)$:

$$e(t) = s_0 \; [1+f] \, / \, E_c(t_0) \; + \Delta s(t) \; [1+c \; f] \, / \, E_c(t_0) \; + \; e_{cs} \tag{9}$$

where $e_{cs} = e_{cs}(t,t_0)$ = shrinkage function.

The value of $c(t,t_0)$ can vary from 0,5 to 1. In most practical problems affects only the final value $c(t_\infty,t_0)$: for $t_0 > 15$ days that value may be assumed always equal to 0.8.

The <u>age adjusted modulus of elasticity</u> is then:

$$E_c^*(t,t_0) = E_c(t,t_0) \, / \, (1+ c.f) \tag{10}$$

and then:

$$e(t) = s_0 \; [1+f] \, / \, E_c(t_0) \; + \Delta s(t) \, / E_c^*(t,t_0) \; + \; e_{cs}(t,t_0) \tag{11}$$

A graphical representation if the relation (11) is shown below where the first increase of the concrete stress is the segment O-A due to the instantaneous application of forces (initial elastic strain $e_0 = s_0 \, / \, E_c(t_0)$ in the first term of (11)).

The horizontal segment A-B corresponds to the application of the free distortion due to creep ($s_0 \cdot f(t,t_0)/ E_c(t_0)$) + shrinkage ($e_{cs}(t,t_0)$, from time t_0 to t.

Segment B-C is the second term of the (11) and is equivalent to a second elastic solution using the modulus adjusted for aging E_c^*.
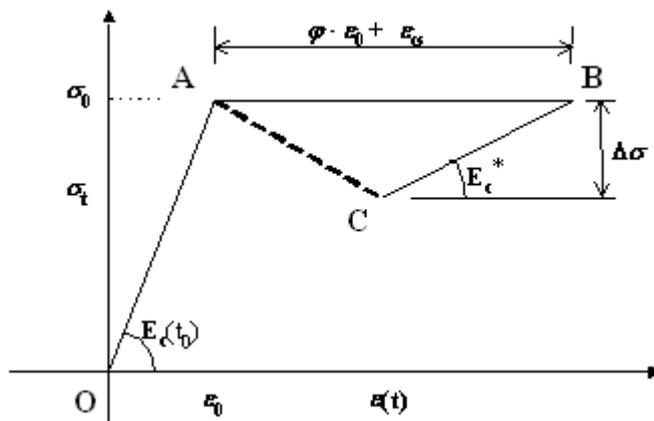
<u>Stresses evaluation with AAEM method</u>

We study a general cross section consisting of one or more concrete regions (also of different class and rheological characteristics), and a set of longitudinal bars however placed.

In a first phase (at time t_0) section is subjected to the forces N, Mx, My to which correspond the strains a_0, b_0, c_0 obtained by the iterative solution

of the linear system (5), with elastic modulus E_{cj} and evalued at time t_0. Elastic modulus of steel bars is always E_s in the two pases. From a_0, b_0, c_0, we know the stresses (3) in every point of the section..

$\Delta a_0, \Delta b_0, \Delta c_0$ from time t_0 to final time t due to creep and shrinkage at constant values of N, Mx, My forces.

Now we apply to the section the forces N^*, Mx^*, My^* such as to prevent any deformation caused by the rheological phenomena. For each concrete region j (no constrains with other regions and with and with bars) at time t the strains would be:

$$\Delta a_j = f_j(t,t_0) \cdot a_0$$
$$\Delta b_j = f_j(t,t_0) \cdot b_0 \tag{12}$$
$$\Delta c_j = f_j(t,t_0) \cdot c_0 + e_{cs\,j}(t,t_0)$$

To get the forces N*, Mx*, My* we perform the following numerical integrals with the adjusted modulus $E_{cj}^* = E_{cj}^*(t,t_0)$:

$$N^* = \sum_j E_{cj}^* A_{cj} \Delta c_j + \sum j\, E_{cj}^* Sx_{cj} \Delta b_j + \sum_j E_{cj}^* Sy_{cj} \Delta a_j$$

$$Mx^* = \sum_j E_{cj}^* Sx_{cj} \Delta c_j + \sum j\, E_{cj}^* Jx_{cj} \Delta b_j + \sum_j Ecj^* Jxy_{cj} \Delta a_j \tag{13}$$

$$My^* = \sum_j E_{cj}^* Sy_{cj} \Delta c_j + \sum j\, E_{cj}^* Jxy_{cj} \Delta b_j + \sum_j E_{cj}^* Jy_{cj} \Delta a_j$$

or, in matrix notation:

$$
\begin{vmatrix} N^* \\ Mx^* \\ My^* \end{vmatrix} = \sum_j E_{cj}^* \begin{vmatrix} A_{cj} & Sx_{cj} & Sy_{cj} \\ Sx_{cj} & Jx_{cj} & Jxy_{cj} \\ Sy_{cj} & Jxy_{cj} & Jy_{cj} \end{vmatrix} \begin{vmatrix} \Delta c_j \\ \Delta b_j \\ \Delta a_j \end{vmatrix}
$$

$$\tag{14}$$

To restore the consistency of the deformations in the overall section we have to resolve the following system (5) with $E_{cj} = E_{cj}^*$:

$$
\begin{vmatrix} N^* \\ Mx^* \\ My^* \end{vmatrix} = \begin{vmatrix} A & Sx & Sy \\ Sx & Jx & Jxy \\ Sy & Jxy & Jy \end{vmatrix} \begin{vmatrix} \Delta c_0 \\ \Delta b_0 \\ \Delta a_0 \end{vmatrix}
$$

$$\tag{15}$$

Known $\Delta a0, \Delta b0, \Delta c0$, the stress increase fro t_0 to t is calculated with the equation (9) applied at point x,y of the region j:

$$\Delta s_j = E_{cj}^* \Delta e_{cj} - E_{cj}^* (f_j \cdot e_0 + e_{cs\,j})$$

where:

$$\Delta e_{cj} = \Delta a_0 \cdot x + \Delta b_0 \cdot y + \Delta c_0$$
$$e_0 = a_0 \cdot x + b_0 \cdot y + c_0$$

The total strain in the section at time t is:

$$a_{tot} = a0 + \Delta a0 = \chi_{y\ tot}$$
$$b_{tot} = b0 + \Delta b0 = \chi_{x\ tot} \tag{16}$$

$$c_{tot} = c0 + \Delta c0 = e_{Origine}$$

So that neutral axis is given by:

$$a_{tot}\ x + b_{tot}\ y + c_{tot} = 0$$

Final stress in concrete are:

$$s_{tot} = s_t = s_0 + \Delta s$$

Final stress in the bar i of coordinates x_i, y_i :

$$s_{i\ tot} = s_{i\ 0} + \Delta s_i$$

where:

$$s_{i0} = E_s\ (a_0 \cdot x_i + b_0 \cdot y_i + c_0)$$
$$\Delta s_i = E_s\ (\Delta a_0 \cdot x_i + \Delta b_0 \cdot y_i + \Delta c_0)$$

To simplify the calculations in the case of cracking of section in the first loading (at time t_0), we regard the compressed part of the section as not variable in form from t_0 to t.

EM method

The fundamental relation of AAEM method is:

$$E_c^*(t,t_0) = E_c(t,t_0) / (1 + c.f)$$

(10)

EM method (*Effective Modulus method*) is a simplification of (10) where $\chi = 1$:

$$E_c^*(t,t_0) = E_c(t,t_0) / (1 + f)$$

(17)

Because of the uncertainty of the evaluation of the mechanical and rheological parameters the numerical difference between the two methods is minimal so it would seem useless the treatment carried out for AAEM method with respect to that leaner permitted by (17). In fact, the advantage of the AAEM method consists in the ability to take account of the shrinkage jointly with creep, while using the EM method the effect of the shrinkage must be calculated separately with different

procedure (see (7.21) in §7.4.3 EC2 valid for beams but not for columns).

3.6.2   Limitation of crack widths

Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure. The limit recommended values w_{max} for crack width are given, for relevant exposure classes and for quasi-permanent combinations, in the table 7.1 of EC2 :

0.4 mm for classes X0, XC1
0.3 mm for classes XC2,XC3,Xc4,XD1,XD2,XS1,XS2,XS3

The two value can be modified in Code and reinforcement options[70] window.
Program controls cracking by means direct calculation of crack widths (§7.3.4 EC2).
The characteristic crack width w_k may be obtained from the relation:

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

where

$s_{r,max}$ maximum crack spacing for final crack state according to eq. (7.11)

ε_{sm} mean strain of reinforcement considering contribution of tension stiffening

ε_{cm} mean strain in concrete between cracks

$$s_{r,max} = k_3 \cdot c + (k_1 \cdot k_2 \cdot k_4 \cdot \Phi) / \rho_{eff}$$

where

k_3 factor (recommended value: 3.4)

c concrete cover of longitudinal reinforcement

k_1 factor to consider bond properties (0.8 for bars with high bond properties; 1.6 for bars with plain surface)

k_2 factor to consider distribution of strain (0.5 for bending; 1.0 for tension only)

k_4 factor (recommended value: 0.425)

Φ is the diameter of tensile bars within $A_{c,eff}$

The setting of bond properties of bars k_1 and of recommended values is possible in Code and reinforcement options[70] window.

$$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - k_t \cdot f_{ct,eff}/\rho_{eff} \cdot (1 + \alpha_e \, \rho_{eff})] / E_s \geq 0.6 \cdot \sigma_s / E_s$$
where

σ_s stress in tension reinforcement under assumption of cracked section

k_t factor for creep of bond (k_t = 0.6 for short-term loading (frequent comb.); k_t = 0.4 for long-term loading (quasi-permanent comb.)

$f_{ct,eff}$ effective tensile strength of concrete at relevant point of concrete: in program is always equal to f_{ctm}

ρ_{eff} = $A_s/A_{c,eff}$

A_s total area of bars placed within $A_{c,eff}$

$A_{c,eff}$ effective tension area (surrounding the tension bars) of depth $h_{c,ef}$, where $h_{c,ef}$ is the lesser of 2.5(h-d) , (h-x/3) or h/2 (see Figure 7.1 in EC2)

Program allows to calculate (see check box "Crack width (effective tension area)" in General data⌐9⌐ window) coefficient k_2 with reference to the fibre of the area $A_{c,eff}$ less tensioned and <u>also with compression axial force</u>, so to have:

$k_2 = (\varepsilon_1 + \varepsilon_2)/2\varepsilon_1$ (7.14) EC2

where

ε_1 is the greater and ε_2 the lesser tensile strain at the boundary of $A_{c,eff}$ in cracked section

The definition of $A_{c,eff}$ is easy for predefined sections in uniaxial bending; but for biaxial bending or for general section the program implements the following criterion:

- the lengths 2.5(h-d) ,(h-x/3), h/2 are measured normally to neutral axis (so to have the effective inclined depth $h_{c,eff}$)

- $A_{c,eff}$ is the area obtained adding the concrete circular areas around all tensile bar within the depth $h_{c,eff}$: the circle diameter is 7 times the diameter Φ of the correspondent bar.

The entire area of all section is discretized in little squares that are summed to give $A_{c,eff}$ if their centre is within the depth $h_{c,eff}$ and within the above circle.

Minimum reinforcement areas check

If the section is in Low Ductility Class (only EC2 applies) program compares the areas A_s of bars included in the effective tension area $A_{c,eff}$ with the minimum tension area $A_{s,min}$ specified by EC2:

$$A_{s,min} \, \sigma_s = k_c \, k \, f_{ctm} \, A_{ct} \qquad \text{(7.1) EC2}$$

where

A_{ct} is the area of concrete in the tensile zone calculated just before formation of the first crack.

σ_s is the absolute value of the max stress permitted in the reinforcement immediately after the formation of the crack

k coefficient which allows for the effect of non uniform self-equilibrating stresses

= 1.0 for webs with h≤300 mm or flanges with widths less than 300 mm

= 0.65 for webs with h≥800 mm or flanges with widths greater than 800 mm

intermediate values may be interpolated

k_c coefficient which take account of the nature of stress distribution

= 1.0 for pure tension

= 0.4 $[1-\sigma_c/(k_1 (h/h^*)F_{ctm})] \leq 1$ for bending in webs of box section and T sections

= 0.9 $F_{cr} / (A_{ct} f_{ctm}) \geq 0.5$ for flanges of box sections and T sections

σ_c = N_{Ed}/bh mean stress of the concrete acting in the part of the section under consideration

h^* = h for h < 1.0 m

= 1.0 m for h ≥ 1.0 m

k_1 coefficient considering the effects of axial force on stresses

= 1.5 if N_{Ed} is a compressive force

= $2h^*/(3 \, h)$ if N_{Ed} is a tensile force

F_{cr} absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment

3.6.3 Limitation od deformations

§ 7.4.3(1)P states that when a calculation is deemed necessary, the deformations shall be calculated under load conditions which are appropriate to the purpose of the check.

The calculation method adopted by the program is the most rigorous outlined in §7.4.3(7): curvatures are evaluated at frequent sections along the member and then are calculated the deflections twice,

assuming the whole member to be in the uncracked and fully cracked condition in turn, and then interpolate using the following expression:

$$\alpha = \zeta \, \alpha_{II} + (1 - \zeta) \, \alpha_{I} \qquad (7.18) \; EC2$$

where

α   is the deformation parameter considered which may be, for example, a strain, a curvature, or a rotation

α_{I}, α_{II} are the values of the parameter calculated for the uncracked and fully cracked conditions respectively

ζ   is a distribution coefficient (allowing for tension stiffening at a section) given by the expression:

$$\zeta = 1 - \beta \, (\sigma_{sr}/\sigma_{s})^{2} \qquad (7.19) \qquad EC2$$

β   is a coefficient taking account of the influence of the duration of the loading or of repeated loading

= 1.0 for a single short-term loading. <u>Program assumes this value when Characteristic or Frequent comb. option is checked</u>

= 0.5 for sustained loads or many cycles of repeated loading. <u>Assumed by program when Quasi-permanent comb. option is checked</u>

σ_{s} is the stress in the tension reinforcement calculated on cracked section

σ_{sr} is the stress in the tension reinforcement calculated on a cracked section under loading causing first cracking

$\sigma_{sr}/\sigma_{s} = M_{cr}/M = f_{ctm}/\sigma_{ct,min}$ where M_{cr} is the cracking moment; $\sigma_{ct,min}$ is the min concrete tension in the uncracked section

For loads with a duration causing creep, the total deformation including creep may be calculated by using an effective modulus of elasticity for concrete according to:

$$E_{c,eff} = E_{cm} / (1 + \varphi(\infty, t_{0})) \qquad (7.20) \; EC2$$

where

$\varphi(\infty, t_{0})$ is the creep coefficient[73] relevant for the load and time interval

If in the input window[56] you select the option "Characteristic comb." only short-time deflection are calculated. If, instead, you select "Quasi-permanent comb." long-time deflection are calculated using the age adjusted effective modulus (see AAEM method[118]):

$$E_{c,eff}(\infty,t_0) = E_{cm} / (1 + c.\varphi(\infty,t_0))$$

where

c = aging coefficient: $c(\infty,t_0)$: for $t_0 > 15$ days that value may be assumed equal to 0.8 or 1.0 if you want to adopt the EM method suggested by (7.20) EC2.

The values of $\varphi(\infty,t_0)$ and c must be assigned (changing the default values) in the Materials library[67] of the sections of sub-element[56] (one or more) that compose the beam. The same for the eventual shrinkage coefficient $\varepsilon_{cs}(\infty,t_0)$ to take in account. Program do not use the relation (7.21) in EC2 but the unified theory outlined in AAEM method[118] paragraph.

Serviceability limit value of deflection for beam, slab or cantilever subjected to quasi-permanent loads must not exceed span/250. For the deflections <u>after construction</u>, span/500 is normally an appropriate limit for quasi-permanent loads: in the case of partitions may be appropriate to calculate deflections (to limit at max span/500) as difference between total long-time ($t=\infty$) deflection and the deflection due to self weight + dead load before partition construction (so we have to do two separate calculations).

Load history influences the value of deflections. A typical load history for building could be:

• Application of self weight g_1 at time t_1 (about 10 days)

• Application of remaining dead loads g_2 (including partitions) at time t_2 (about 60 days)

• Application of quasi permanent live load $_{02} \cdot g_3$ at time t_3 (365 or typically ∞)

A simplified method to avoid to use (because the load history) different values for coefficient φ is the simplified method in /8[173]/; this method consists in the following definition of an equivalent creep coefficient φ_{eq} :

$$\varphi_{eq} = [g_1 \varphi(\infty,t_1) + g_2 \varphi(\infty,t_2) + g_3 \varphi(\infty,t_3)] / (g_1 + g_2 + _{02} \cdot g_3)$$

∞ may be any time values in days.

Program do not compare the calculated deflections with their limit values assessed by the user.

Section deformations

Each sub-elements of the beam is divided in segments by means of an assigned (in the input window [56]) length of discretization. The deformation of the section calculated in the middle of each segment is assumed constant for the whole segment. Program calculates two deformations value for each mean section:

1) <u>Instantaneous deformation</u> (short-time effects)

The state I is the uncracked state with unlimited tension strength of concrete with a_1, b_1, c_1 as deformation parameters. We have a state II when under N,Mx force the section cracks assigning 0 value to tension strength of the concrete with a_2, b_2, c_2 as deformation parameters. For eq. (7.18) EC2 the mean deformation of the section at time t_0 is:

$$\chi_{ym} = a_m (t_0) = (1 - \zeta) \cdot a_1 + \zeta \cdot a_2$$
$$\chi_{xm} = b_m (t_0) = (1 - \zeta) \cdot b_1 + \zeta \cdot b_2$$
$$\varepsilon_{0m} = c_m (t_0) = (1 - \zeta) \cdot c_1 + \zeta \cdot c_2$$

where

$$\zeta = 1 - \beta (f_{ctm}/f_{ct,min})^2$$

$\quad \zeta = 0 \quad$ if $f_{ct,min} < f_{ctm}$ the section is not cracked under N, Mx forces

$\quad \sigma_{ct,min} \quad$ is the min concrete tension in state I under N,Mx forces in the uncracked section.

1) <u>Long time deformation</u>

Program calculate the increase of deformations $\Delta a_1, \Delta b_1, \Delta c_1$ at state I due to creep and shrinkage by means of AAEM procedure [118]. So the deformation at time t is:

$$c_{y3} = a_3 = a_1 + \Delta a_1$$
$$c_{x3} = b_3 = b_1 + \Delta b_1$$
$$\varepsilon_{O3} = c_3 = c_1 + \Delta c_1$$

At state II $\Delta a_2, \Delta b_2, \Delta c_2$ are the increase of deformation. Total deformations at time t are then:

$$c_{y4} = a_4 = a_2 + \Delta a_2$$
$$c_{x4} = b_4 = b_2 + \Delta b_2 \qquad\qquad (1)$$
$$\varepsilon_{O4} = c_4 = c_2 + \Delta c_2$$

By means the distribution coefficient ζ we obtain the mean deformations at time t:

$$c_{ym} = a_m(t) = (1 - \zeta) \, a_3 + z \, a_4$$
$$c_{xm} = b_m(t) = (1 - \zeta) \, b_3 + z \, b_4 \qquad\qquad (2)$$
$$\varepsilon_{Om} = c_m(t) = (1 - \zeta) \, c_3 + z \, c_4$$

Deflections in single beam

Beam can be assigned as consisting as one or more sub-elements: each sub-element is characterized by a constant section (as geometry and reinforcement) and a distributed load acting on it. Two sub-element may be different only for different placing of longitudinal bars or for different distributed load. Before of this input it is necessary to input ad save with different names the above sections that define the different sub-elements (even a single). At each sub-element correspond a section that may be predefined or general but always with <u>uniaxial bending forces</u>. Pertinent creep, shrinkage and aging coefficients must be assigned to each section in Materials library[67] and they are taken in account in the deflection calculation of the beam.

The beam can be hyperstatic as belonging to a frame, but solving the frame, for the combination of interest (usually quasi-permanent), we can make the beam isostatic (on two supports) applying the two hyperstatic bending moments at the end. This is a simplification that disregards the true interaction in the time of hyperstatic forces with creep and shrinkage effects: specific studies have shown that the difference in deflection is negligible.

Beam on two supports
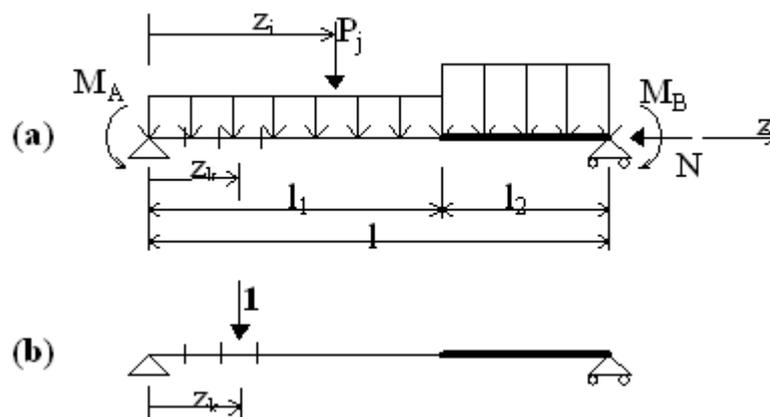
To fix the ideas with an example we show the calculation scheme in the above figure (a) that refers to a beam consisting of two sub-elements whose lengths are indicated as l_1 and l_2. The two sub-element have different sections and acting loads (self-weight shall be included in the distributed load). Bending moments M_A and M_B at the end of the beam may represent hyperstatic action from adjacent structural elements. Concentrated loads P_j may be assigned in any point of the span. A constant axial force N may be present (so we can also study, for example, the shortening of columns under creep effects in height buildings).

In uniaxial bending we denote with $\chi(z)$ the distribution of curvatures along the beam due to the acting loads in the scheme (a) and with M'(z) the bending moment produced by a unit load in the service scheme in (b) figure. The principle of virtual work gives the following deflection formulation for η at the abscissa z of application of unit load:

$$\eta(z) = \int M'(z)\chi(z)\,dz$$

To resolving the integral in numeric form we subdivide the whole length of the beam in N little segments. Each segment has a length Δz_k and abscissa of its mean point equal to z_k. The numerical integration can be express:

$$\eta(z_k) = \sum_{i=1}^{n} M'(z_i)\,\chi(z_i)\,\Delta z_i \qquad (k=1,\ldots,n)$$

If we substitute in the summation short-time values of curvatures χ of the sections we obtain the instantaneous ($t=t_0$) beam deflections. Long-time deflection ($t=\infty$) are obtained with the substitution of long-time values of curvatures.

Cantilever beam
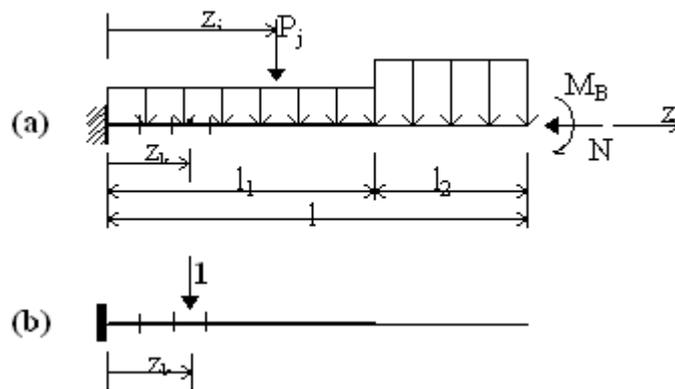
The same type of scheme and relationships has been applied to the cantilever typology of beam (very sensitive to bending deflections).

4   Examples

This section discussed a number of examples performed with the program and saved in "\ Examples" folder of the installation directory.
Almost all are taken from reliable worked examples for which references are given in the bibliography at the end of this user guide.

4.1   Uniaxial Bending Check
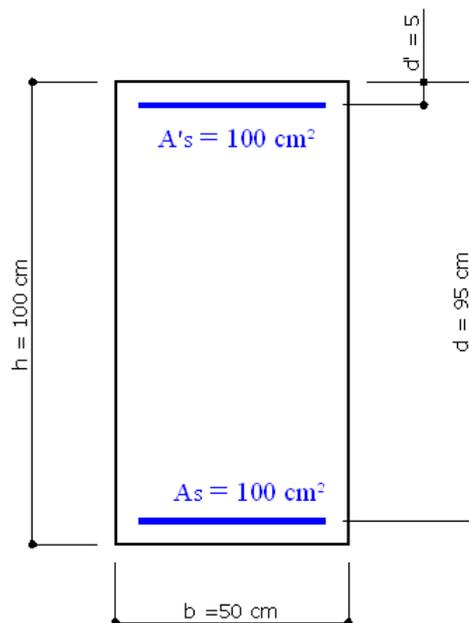
The above cross section of a column refers to Example 6.1 in /4[173]/.
C30/37 is the concrete class and S450C is the steel grade.
The concrete design strength is: $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 30 / 1.5 = 17.0$ Mpa
The steel design yield strength is: $f_{yd} = f_{yk} / \gamma_s = 450/1.15 = 391.3$ Mpa
The problem is to assess uniaxial moments of resistance under the following axial loads: 600 kN; 2000 kN; 5000 kN; 10000 kN.
We want check the same section with the program and compare the results.

INPUT DATA

Once opened a new calculation (by means the command "New" in the menu File) it is important to control if the default Code setting options agree with those in the given example to compare.

In this example we control, in particular, the parameters γ_c, γ_s, α_{cc}. The EC2 recommended value of the last parameter α_{cc} is 1.00 and it is to change in 0.85 as in the example. If a specific National Annex is selected no change is necessary.

The others default code parameters are the same.

The filled General Data⌐9⌐ and the pertinent Section Data windows are:

In the above General Data window we have selected the "Predefined" typology and "Columns" as member characteristics. We left unchanged the default value of Load path (N=cost.) so to calculate bending capacities with assigned values of N taken as constants. The section belongs to a structure not in seismic zone then low ductility class DCL applies (only EC2 rules).

In the Section Data window we left the default type of calculation (Check) and typology (Predefined section for uniaxial forces check). In the three combo boxes we selected the shape (rectangular) and the same classes of the materials provided in the example. After setting the actual sizes of the section (50x100) and the covers (5 cm), we simulated the areas of the two row of longitudinal reinforcement $A_s = A'_s$ = 50 cm² as obtained by 6+6 bars all with the same fictitious diameter of 32.573 mm.

In the above Forces window we assigned, as ULS combinations, the above four axial forces and generic values (300 kNm) for the correspondent moments; this because we want to assess only the bending capacities without compare the actual moments.

RESULTS

Selecting the Checks tab we obtained the below results window, printed results window (command on the local tab bar) and interaction diagram N-Mx (option in the lower part of this window):

General Data | Section Data | Forces | **Checks** | Stirrups-Ductility |

general check: ok

Type of Combination	N.Comb.	Not checked
ULS Capacity combinations	4	
SLS Characteristic combinations	0	
SLS Frequent combinations	0	
SLS Quasi-Permanent combinations	0	

Combination Number 1 ◀ ▶ CHECK COMBIN. OK

ULS Bending-Axial Forces: Safety factor =6.785 [OK if > 1.0]

Sforzi assegnati	Sforzi ultimi (asse x baricentrico)	Deform
N = 600.00 kN	N ult = 600.11 kN	max
Mx = 300.00 kNm	Mx ult = 2035.43 kNm	min
		strain
	Ultim. Curv.=0.000333096 1/cm	max
	Yielding Curv.=0.000033434 1/cm	min
	As tot.= 100.0 cm²	Neutral
	Min edge cover of bars = 3.7 cm	

Run and display

⊙ Stresses and strains of cross-section
○ ULS interaction diagram N-Mx

STRENGTH DOMAIN N-Mx (ULTIMATE LIMIT STATE)

Section name: Example Uniaxial Bending Check.secEC
File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\EXAMPLES\Example Uniaxial Bending Check.secEC

Grid spacing Mx = 325 kNm   Grid spacing N = 986 kN
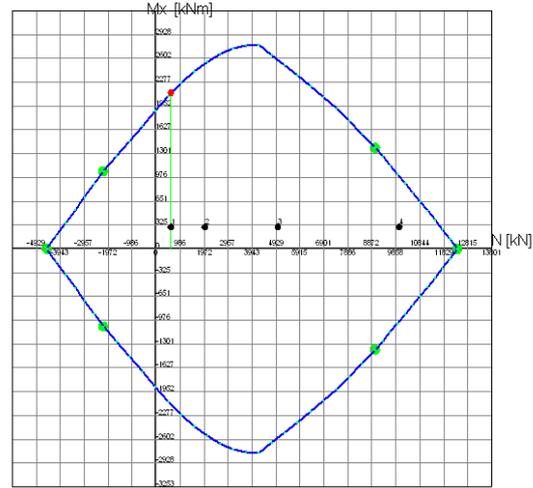
File Modifica Formato

arial narrow   ▼ 9 ▼ | **G** *I* S | ...

ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult,Mxult) to (N,Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [[5.10]EC2]
As Total	Total area of longitudinal bars [cm²]. [Between brackets is shown the min.from eq. (9.12) EC2]]

Comb.N.	Check	N	Mx	N ult	Mx ult	S.F.	Yn	M sn	x/d	D	As Total
1	OK	600.00	300.00	600.11	2035.43	6.785	89.5	1911.53	—	—	100.0 (10.0)
2	OK	2000.00	300.00	1999.98	2518.64	8.395	70.9	2384.26	—	—	100.0 (10.0)
3	OK	5000.00	300.00	5000.16	2582.28	8.608	33.7	0.00	--	—	100.0 (12.8)
4	OK	10000.00	300.00	10000.24	986.17	3.287	-15.4	0.00	--	—	100.0 (25.6)

ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.01078	100.0	0.00183	95.0	-0.02814	5.0
2	0.00350	-0.00165	100.0	0.00290	95.0	-0.00792	5.0
3	0.00350	0.00124	100.0	0.00324	95.0	-0.00151	5.0
4	0.00318	0.00200	100.0	0.00304	95.0	0.00056	5.0

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In the below table the values of ultimate moments M_{Rd} obtained in the aforementioned example are compared with the just reported results Mx ult. The differences are very small despite the different methods of calculation:

N_{Ed} (kN)	M_{Rd} (kN m)	$M_{x\ ult}$ (kN m)
600	2031	2035
2000	2524	2519
5000	2606	2582
10000	1000	986

4.2   Uniaxial Bending Design
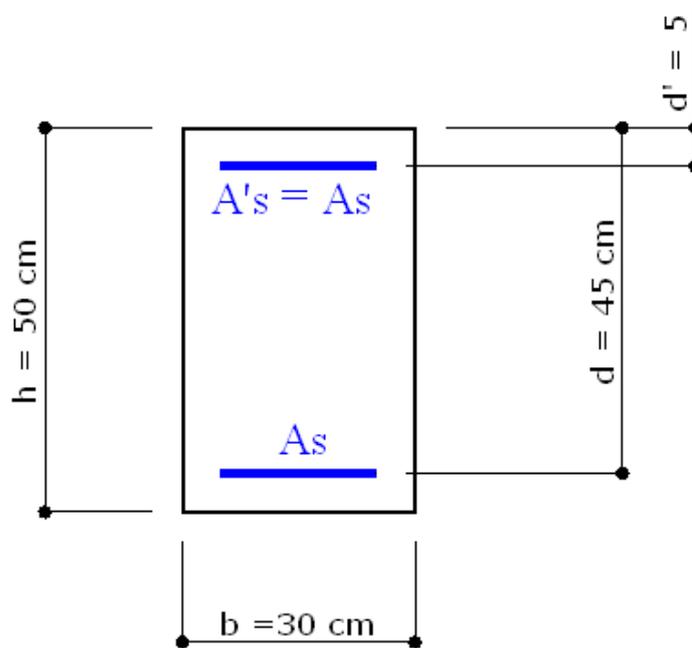
The above cross section of a column refers to Example 5.3 in /6 173/.
C30/37 is the concrete class and S500C is the steel grade.
The concrete design strength is: $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 30 / 1.5 =$ 17.0 Mpa
The steel design yield strength is: $f_{yd} = f_{yk}/ \gamma_s = 450/1.15 = 391.3$ Mpa
The problem is to design the longitudinal reinforcement distributed over only two level of the same areas A_s under the assigned ULS forces:
N=1875 kN; Mx=280 kN.
In addition to the above question we want to design shear reinforcement for a ULS shear design force $V_{Ed} = 300$ kN.

INPUT DATA

Once opened a new calculation (by means the command "New" in the menu File) it is important to control if the default Code setting options agree with those in the given example to compare.
In this example we control, in particular, the parameters γ_c, γ_s, α_{cc}. The EC2 recommended value of the last parameter α_{cc} is 1.00 and it is to change in 0.85 as in the example. If a specific National Annex is selected no change is necessary. In the design context of this example it is important to set the following reinforcement options:

- Diameter of longitudinal reinforcement (below set at 14 and 16 mm) but for predefined section the diameter is an input parameter in design context
- Diameter of stirrups (below set to 8 and 10 mm)
- A'/A below set to 1 because the member of the section is a column
- Min net distance between longitudinal bars set to 2 (this distance must be assigned in according to § 8.2 EC2 and with the diameters just assigned)
- Max distance between longitudinal bar set to 45 cm to avoid the design of bars at intermediate level not provided in the assumed example
- Max personal max value of cot θ = 2.5 as usual in reinforcement shear design

In the below "General Data" windows we selected the typology "Rectangular section of column" suited for the present example; the other data are the same of the previous example.

In the below Section Data window we select:
- The option "Reinforcement Design" to perform, at the same time, bending and shear design or reinforcement (when you reopen this

window the selection is at check option so you must reselect the design option to run in this modality)

- Materials classes for concrete and steel bars (for longitudinal and stirrups reinforcement)

- Dimension of rectangular section

- Type of arrangement of corner bars (one bar for each corner) and bar diameter

General Data | Section Data | Forces | Checks |

Description: [   ]

Typology of the section

- ○ Predefined (uniaxial bendind and shear for beam or column)
- ○ General (biaxial bending and shear)
- ● Rectangular section of column (biaxial bending and shear)
- ○ Wall (not ductile, ductile or large lightly reinforced))
- ○ Polygonal section in torsion
- ○ SLS deflection in single beam (long-term deformations too)

Exposure Class [§4.2 EC2]

[XC2 - Carbonation (long-term water contact/foundations) ▼]

Member characteristics (for detailing design)

[Column (compression member) ▼]

Section position in member
- ○ Section is into a critical region (near joints) of the member
- ○ Section is out of critical regions of the member

Load path for moments of resistance (U.L.S.)

- ● N = const. (constant axial forces)
- ○ M/N = const. (constant eccentricity)

Options for serviceability limit states (S.L.S.)

- ● n method   ☐ Crack width always in cracked state
- ○ AAEM/EM methods ☐ Crack width (effective tension area)

Ductiliy Class (also for detailing)

- ● DCL (low) only EC2 applies
- ○ DCH (height) EC2+EC8 applies
- ○ DCM (middle) EC2+EC8 applies

General Data | **Section Data** | Forces | Checks |

Type of calculation
- ○ Check   ● Reinforcement design
- ○ Stability analysis for one span column (nominal curv. method)

Concrete

Concrete class [C30/37 ▼]

Dimensions of rectangular section (cm)

Side X [30]   Side Y [50]

Reinforcement

Steel grade [B500C ▼]

Corner bars
- ● 1 bar for each corner
- ○ 3 bars for each corner   Diameter [16] mm

Cover (from centroidal axis of bars) [5] cm

Distance betw. bars in the same corner [2] cm

Web bars

N. web bars X sides [0]   Diam. web X [0] mm

N. web bars Y sides [0]   Diam. web Y [0] mm

Stirrups (set Diameter=0 to get automatic design)

Steel grade [B500A ▼]
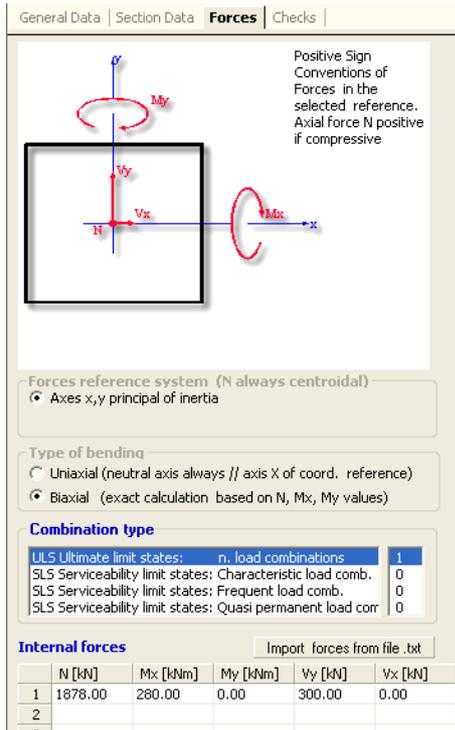
In the above Forces window we assigned, as ULS combinations, the axial and uniaxial bending forces of assumed Example 5.3 in /6⌐173┐/. For the shear design we added the uniaxial shear force Vx = V_{Ed} = 300 kN.

RESULTS
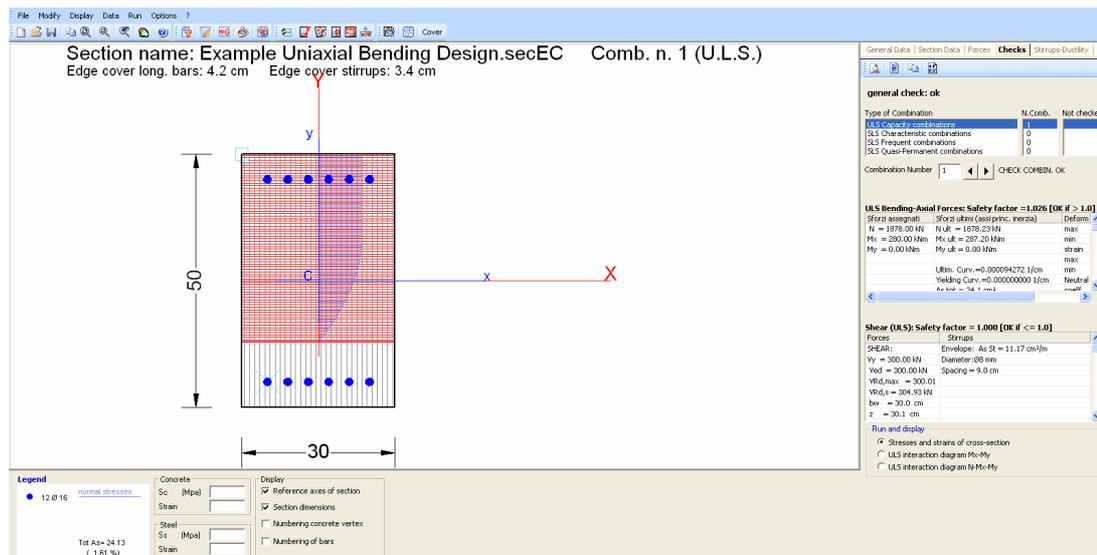
The total area of designed bars (6+6 Φ 16) is 24.13 cm² (as you may see in the legend box). This area is very near to that (23.45 cm²) of the assumed example calculated by means of an abacus.

The bending resistance (287.2 kNm) is slightly greater than the design value (280.0) as you may graphical control in the below interaction diagram (black circle is the design value and the red one is the reistance).

STRENGTH DOMAIN Mx-My (ULTIMATE LIMIT STATE)

Section name: Example Uniaxial Bending Design.secEC

File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\EXAMPLES\Example Uniaxial Bending Design.secEC
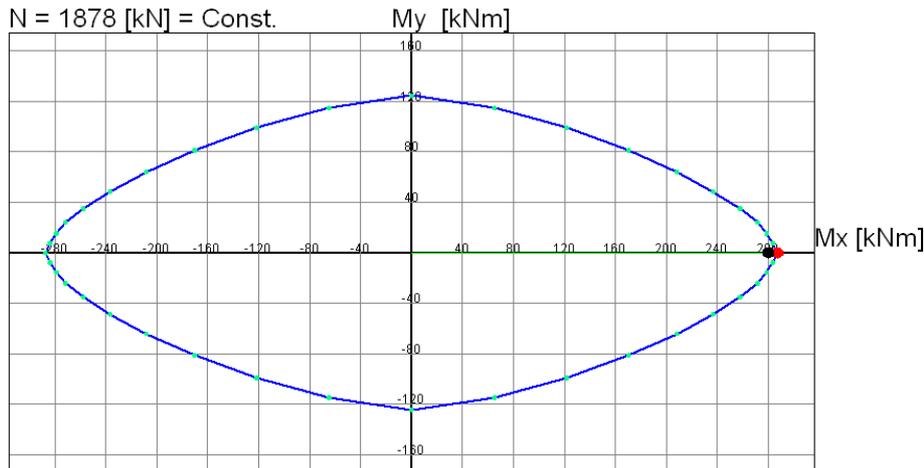
Grid spacing moments Mx, My = 40 kNm

The shear design performed by the program has produced the following two legs stirrups: Φ 8 / 9 cm. The check results are:

ULTIMATE LIMIT STATES - SHEAR CHECKS

Stirrup diameter:	8	mm	> Min diameter for code = 6.0 mm
Stirrups spacing(pitch):	9.0	cm	≤ Max spacing for code = 30.0 cm
Area stirrup/m:	11.2	cm²/m	> Stirrup area strictly necessary for shear = 11.0 cm²/m

Check	Result of check
Ved	Design Shear [kN] = sum of Vx and Vy projected onto the normal to the neutral axis
Vrd,max	Shear resistance [kN] by concrete diagonal struts [[6.9] EC2]
Vrd,s	Shear resistance [kN] by stirrups [[6.8] EC2]
Zm	Main weighted inner lever arm [cm] of strips orthogonal to neutral axis: are not included strips without an extreme compressed. The weights are constituted by the length of lever arm for each strips.
Dm	Main weighted effective depth [cm] of strips orthogonal to neutral axis.
bw	Mean weighted shear width [cm] misurate parallel to the neutral axis = = ratio of area of resistent strips to Zmed value
Theta	Angle [°] of the inclined concrete struts in the web
Ast	Area of hoops+cross-ties strictly necessary for shear force[cm²/m]
A.Eff	Effective Area of hoops+cross-ties in the shear direction in current comb.[cm²/m]

Comb.N.	Check	Ved	Vrd,max	Vrd,s	Zm|Dm	bw	Theta		Ast	A.Eff
1	OK	300.00	300.01	304.93	30.1|45.0	30.0	25.65°	—	11.0	11.2

4.3 Uniaxial_Shear_design
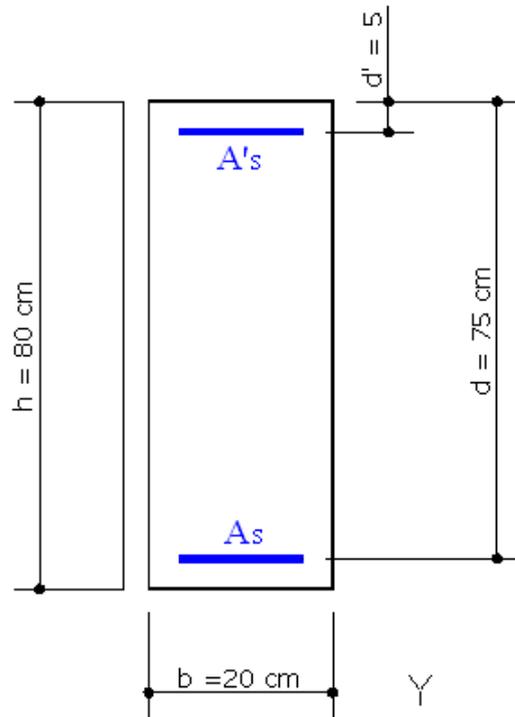
The above cross section of a beam refers to Example 6.4 in /3⌐173⌐/. C30/37 is the concrete class and S450C is the steel grade of stirrups.

The concrete design strength is: $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 30 / 1.5 = 17.0$ Mpa

The steel design yield strength is: $f_{yd} = f_{yk} / \gamma_s = 450/1.15 = 391.3$ Mpa

The coefficient $\nu_1 = 0.616$ (in eq. (6.9) EC2)

Longitudinal reinforcement for bending resistance are not given but it is known only the lever arm of internal force: z = 67.5 cm

The problem is to design stirrups to satisfy a design shear force $V_{Ed} = 600$ kN.

We want to do the same and compare the results.

INPUT DATA

Once opened a new calculation (by means the command "New" in the menu File) it is important to control if the default Code setting options agree with those in the given example to compare.

In this example we control first the parameters γ_c, γ_s, α_{cc}. The EC2 recommended value of the last parameter α_{cc} is 1.00 and it is to change in 0.85 as in the example. If a specific National Annex is selected no change is necessary.

The value $v_1 = 0.616$ is not conform to EC2 recommended value $0.6(1-f_{ck}/250) = 0.528$; then we entered the assigned value 0.616 in the below window.

To obtain the same diameter for stirrups bars we assigned the same diameter 12 mm to the first and the second choice.

In the General Data windows we selected the "predefined" and Beam as Member Characteristic.

In the below Section Data window we select:

- The option "Check" as the longitudinal reinforcement is known
- Materials classes for concrete and steel bars (for longitudinal and stirrups reinforcement)
- Shape (Rectangular) and Dimension B, H of the section
- Cover and data of longitudinal bars: we defined the areas of the two rows of bar such to obtain z=67.5 cm like in the original example.
- To design only shear reinforcement stirrups we set to 0 the diameter of stirrups:

In the Force window we cannot assign only the shear $V_y = V_{Ed} = 600$ kN but also the axial and moment force (not present in the original example).

RESULTS

Below printed results show that the designed stirrups are 2-leg Φ 12 /17.8 cm very near to the -leg Φ 12 /17.0 cm in the original example. The angle θ of inclination of shear struts is 29°.04 (29°.0 is the original result).

SHEAR REINFORCEMENT (FOR THE ENVELOPE OF ALL ASSIGNED COMBINATIONS)

Stirrup diameter: 12   mm
Stirrups spacing (pitch):   17.8   cm   < Max distance for code = 24.0 cm
Number of branches of stirrup: 2
Area stirrups/m: 12.7   cm²/m   > Area stirr. strictly necessary for shear = 12.6cm²/m
Mean distance between branches: 15.8   cm   < Max dist.(9.8 N)EC2 = 56.3 cm

ULTIMATE LIMIT STATE - SHEAR CHECKS

Check   Result of shear/torsion check
Ved   Design applied Shear Vy [kN] (uniaxial force)
Vrd,c   Shear resistance [kN] without shear reinforcement [§6.2.2 EC2]
Vrd,max   Shear resistance [kN] limited by crushing of concrete struts [eq.(6.9)EC2]
Vrd,s   Shear resistance [kN] sustained by yieldin of shear reinforcement [eq.(6.8)EC2]
z   Inner level arm [cm] of resistant cross section (§6.2.3(1) EC2)
bw   Minimum width [cm] of cross section parallel to neutral axis (§6.2.3(1) EC2)
Theta   The angle [°] between the concrete compression struts and the beam axis (§6.2.3(1) EC2)
Ast min   Minimum admissible area of stirrups [cm²/m] [eq.(9.5N)EC2]
Rl | Scp   Reinforcement ratio of tensile long. reinforcement | Normal mean stress (N/Ac) [Mpa], see note 2 of §6.2.3(3)
Acw=1.000   Coeff. taking account of compression stress in eq.(6.9)EC2
Crd,c=0.120   Coeff. for shear resistance without shear reinforcement in eq.(6.2.a)EC2
k1=1.516   Coeff. for shear resistance without shear reinforcement in eq.(6.2.a)EC2
v min =3.580   Coeff. for shear resistance without shear reinforcement in eq.(6.3N)EC2

Comb.N.	Check	Ved	Vrd,c	Vrd,max	Vrd,s	z	bw	Theta	Ast min	Rl | Scp
1	OK	600.00	90.75	600.01	604.49	67.5	20.0	29.04		0.011 | 0.2

4.4 Biaxial Bending Check
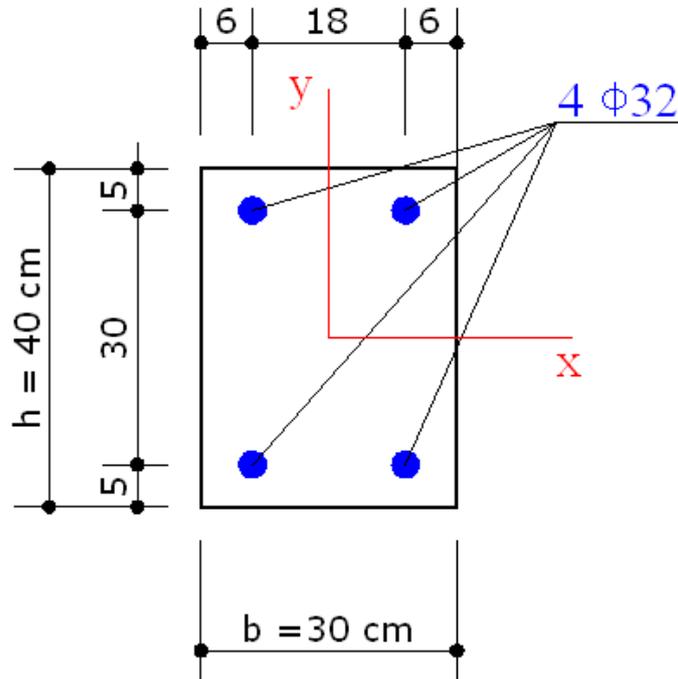
The above cross section of a column refers to Example 5.4 in /6|173|/. C50/60 is the concrete class and S500C is the steel grade.

The concrete design strength is: $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 50 / 1.5 = 28.3$ Mpa

The steel design yield strength is: $f_{yd} = f_{yk} / \gamma_s = 500/1.15 = 434.8$ Mpa

The authors in /6|173|/ claim that the above reinforcement is the exact and rigorous solution to the project under the forces N= 2000 kN, Mx = 234 kNm; My = 100 kNm.

To control that statement we perform a biaxial check of the section with the given reinforcement in order to compare the capacity forces with the design ones.

In addition to the previous question we want to design shear reinforcement for a ULS biaxial shear design force Vx = 420 kN, Vy = 180 kN and compare the results with uniaxial separate checks.

INPUT DATA

Once opened a new calculation (by means the command "New" in the menu File) it is important to control if the default code setting options agree with those in the given example to compare.

In this example we control, in particular, the parameters γ_c, γ_s, α_{cc}. The EC2 recommended value of the last parameter α_{cc} is 1.00 and it is to

change in 0.85 as in the example. If a specific National Annex is selected no change is necessary. We left the values of the shear parameters as recommended in EC2.

In the context of this example, we could possibly modify the following option:

- The diameter of first (8 mm) and second (10 mm) diameter of stirrups
- Max personal max value of cot θ = 2.5 as usual in reinforcement shear design.
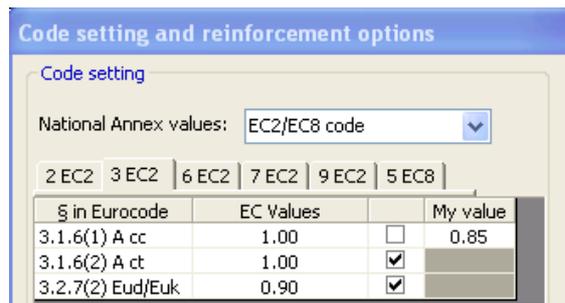
In the below General Data windows we should selected the typology "Rectangular section of column", but this special typology provides the same cover for all the bars. As in this case the covers are different for the two sides, we are forced to select the "General" typology. Then we select "Column" as Member characteristic and DCL as Ductility class (only EC2 applies).

In the below Section Data window we select or assign:

- The option "Check" to perform bending check and "0" as diameter of stirrups in order to perform the design of stirrups shear in biaxial shear (after calculation when you reopen this window the selection is at check option so you must reselect the design option to run in this modality)
- N. of concrete region = 1 for the simple section at study which has a shape "Polygonal full"
- Coordinates of the four vertices of the concrete section with manual input (for complex sections you can import geometry and bars from a dxf file)
- As the different covers we are forced to select "No bars" option and manually enter the coordinates of the centroid of the four bars
- Materials classes for concrete and steel bars (for longitudinal and stirrups reinforcement)

In the Force window we assign the design forces described above for axial, biaxial bending and bending shear.

To compare the biaxial results with uniaxial ones we assign two additional combination (see below window) with separate uniaxial bending moment and shear forces (leaving unchanged the axia force).

RESULTS

The numerical output results for biaxial bending (below reported) show a very slight difference the original example forces and the resulting resistances calculated by the program:

$Mx_{Ed} = 234$ kNm; $Mx_{Rd} = 233.79$ kNm

$My_{Ed} = 100$ kNm; $Mx_{Rd} = 99.90$ kNm

ULTIMATE LIMIT STATES - N-MX-MY CAPACITY CHECKS

Check	Result of check
N	Design axial force [kN] applied at the centroid of concrete section (+ if compressive)
Mx	Design bending moment [kNm] around x axis principal of inerzia
My	Design bending moment [kNm] around y axis principal of inerzia
N ult	Axial force capacity [kN] (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis principal of inertia
My ult	Bending moment capacity [kNm] around y axis principal of inertia
S.F.	Safety Factor = vectorial ratio of (N ult,Mxult,My ult) to (N,Mx,My). Check OK if ratio >=1.00

Comb.N.	Check	N	Mx	My	N ult	Mx ult	My ult	S.F.
1	NotOK	2000.00	234.00	100.00	2000.16	233.79	99.90	0.999
2	OK	2000.00	234.00	0.00	2000.22	335.24	0.00	1.433
3	OK	2000.00	0.00	100.00	2000.00	0.00	219.72	2.197

1) Vectorial bending moment > Resistance bending moment

ULTIMATE LIMIT STATE - BENDING AND AXIAL FORCE - STRAIN VALUES

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Xc max	X-coordinate [cm] in the concrete point in wich is ec max
Yc max	Y-coordinate [cm] in the concrete point in wich is ec max
es max	Max strain in steel bars (+ if compressive)
Xs max	X-coordinate [cm] of bar in wich is es max
Ys max	Y-coordinate [cm] of bar in wich is es max
es min	Min strain in steel bars (+ if compressive)
Xs min	X-coordinate [cm] of bar in wich is es min
Ys min	Y-coordinate [cm] of bar in wich is es min

Comb.N.	ec max	ec*	Xc max	Yc max	es max	Xs max	Ys max	es min	Xs min	Ys min
1	0.00350	0.00104	15.0	40.0	0.00260	9.0	35.0	-0.00134	-9.0	5.0
2	0.00350	0.00112	-15.0	40.0	0.00281	-9.0	35.0	-0.00135	-9.0	5.0
3	0.00350	0.00102	15.0	40.0	0.00234	9.0	35.0	-0.00113	-9.0	5.0

The numerical output for shear reinforcement (below reported) show that for the biaxial design shear forces is strictly necessary a two-leg hoop $\Phi 10$ / 4cm i.e. $A_{sw} = 38.5$ cm²/m.while for separate uniaxial shear checks are strictly necessary $A_{sw} = 20.5$ cm²/m for shear in y direction and $A_{sw} = 9.3$ cm²/m for shear in x direction. This huge difference demonstrates the inadequacy of the calculation of the double uniaxial shear instead of the biaxial one.

ULTIMATE LIMIT STATES - SHEAR CHECKS

Stirrup diameter:	10	mm	> Min diameter for code = 8.0 mm		
Stirrups spacing (pitch):	4.0	cm	≤ Max spacing for code = 18.0 cm		
Area stirrup/m:	39.3	cm²/m	> Stirrup area strictly necessary for shear = 38.5cm²/m		

Check	Result of check
Ved	Design Shear [kN] = sum of Vx and Vy projected onto the normal to the neutral axis
Vrd,max	Shear resistance [kN] by concrete diagonal struts [(6.9) EC2]
Vrd,s	Shear resistance [kN] by stirrups [(6.8) EC2]
Zm	Main weighted inner lever arm [cm] of strips orthogonal to neutral axis:
	are not included strips without an extreme compressed.
	The weights are constituted by the length of lever arm for each strips.
Dm	Main weighted effective depth [cm] of strips orthogonal to neutral axis.
bw	Mean weighted shear width [cm] misurate parallel to the neutral axis =
	= ratio of area of resistent strips to Zmed value
Theta	Angle [°] of the inclined concrete struts in the web
Ast	Area of hoops+cross-ties strictly necessary for shear force[cm²/m]
A.Eff	Effective Area of hoops+cross-ties in the shear direction in current comb.[cm²/m]

Comb.N.	Check	Ved	Vrd,max	Vrd,s	Zm|Dm	bw	Theta		Ast	A.Eff
1	OK	423.72	423.73	432.31	20.6|27.7	30.9	39.13°	—	38.5	39.3
2	OK	420.00	420.01	803.81	24.9|35.0	30.0	27.88°	—	20.5	39.3
3	OK	180.00	332.86	757.39	17.7|25.0	40.0	21.80°	—	9.3	39.3

4.5   Ductility check
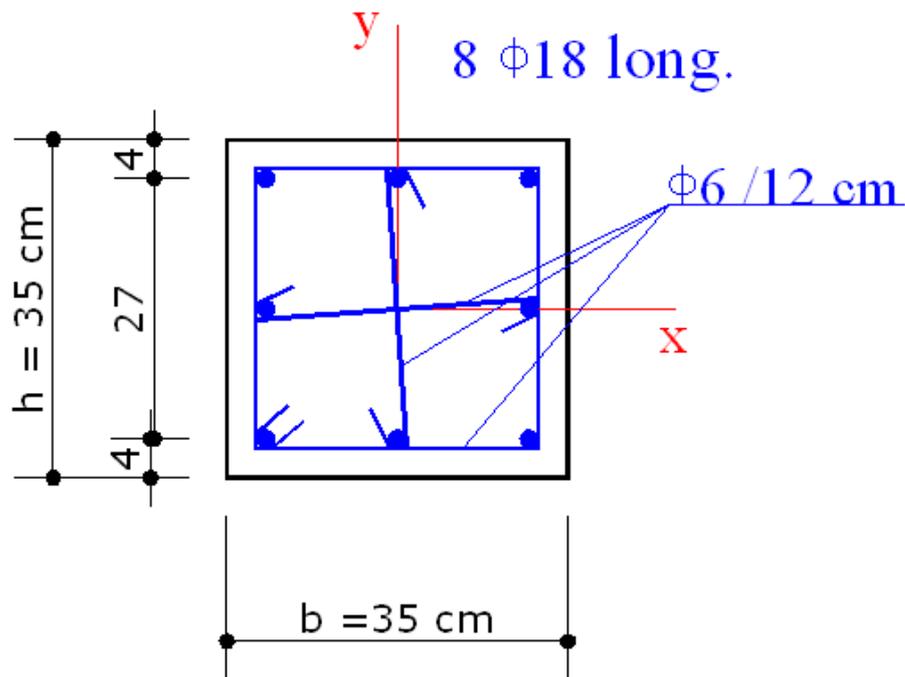
The above cross section of a column is within the critical region at the base C30/37 of a structure designed in DCM ductility class.

C30/37 is the concrete class and B450C is the steel grade.

We want to perform a verification of resistance and a verification of ductility with reference both uniaxial than biaxial forces. The uniaxial acting forces in y direction are (first combination):

N = 400 kN; Mx= 100 kNm; My = 0; Vy = 150 kN; Vx = 0

The biaxial forces (second combination) Mx,My, Vy,Vx have the same vectorial intensity but an inclination of 45° with respect x axis:

N = 400 kN; Mx = 70.71; My = 70.71; Vy = 106 kN; Vx = 106 kN.

The curvature ductility factor (§ 5.2.3.4(3) EC8) is μ_Φ = 6.8 for all seismic directions.

INPUT DATA

The concrete design strength is: $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$ = 1.00 · 50 / 1.5 = 20.0 Mpa

The steel design yield strength is: $f_{yd} = f_{yk} / \gamma_s$ = 450/1.15 = 391.3 Mpa

In this example we first control, in particular, the parameters γ_c, γ_s, α_{cc}, cot θ in Code and reinforcement option 70 window.

In the below General Data windows we should selected the typology "Rectangular section of column". We select DCM as Ductility class and set the above curvature ductility factor (6.8).

In the below Section Data window we select and assign:

- The option "Check" to perform resistance check and "6" as diameter of stirrups in order to perform the check of stirrups for shear resistance check and ductility check.

- Materials classes for concrete and steel bars (for longitudinal and stirrups reinforcement)

- Dimension and cover of the rectangular concrete section and the bar reinforcement provision.

- Stirrups diameter and pitch: Note that in this first input it is not possible to assign specific cross-tie: it will be possible only after a first run in the Stirrups-Ductility 43 window.

The report for moment forces capacity check is:

ULTIMATE LIMT STATES - N-MX-MY CAPACITY CHECKS

Check		Result of check
N		Design axial force [kN] applied at the centroid of concrete section (+ if compressive)
Mx		Design bending moment [kNm] around x axis principal of inerzia
My		Design bending moment [kNm] around y axis principal of inerzia
N ult		Axial force capacity [kN] (+ if compressive)
Mx ult		Bending moment capacity [kNm] around x axis principal of inertia
My ult		Bending moment capacity [kNm] around y axis principal of inertia
S.F.		Safety Factor = vectorial ratio of (N ult,Mx ult,My ult) to (N,Mx,My). Check OK if ratio >=1.00

Comb.N.	Check	N	Mx	My	N ult	Mx ult	My ult	S.F.
1	OK	400.00	100.00	0.00	400.20	159.16	0.00	1.592
2	OK	400.00	70.71	70.71	400.16	99.67	99.67	1.410

ULTIMATE LIMIT STATE - BENDING AND AXIAL FORCE - STRAIN VALUES

ec max		Ultimate compressive strain in concrete
ec*		Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Xc max		X-coordinate [cm] in the concrete point in wich is ec max
Yc max		Y-coordinate [cm] in the concrete point in wich is ec max
es max		Max strain in steel bars (+ if compressive)
Xs max		X-coordinate [cm] of bar in wich is es max
Ys max		Y-coordinate [cm] of bar in wich is es max
es min		Min strain in steel bars (+ if compressive)
Xs min		X-coordinate [cm] of bar in wich is es min
Ys min		Y-coordinate [cm] of bar in wich is es min

Comb.N.	ec max	ec*	Xc max	Yc max	es max	Xs max	Ys max	es min	Xs min	Ys min
1	0.00350	-0.00147	-17.5	17.5	0.00218	-13.5	13.5	-0.00676	-13.5	-13.5
2	0.00350	-0.00009	17.5	17.5	0.00254	13.5	13.5	-0.00392	-13.5	-13.5

ULTIMATE LIMIT STATE - POSITION OF NEUTRAL AXIS FOR EACH COMBINATION

a, b, c		Coeff. a, b, c in neutral axis equation: aX+bY+c=0 reference X,Y,O
x/d		Ratio of the depth of neutral axis to the effective depth of the section
D		Ratio of redistributed moment to the elastic moment in continuous beams [eq.(5.10)EC2]

Comb.N.	a	b	c	x/d	D
1	0.000000000	0.000331011	-0.002292685	—	—
2	0.000119751	0.000119751	-0.000691286	—	—

The safety factor for biaxial second combination is lesser than the first uniaxial combination as you also see by the interaction diagram:

STRENGTH DOMAIN Mx-My (ULTIMATE LIMIT STATE)

Section name: Example_Ductility.secEC

File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\EXAMPLES\Example_Ductility.secEC

N = 400 [kN] = Const.

Grid spacing moments Mx, My = 40 kNm
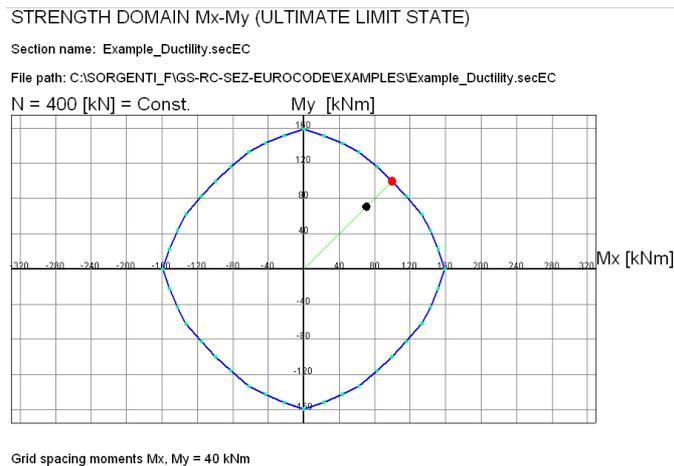
The capacity and ductility shear check report is:

ULTIMATE LIMIT STATE - SHEAR CHECK - LOCAL DUCTILITY CONTROL

Stirrup diameter:	6	mm	> Min diameter for code = 6.0 mm
N.Cross-Ties / Diameter :	2/6	mm	
Stirrups (& cross-ties) spacing :	12.0	cm	< Max spacing for code = 12.1 cm
Area stirrup/m:	7.1	cm²/m	> Stirrup area strictly necessary for shear = 6.1cm²/m

Check	Result of check
Ved	Design Shear [kN] = sum of Vx and Vy projected onto the normal to the neutral axis
Vrd,max	Shear resistance [kN] by concrete diagonal struts [(6.9) EC2]
Vrd,s	Shear resistance [kN] by stirrups [(6.8) EC2]
Zm	Main weighted inner lever arm [cm] of strips orthogonal to neutral axis:
	are not included strips without an extreme compressed.
	The weights are constituted by the length of lever arm for each strips.
Dm	Main weighted effective depth [cm] of strips orthogonal to neutral axis.
bw	Mean weighted shear width [cm] misurate parallel to the neutral axis =
	= ratio of area of resistent strips to Zmed value
Theta	Angle [°] of the inclined concrete struts in the web
Ast	Area of hoops+cross-ties strictly necessary for shear force[cm²/m]
A.Eff	Effective Area of hoops+cross-ties in the shear direction in current comb. [cm²/m]
	Between brackets is shown the value related to the cross-ties only
	Cross-ties area is reduced by factor L/d_max (L=ties length projected on the shear
	direction; d_max= max effective section width on the shear direction.
Alpha	= 0.455 Confinement effectiveness factor = AlphaN*AlphaS [(5.15) EC8]
AlphaN	= 0.719 [from eq.(5.16) EC8]
AlphaS	= 0.633 [from eq.(5.17) EC8]
OMst	Mechanical volumetric ratio of confining hoop and cross-ties in the shear direction [(5.15)EC8].
	(Between brackets is shown: OMst min value = ratio of 1/2 second member of (5.15) value to the confinement effectiveness factor)

Comb.N.	Check	Ved	Vrd,max	Vrd,s	Zm|Dm	bw	Theta		Ast	A.Eff	OMst
1	OK	150.00	323.10	185.12	26.8|31.0	35.0	21.80°	—	5.7	7.1(2.4)	0.047(0.047)
2	OK	149.91	295.27	173.59	25.1|29.8	34.1	21.80°	—	6.1	7.1(2.4)	0.049(0.047)

Shear resistance of concrete struts and of stirrups are very near both the first(uniaxial) and the second (biaxial) combination. The ductility checks performed with the (5.15)EC8 equation are positive for both combinations.

The direct moment-curvature diagram for the biaxial combination leads to a negative check for the ductility factor $\mu_\Phi = 4.6 < 6.8$ so showing the limit of the (5.15) check in biaxial assessment:
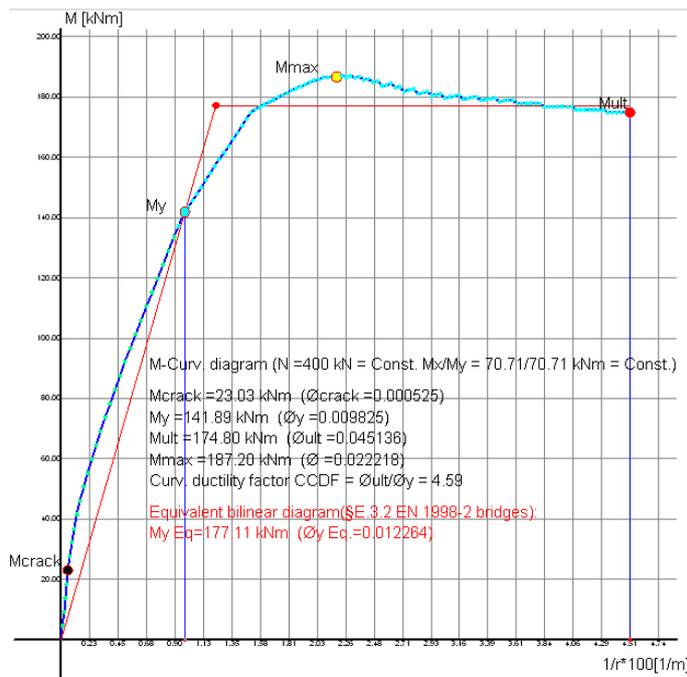
4.6 Crack width check
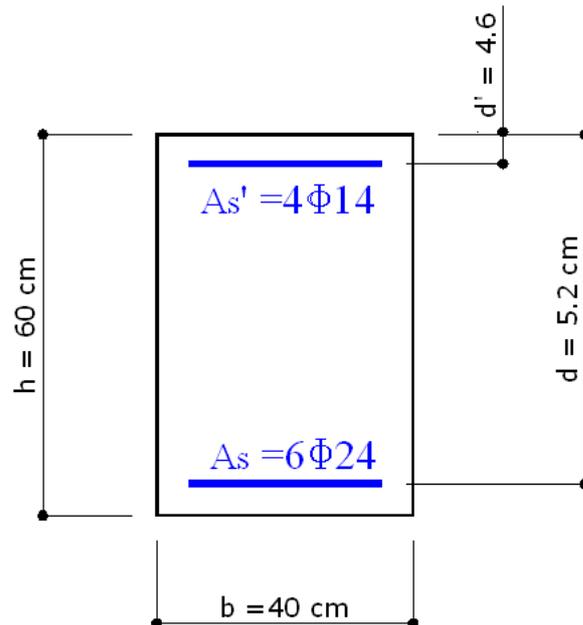
The above cross section of a beam refers to Example 7.3 in /4[173]/. C30/37 is the concrete class and S450C is the steel grade.

The concrete design strength is: $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 30 / 1.5 = 17.0$ Mpa

The steel design yield strength is: $f_{yd} = f_{yk} / \gamma_s = 450/1.15 = 391.3$ Mpa

The problem is to assess crack width under the following bending moment Mx = 300 kNm (SLS frequent combination) assuming $\alpha_e = n = E_s/E_c = 15$.

We want check the same section with the program and compare the results.

INPUT DATA

Once opened a new calculation (by means the command "New" in the menu File) it is important to control if the default Code setting options agree with those in the given example to compare.

In Materials Library we must control if the parameter $\alpha_e = n = E_s/E_c$ is set to 15 (see column n coeff.) for the concrete class C30/37:

The filled General Data ⁹ and the pertinent Section Data windows are:

In General Data window the section has been entered as a Predefined typology; as Member Characteristic we selected "beam"; as Options for SLS we selected "n method".
In Section Data window we entered the exact data of the of the aforementioned comparative example.
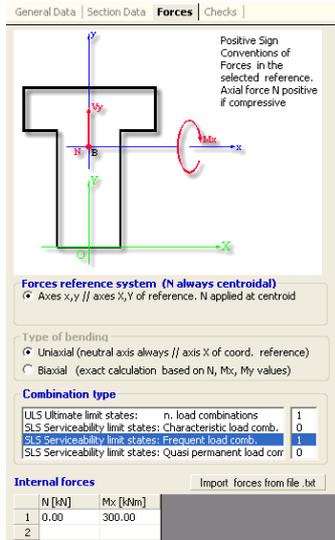
In the above Forces window we assigned, we selected the SLS Frequent load comb. row and then entered in the grid the values 300 kNm for the moment Mx.

Note that we have to assign, for any section to calculate, at least one Ultimate Load combination (in this example we assigned Mx=450 kNm).

RESULTS

Selecting the Checks tab we obtained the below results window and the printed results window (which command is on the local tab bar) with reference to SLS load combination of our interest. In particular the crack width w_k is 0.184 mm as in the reference example.
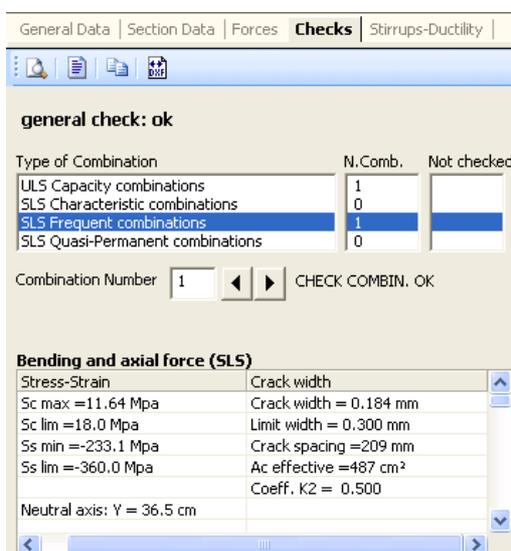
SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm²] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm²] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2]
	(D bars = 0 indicates spacing > 5(c+Ø/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	11.64	60.0	0.00	36.5	-233.1	55.4	12.2	487	27.1	5.9

SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration od the load [see eq.(7.9) EC2]
k2	= 0.5 for bending; =(e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ø | c	Equivalent bar diameter [mm] cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)]
	Between brackets: Minimum value of eq.(7.9) = 0.6 Smax/Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ø | c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00136	0	0.500	24|4.0	0.056	0.000879 (0.00070)	209	0.184 (0.30)	106.86

In the above output are printed also the stress and all intermediate values calculated by program (for example the effective tension area of concrete Ac = 487 cm²).

The program perform also the check of minimum reinforcement areas (§ 7.3.2):

SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm²] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm²] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	0.86	0.37	1100	67.93	2.83	1.50	---	27.1	2.3

If we remake the input of the same section as a general section we can note a slight difference between the crack width w_k.(w_k =0.183 mm instead of 0.184). This is due to the different way to assess the tension

effective concrete area A_c (see Limitation of crack widths[124] topic). The below graph showed by program for SLS combinations make in evidence with blue points the effective tension area calculated as circles the diameter of which is equal to 7 diameters of the corresponding tensile bars (the circles are limited by the rules laid down in §7.3.2(3) EC2).

Section name: Example Crack Width Biaxial.secEC   Comb. n. 1 (S.L.S. freq.)
Edge cover long. bars: 3.8 cm   Edge cover stirrups: 3.2 cm
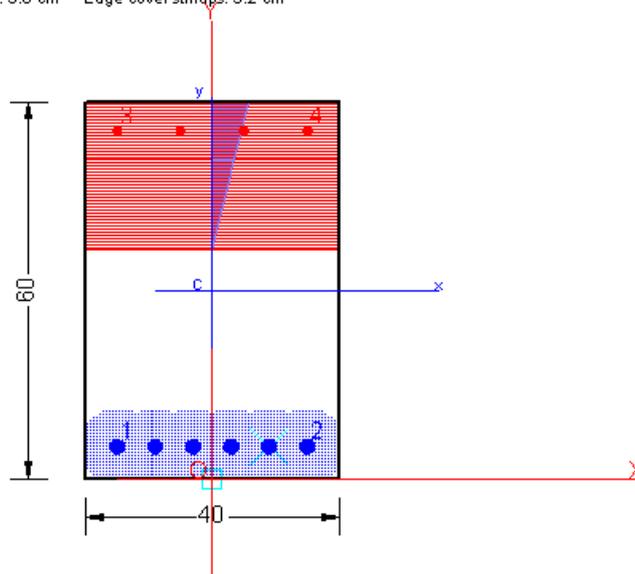
4.7   Deflection check
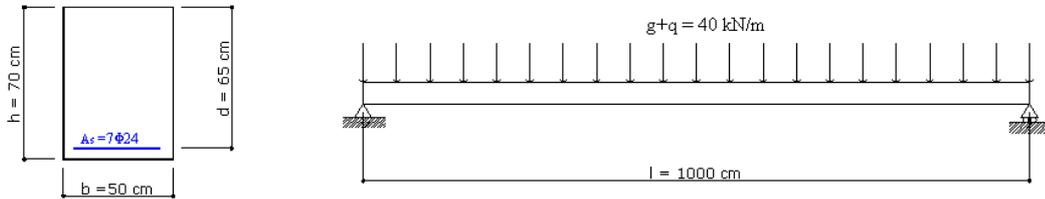
The above beam refers to Example 7.6 in /4[173]/ that concerns the evaluation of the vertical displacement in the mid-span of the beam with the constant transversal section depicted in the same figure. This problem is resolved firstly in a cumulative way stating $\alpha_e = E_s/E_c = 15$.

C30/37 is the concrete class and S450C is the steel grade.

The use of the α_e parameter is equivalent to use the following effective creep coefficient [see eq.(7.20) EC2]:

$$E_{c,eff} = E_{cm}/(1 + \varphi(\infty, t_0) = 15 / E_s = 15/2000000 = 1/133333$$

from which for the assigned concrete $E_{cm} = 328370$ then the equivalent creep coefficient φ to use in the program is:

$$\varphi = \varphi(\infty, t_0) = E_{cm}/133333) - 1 = 1.463$$

Before to calculate the beam deflection we have to generate the constant predefined transversal section which file that we named "Transversal_1.secEC". In this section we have select the above materials and set in the C30/37 concrete row of Materials Library[67]: creep coefficient $\varphi = 1.463$, aging coefficient $\chi = 1$ (EM method) and shrinkage coefficient = .000000001 i.e. null value:

Material Library

Concrete

N°	Class	Values based on fck	Ecm	fcm	fck	fcd	fctd	fctm	Creep coefficient	Shinkage coefficient	Aging coefficient	n coeff.
1	C20/25	✔	29962	28.0	20.0	11.3	1.0	2.2	2.5	.00025	0.8	
2	C25/30	✔	31476	33.0	25.0	14.2	1.2	2.6	2.5	.00025	0.8	
3	C28/35	✔	32308	36.0	28.0	15.9	1.3	2.8	2.5	.00025	0.8	
4	C30/37	✔	32837	38.0	30.0	17.0	1.4	2.9	1.463	0.0000001	1	

Reinforcing Steel

The input windows of the section "Transversal_1.secEC" are:

Assigned <u>Stirrups data and Forces are not used in the deflection calculation</u> but their definition have the sole purpose to complete input and so save the file section.

At this point, in a new and separate calculation, we made the following input of the "Single beam deflections" typology:

Firstly we selected the directory were saved the transversal section named "Transversal_1.secEC". Then the type of span (Beam on two supports" and the following numerical data:

- N. sub-elements of beam = 1 (in this case we have a constant section over the entire span of the beam)
- Mean length of discretization of the beam = 20 cm
- Moments at the end supports (both zero in this case)

In the Data of sub-Elements of beam (there is only one sub-element) we have to select (in the drop-down list) the name of the above section, the length of the sub-element (in this case coincides with the length of span = 1000 cm), the assigned distributed load Py= 40 kN/m. There is no concentrated load over the span.

Immediate following results are furnished by the program:
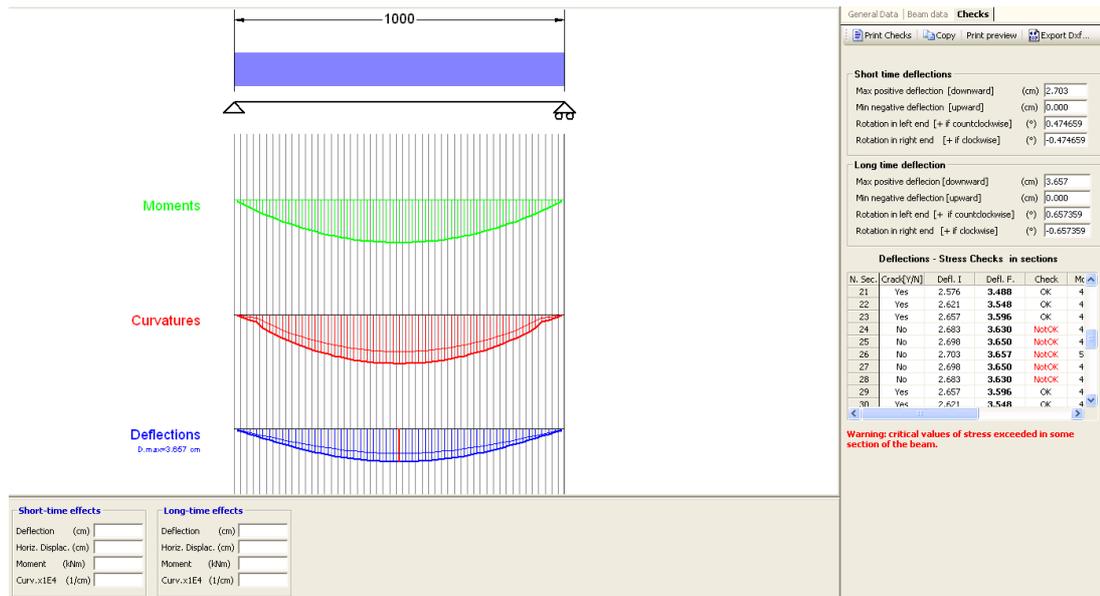
In the check window we can see the max positive deflection for short time effects (2.703 cm > 2.164 cm of the original example) and for long time effects (3.657 cm > 3.571 cm of the original example). The slight difference is due to the simplified method performed in in /4 173/.

Note the warning relating the max stress check calculated with the AAEM method. Maximum value of concrete compressive stress is 13.62 Mpa > 13.50 Mpa critical concrete compressive stress. Performing the stress with n simplified method (α_e = 15) max stress results, instead, equal to 12.94 Mpa < 13.50.

4.8   Instability Check
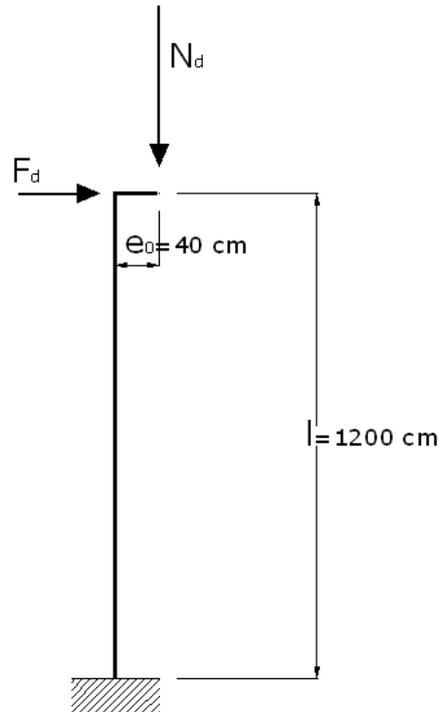
Section name: Example_Instability.secEC

We want to assess the ultimate limit state of instability for the above isolated cantilever column only for X direction as in the example reported in /7/.

In the above Section Data window are assigned (no data for Y direction):
- Concrete class: C40/50
- Steel: B450C

- Dimensions (Side X, Side Y) of the rectangular section of the column: 70x70

- The longitudinal bars 10+10Φ24 are assigned as 1 bar Φ24 /corner and n. 8 web bar Φ24 along Y direction.

- Design constant axial force: 1000 kN

- First order moment $MX_{0d} = N_d \cdot e_0 + F_d \cdot I =$ $1000 \cdot 0.40 + 40 \cdot 12.00$ = 880.00 kNm

- Eccentricity eX for geometric imperfections: 0.00 cm (already included in e_0)

- Effective length $l_0X = 1200 \cdot 2 = 2400$ cm according to fig. 5.7 g) in §5.8.3.1

If we check the option "with uniaxial eq. (5.34)" the output is:

COLUMN STABILITY WITH NOMINAL CURVATURE METHOD [§5.8.8EC2]

INTERNAL FORCES AND RESULTS

Constant design axial load Nd:	1000.00	kN
Bending first order moment around Y axis MoY':	880.00	kNm
Effective lenght of column in X direction (§5.8.3.2) loX:	2400.0	cm
Slendemess ratio with respect to X dir. LX = loX/iX:	118.8	
Min Slendemess ratio (eq.5.13) Lmin:	39.8	
Eccentricity of Nd by imperfections in X direction eiX:	0.00	cm
Total first order moment around Y: MoY=MoY'+ Nd*eiX	**880.00**	**kNm**
Curvature correction factor dependent on Nd [eq.(5.36)] Kr:	1.00	
Curvature correction factor dependent on creep X dir.[eq.(5.37)] Kf:	1.00	

RESULTS IN X DIRECTION

Effective section width in X direction d:	65.0	cm
Curvature in X direction by eq.(5.34) 1/rX:	0.000066890	1/cm
Max top column X displacement [eq.(5.33)] e2:	38.53	cm
2nd order moment around Y axis [eq.(5.33)] M2Y=Nd*e2:	385.28	kNm
Total design moment around Y axis [eq.(5.31)] MdY = MoY+M2Y:	**1265.28**	**kNm**
Moment of resistance around Y axis MRY:	**-1361.07**	**kNm**
COLUMN STABILITY IN X DIR. OK: MRY >MdY		

The second order moment 385 kN is very near to 390 kN obtained in /7/.

If we check the option "with biaxial moment-curvature diagram" the output is:

COLUMN STABILITY WITH NOMINAL CURVATURE METHOD [§5.8.8EC2]
[Curvature 1/r derived from biaxial Mom-Curv. diagram (see "Model Column" method)]

INTERNAL FORCES AND RESULTS

Constant design axial load Nd:	1000.00	kN
Effective creep ratio [according to eq.(5.19)] Kf:	2.50	
Bending first order moment around Y axis MoY':	880.00	kNm
Effective lenght of column in X direction (§5.8.3.2) loX:	2400.0	cm
Slenderness ratio with respect to X dir. LX = loX/iX:	118.8	
Min Slenderness ratio (eq.5.13) Lmin:	39.8	
Eccentricity of Nd by imperfections in X direction eiX:	0.00	cm
Total first order moment around Y: MoY=MoY'+ Nd*eiX	**880.00**	**kNm**
Vectorial total first order moment Md:	**880.00**	**kNm**
Curvature correction factor for creep (eq.(5.37)) Kf:	1.00	
Vectorial equilibrium curvature 1/r:	0.000048547	1/cm
2nd order vectorial moment at the corresponding 1/r curv. M2:	279.63	kNm
Total vectorial design moment [eq.(5.31)] Md = Mod+M2:	**1159.63**	**kNm**
Vectorial moment at the corresp. 1/r curvature MRd:	**1287.23**	**kNm**
COLUMN STABILITY OK: MRd >Md		
Max top column vectorial displacement:	27.96	cm
Max concrete strain at said 1/r curvature:	0.0012	
Min steel strain at said 1/r curvature:	-0.0020	

Last results are less conservative but near to the previous. The "model column" diagram is:

MOMENTS-CURVATURES DIAGRAM (Nd = 1000 daN)

Section name: Example_Instability.secEC

File path: C:\SORGENTI_F\GS-RC-SEZ-EUROCODE\EXAMPLES\Example_Instability.secEC

M 1^ord. available=1007.60 kNm

M 2^ord. unavailable=279.63 kNm

Moments grid spacing = 100 kNm   Curvatures spacing = 0.000007238 1/cm
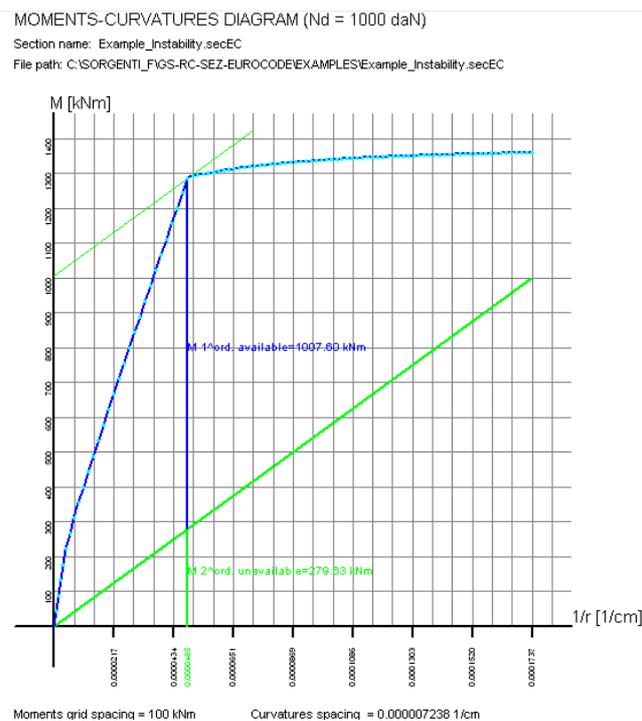
4.9 Pier check
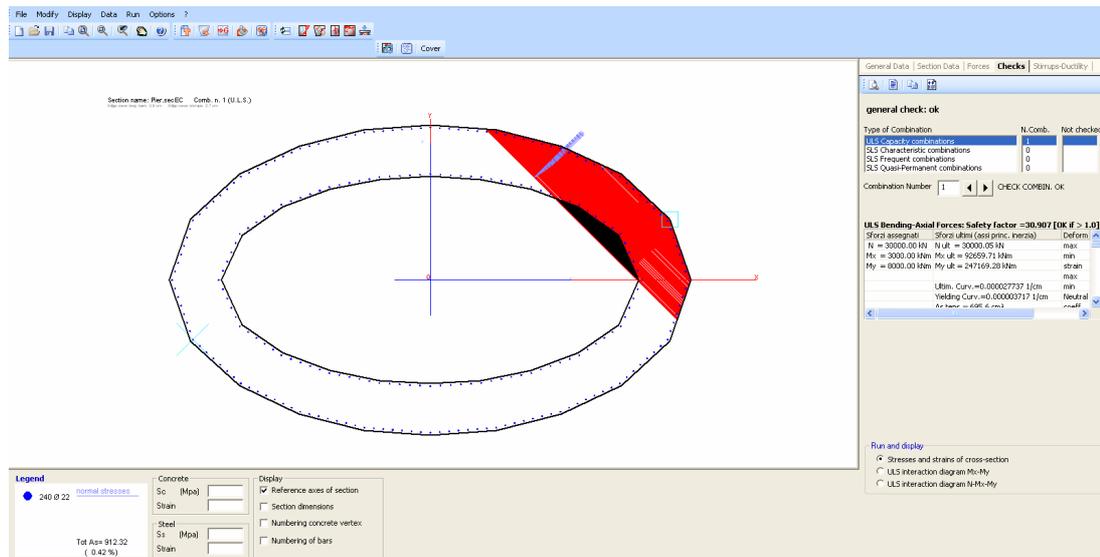
Despite the simple cross sections studied in the previous examples, program is able to assess complex general sections as that shown in the above figure.

Such section refers to an hollow rc pier and its geometry for concrete and reinforcement has been imported from a dxf file, which constitutes the best way to input complex sections.

5   References

/1/   EN 1992-1-1 2005 EUROCODE 2: Design of concrete structures.

/2/   EN 1998-1 EUROCODE 8

/3/   A. Ghali, R. Favre: Concrete structures: Stresses and deformations. Chapman and Hall, 1986

/4/   EUROCODE 2 - WORKED EXAMPLES: European Concrete Platform ASBL, May 2008.

/5/   Progettazione di strutture in calcestruzzo armato - Vol. I - AICAP

/6/   A.W.BEEBY and R.S.NARAYANAN: Designer' Guide to Eurocode 2: Concrete structures. Thomas Telford Limited 2005

/7/   AICAP: GUIDA ALL'USO dell'EUROCODICE- Vol I , Edizione Publicemento

/8/   EUROCODE 2 - COMMENTARY: European Concrete Platform ASBL, June 2008.

/9/   R. Tritto: Un procedimento numerico per la verifica a taglio obliquo nei pilastri in c.a. Rivista "Ingegneria Sismica" n.2/2009,Patron Editore Bologna

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