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USE OF THE PUSHOVER METHOD FOR THE SEISMIC ANALYSIS OF MIXED FRAME-WALL STRUCTURES

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Straus7®
Lineare e Non Lineare

EasyOVER®

INTRODUCTION

REQUIREMENTS

The study of the seismic behavior of buildings with a mixed frame-wall structure places the designer in front of important conceptual problems, such as the verification of the connections between the different construction elements, as well as the distribution of horizontal actions among the resisting walls and the torsional deformability due to the eccentricity between center of stiffness and center of mass.

INTRODUCTION

REQUIREMENTS

The proposed solution consists in implementing the suite of calculation (Straus7) with an Add-on that executes nonlinear static analysis (Pushover analysis) for reinforced concrete buildings, with the following features:

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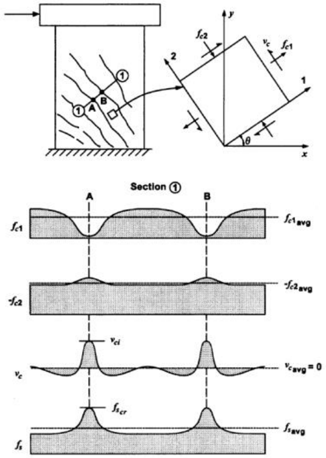
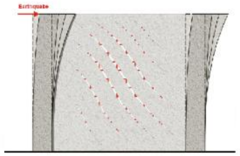
The proposed solution consists in implementing the suite of calculation (Straus7) with an Add-on that executes nonlinear static analysis (Pushover analysis) for reinforced concrete buildings, with the following features:

- Calculation of nonlinear reinforced concrete slabs and walls modeled with Plate/Shell elements according to the Modified Compression Field Theory (MCFT) for slab elements and the Disturbed Stress Field Model (DSFM) for wall elements (models developed by Prof. Frank Vecchio and Michael Collins, University of Toronto).





$$[D_c]' = \begin{bmatrix} \overline{E}_{c1} & 0 & 0 \\ 0 & \overline{E}_{c2} & 0 \\ 0 & 0 & \overline{G}_c \end{bmatrix} \qquad [D_s]_i' = \begin{bmatrix} \rho_i \overline{E}_{si} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}$$

$$\overline{E}_{c1} = \frac{f_{c1}}{\varepsilon_{c1}}; \overline{E}_{c2} = \frac{f_{c2}}{\varepsilon_{c2}}; \overline{G}_c = \frac{\overline{E}_{c1} \cdot \overline{E}_{c2}}{\overline{E}_{c1} + \overline{E}_{c2}} \qquad \overline{E}_{si} = \frac{f_{si}}{\varepsilon_{si}}$$

$$[D_c] = [T_c]^T [D_c]' [T_c] + [T_f]^T [D_f]' [T_f]$$

$$[D_s]_i = [T_s]_i^T [D_s]_i' [T_s]_i$$

$$[T] = \begin{bmatrix} \cos^2\psi & \sin^2\psi & \cos\psi \cdot \sin\psi \\ \sin^2\psi & \cos^2\psi & -\cos\psi \cdot \sin\psi \\ -2\cos\psi \cdot \sin\psi & 2\cos\psi \cdot \sin\psi & (\cos^2\psi - \sin^2\psi) \end{bmatrix}$$

INTRODUCTION

REQUIREMENTS

The proposed solution consists in implementing the suite of calculation (Straus7) with an Add-on that executes nonlinear static analysis (Pushover analysis) for reinforced concrete buildings, with the following features:

- Possibility to perform nonlinear analysis of mixed wall-frame structures, in which it is possible to combine, through the sub-modeling technique, the nonlinearity of Straus7 with the secant stiffness formulation of the smeared crack models implemented for walls and slab elements.

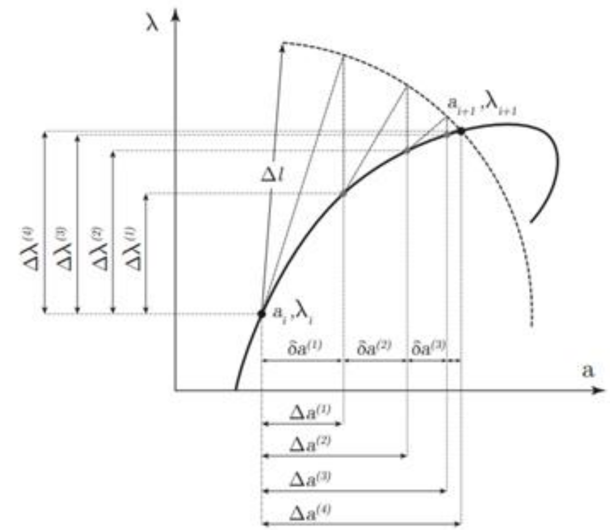


Figure 2.4: A schematic representation of the Arc-Length method iterations. a denotes a normalized displacement whereas λ the load incrementation parameter. The increment is defined by the radius of the circle Δl and the next point is the point of intersection between the path and the circle.

INTRODUCTION

REQUIREMENTS

The proposed solution consists in implementing the suite of calculation (Straus7) with an Add-on that executes nonlinear static analysis (Pushover analysis) for reinforced concrete buildings, with the following features:

- Possibility to perform multimodal adaptive pushover analysis, whose load distribution is continuously updated during the analysis, to reflect the progressive stiffness degradation of the structure.

- adaptive load distribution (belonging to Group 2), continuously updated during the analysis, to reflect the progressive stiffness degradation of the structure. This distribution considers the contribution of multiple modes of vibration, updated at each load step (*Adaptive Analysis* module):

$$F_i = \sqrt{\sum_n \sum_m \rho_{n,m} \cdot F_{i,n} \cdot F_{i,m}}$$

where

$$F_{ij} = \Gamma_j \cdot \phi_{i,j} \cdot M_i \cdot S_a(T_j, \xi)$$

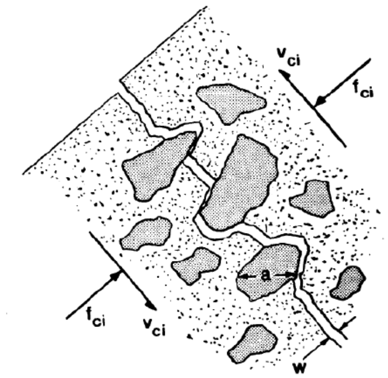
$$\rho_{n,m} = \frac{8\xi^2 \beta_{i,j}^{3/2}}{(1+\beta_{i,j}) \cdot \left[(1-\beta_{i,j})^2 + 4 \cdot \xi^2 \cdot \beta_{i,j}\right]}$$

being
i = node index; j, n, m = index of vibration mode
ξ = damping factor for vibration modes
$\beta_{i,j} = T_j/T_i$ = inverse ratio between the periods of each i-j pair of modes

NONLINEARITY OF THE STRUCTURE

- **Concrete constitutive law**

The tensile bond σ-ε of concrete is generally represented by a linear elastic branch up to the value of the tension equal to the maximum tensile strength ft, and then continues with a softening branch, which represents the capacity of concrete, also unreinforced, to absorb pulling forces after cracking. This "residual" resistance is linked to the aggregate interlock. By increasing the intensity of the load, the crack opens and the contribution offered by this mechanism tends to disappear.



Shear across with crack through aggregate interlock

NONLINEARITY OF THE STRUCTURE

- **Concrete constitutive law**

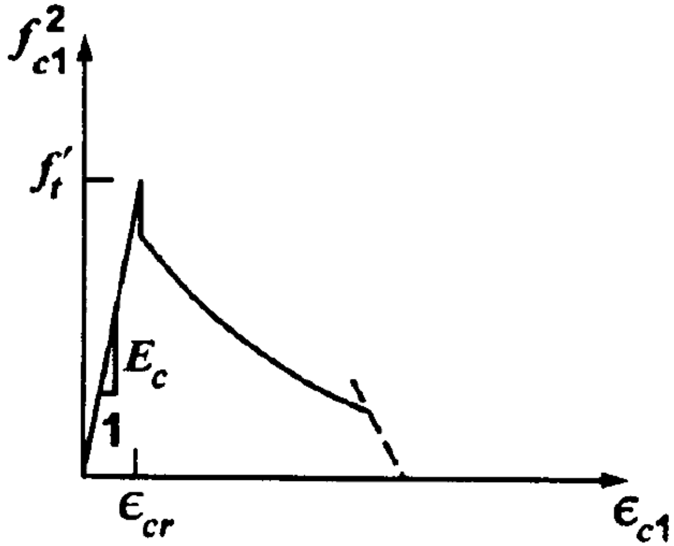
In the case of reinforced concrete, the softening branch can be modified by adding to the resistance offered by the aggregates the contribution due to the tensile stress absorbed by the concrete between two cracks for adherence with the steel bars.
This is the approach generally used in finite element analyses, which implement in the used algorithm a constitutive bond of the concrete in tension, able to model the phenomenon of tension stiffening.

NONLINEARITY OF THE STRUCTURE

- **Concrete constitutive law**

Model of tension stiffening proposed by (Vecchio, 2000)

The constitutive bond of the concrete in tension consists of a first elastic branch and a softening branch whose analytical expression is:



$$f_{c1}^a = \frac{f'_t}{1+\sqrt{c_t \varepsilon_{c1}}} \qquad \varepsilon_{cr} < \varepsilon_{c1}$$

$$c_t = 2.2m$$

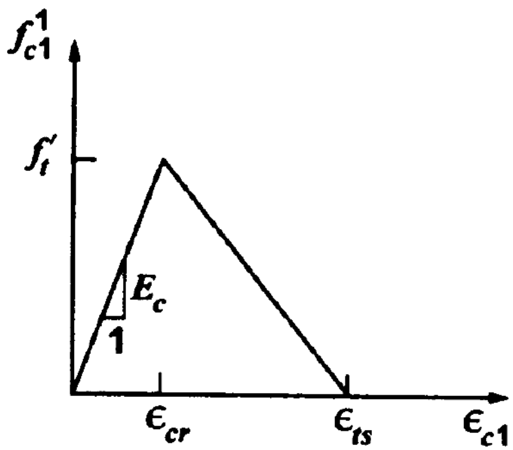
$$\frac{1}{m} = \sum_{i=1}^{n} \frac{4\rho_i}{d_{b_i}} |\cos\theta_{n_{ii}}| \qquad (in\ mm)$$

NONLINEARITY OF THE STRUCTURE

- **Concrete constitutive law**

Model of tension softening proposed by (Vecchio, 2000)

This formulation is sufficient to describe the behavior of a concrete element or a weakly reinforced concrete element in which tension softening prevails over tension stiffening.



$$f_{c1} = max\left(f_{c1}{}^{a}, f_{c1}{}^{b}\right)$$

After cracking, concrete can continue to carry tensile stresses as a result of two independent mechanisms: tension softening and tension stiffening. Tension softening refers to the fracture-associated mechanisms described by Darwin and others. It is particularly significant in concrete structures containing little or no reinforcement; for example, beams containing no web steel. Here, the concrete postcracking tensile stress associated with tension softening f_{c1}^a is calculated

$$f_{c1}^a = f_t'\left[1 - \frac{(\varepsilon_{c1} - \varepsilon_{cr})}{(\varepsilon_{ts} - \varepsilon_{cr})}\right] \qquad (33)$$

where the terminal strain ε_{ts} is calculated from the fracture energy parameter G_f and characteristic length L_r as follows:

$$\varepsilon_{ts} = 2.0\,\frac{G_f}{f_t' \cdot L_r} \qquad (34)$$

The parameter G_f is taken as having a constant value of 75 N/m. The resulting concrete tension softening formulation is illustrated in Fig. 9(b). This linear formulation is sufficient for most applications, but more accurate nonlinear models can be used.

NONLINEARITY OF THE STRUCTURE

- **Concrete constitutive law**

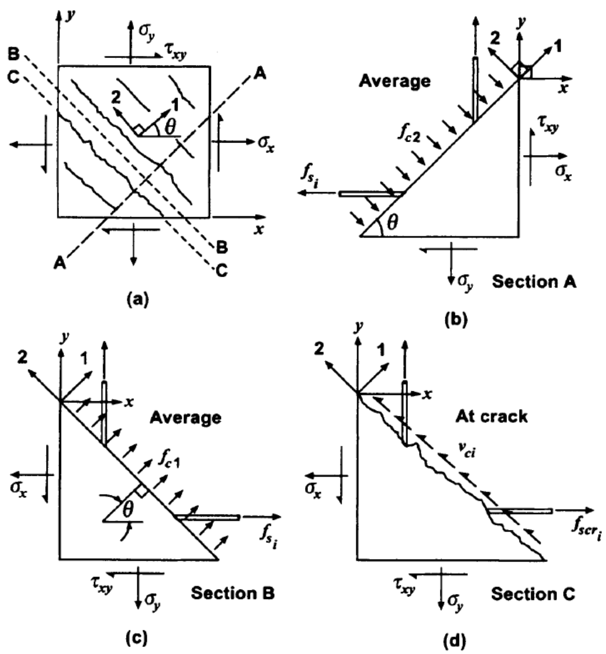


FIG. 6. Equilibrium Conditions: (a) External Conditions; (b) Perpendicular to Crack Direction; (c) Parallel to Crack Direction; (d) Along Crack Surface

Crack interfaces can be considered planes of weakness in the continuum, and it is necessary to check that the average stresses can be transmitted across the cracks. It will be assumed that the component of the concrete principal tensile stresses due to tension stiffening is zero at the crack location. To transmit the average stress f_{c1}, local increases in the reinforcement stresses are necessary. These local stresses are denoted as f_{scr_i}, as shown in Fig. 6(d). The magnitude of f_{c1} that can be transmitted via this mechanism is limited by the reserve capacity of the reinforcement, which is given by the difference between the average stresses and the yield stresses. Hence

$$f_{c1} \leq \sum_{i=1}^{n} \rho_i (f_{y_i} - f_{s_i}) \cdot \cos^2\theta_{n_i} \qquad (5)$$

where ρ_i = reinforcement ratio; f_{s_i} = average stress; f_{y_i} = yield stress for the ith reinforcement component; and the angle θ_{n_i} = difference between the angle of orientation of the reinforcement, α_i, and the normal to the crack surface θ:

$$\theta_{n_i} = \theta - \alpha_i \qquad (6)$$

NONLINEARITY OF THE STRUCTURE

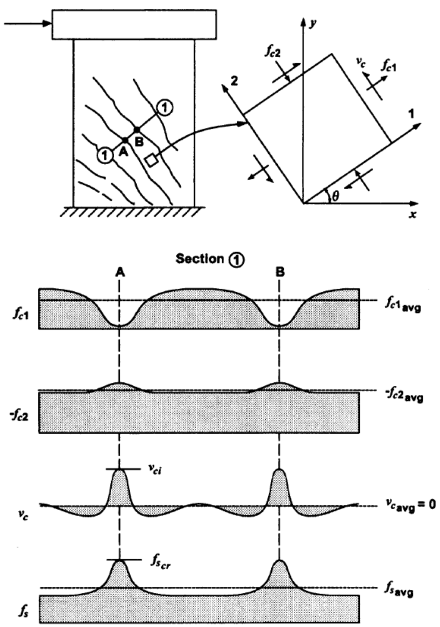
- **Nonlinear behavior of reinforced concrete shear walls**

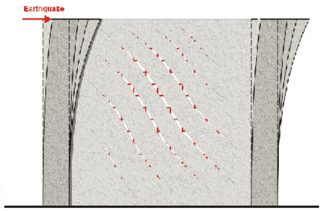
The analytical model at the base of the module has been developed from the Modified Compression Field Theory (MCFT) and Disturbed Stress Field Model (DSFM), developed by Profs. Frank J. Vecchio, and Michael P. Collins (1986) of the University of Toronto, and is able to predict the response of reinforced concrete elements, representing the cracked concrete as an orthotropic material with rotating smeared cracks, while each steel Layer is represented as an orthotropic material able to resist only to uniaxial stresses in the bar direction of the same layer. The various layers of concrete and reinforcing bars are assembled to form an anisotropic elastic material.

NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEARITY OF THE STRUCTURE

- Nonlinear behavior of reinforced concrete shear walls





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$$\overline{E}_{c1} = \frac{f_{c1}}{\varepsilon_{c1}}\ ; \overline{E}_{c2} = \frac{f_{c2}}{\varepsilon_{c2}}\ ; \ \overline{G}_c = \frac{\overline{E}_{c1} \cdot \overline{E}_{c2}}{\overline{E}_{c1} + \overline{E}_{c2}} \qquad \overline{E}_{si} = \frac{f_{si}}{\varepsilon_{si}}$$

$$[D_c] = [T_c]^T [D_c]' [T_c] + [T_f]^T [D_f]' [T_f]$$

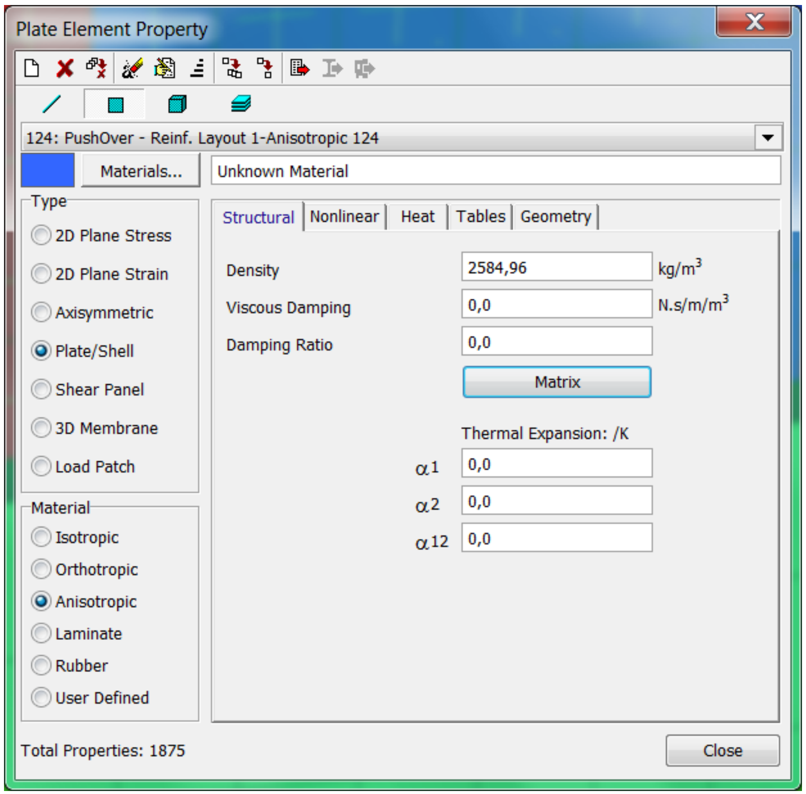
$$[D_s]_i = [T_s]_i^T [D_s]_i' [T_s]_i$$

$$[T] = \begin{bmatrix} \cos^2\psi & \sin^2\psi & \cos\psi \cdot \sin\psi \\ \sin^2\psi & \cos^2\psi & -\cos\psi \cdot \sin\psi \\ -2\cos\psi \cdot \sin\psi & 2\cos\psi \cdot \sin\psi & (\cos^2\psi - \sin^2\psi) \end{bmatrix}$$

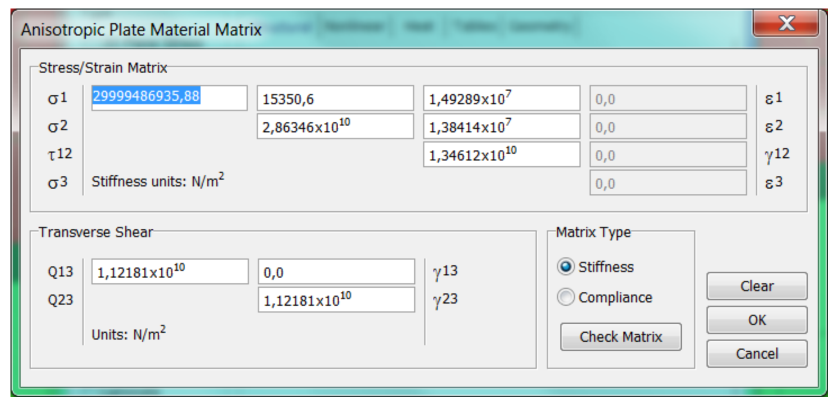
NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**



The elements used in Straus7 are *Anisotropic Plate/Shell* elements.



NONLINEARITY OF THE STRUCTURE

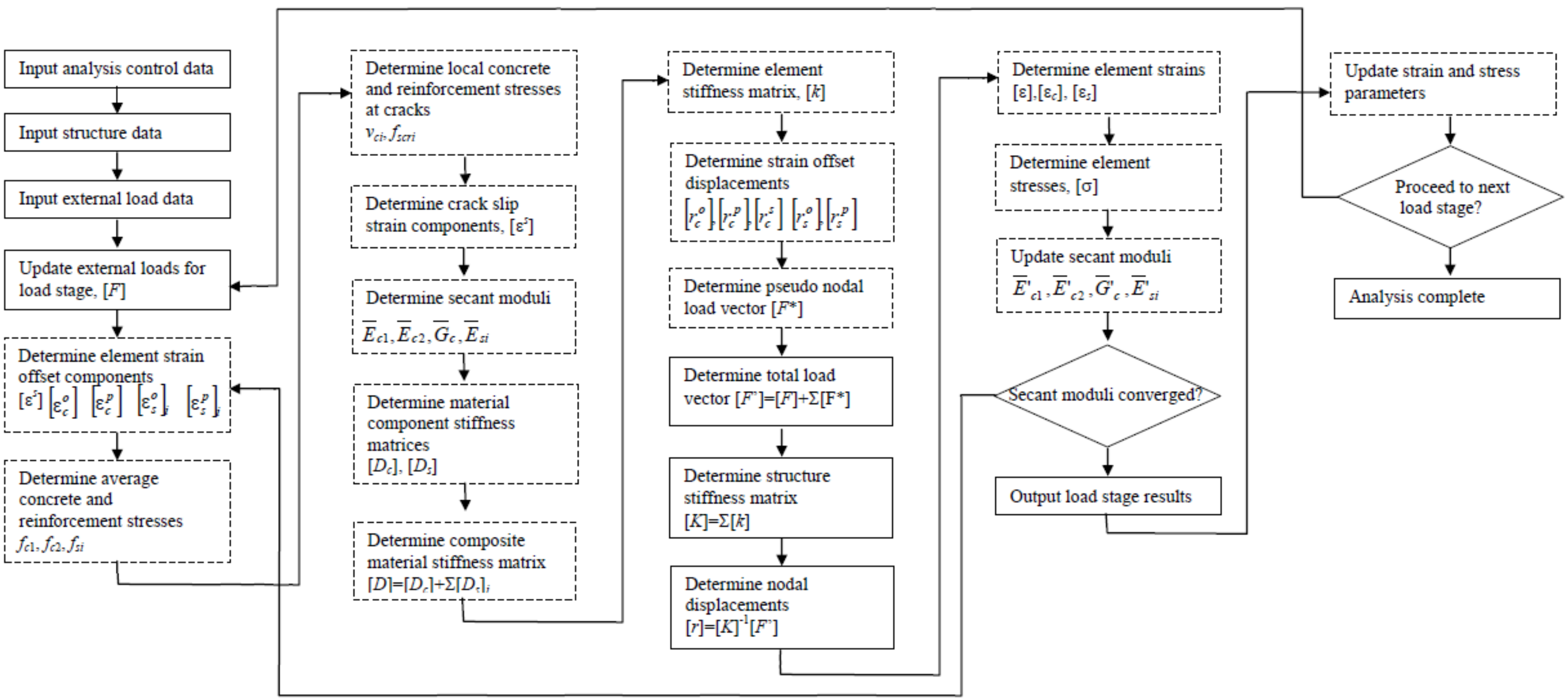
- **Nonlinear behavior of reinforced concrete shear walls**

The nonlinear calculation is obtained by adopting an iterative secant stiffness algorithm, in which the stiffness of the elastic anisotropic materials defined by EasyOver are updated at each iteration into a submodel created for the purpose (in which the displacements obtained in the global model are imposed to the plate elements). Once the convergence in the sub model is achieved, EasyOver updates the main model, by assigning the stiffness matrices obtained by the iterative calculation on the sub-model to the plate elements, applies a new load increment for the current stage, such as to obtain the same displacements obtained in the previous application, then it passes to the next load increment.

NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEARITY OF THE STRUCTURE

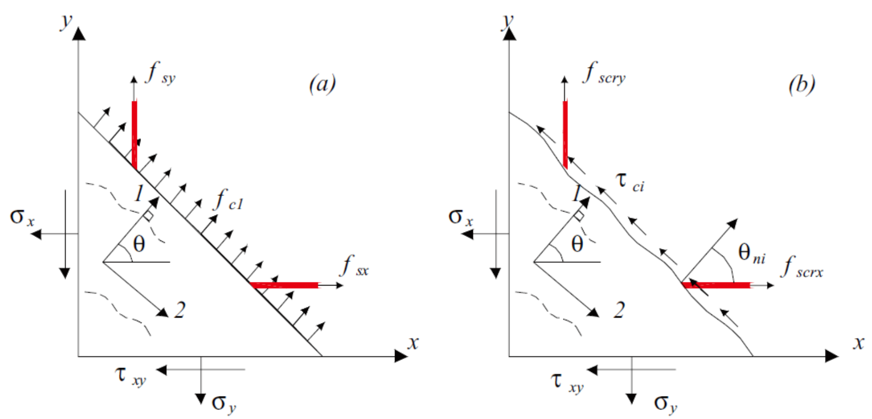
- Nonlinear behavior of reinforced concrete shear walls



NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

The modeling of the cracked concrete element is performed by using an orthotropic material and a rotating smeared crack model. The cracked concrete is treated like a continuous medium with cracks smeared across the membrane element.
This formulation evaluates the average stresses and strains (in the region between the cracks) and the local stresses and strains of the concrete and reinforcement, as well as the widths and orientations of the cracks during the loading. Based on this information, the failure mode of the element can be determined.

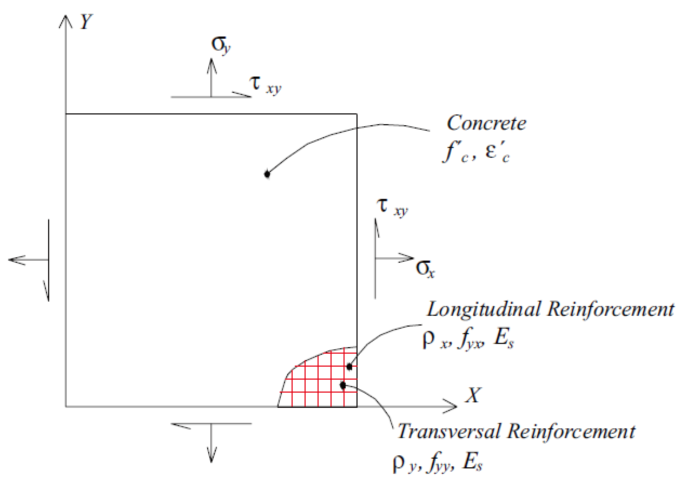


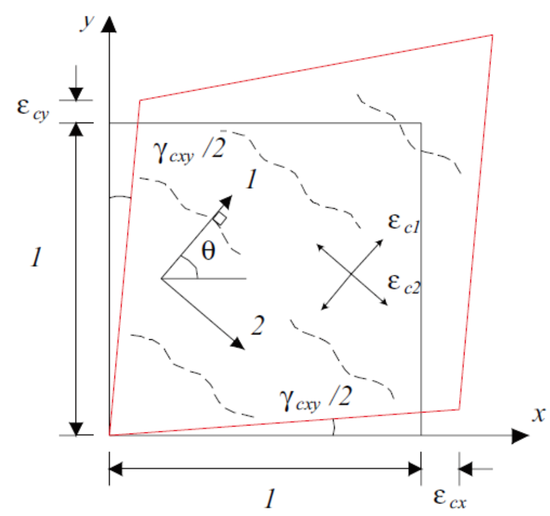
NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

The model is based on four groups of relationships:

- compatibility relationships for the average strains in the concrete and reinforcement

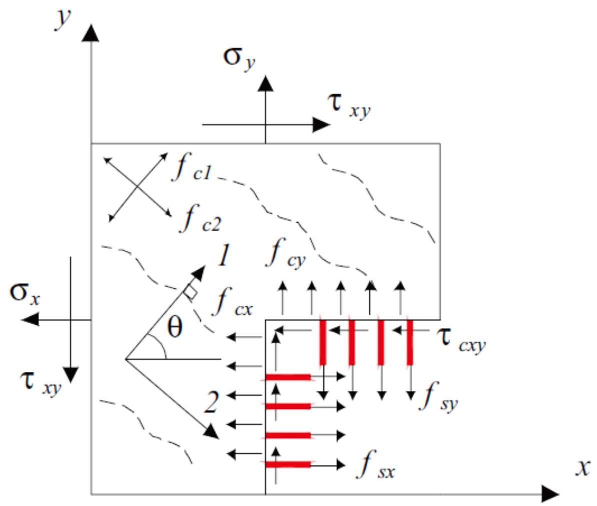


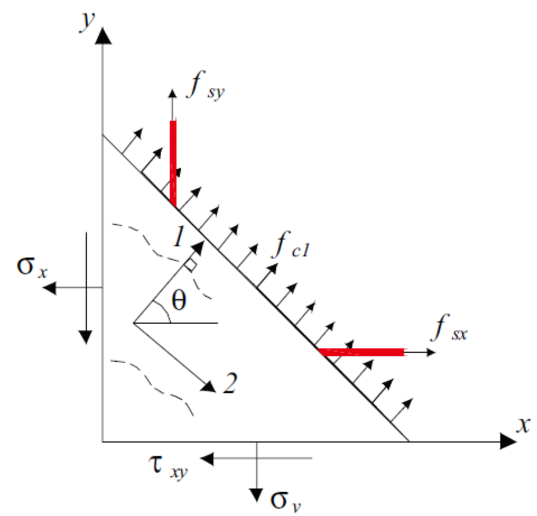


NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

- equilibrium relationships for the average stresses in the concrete and reinforcement



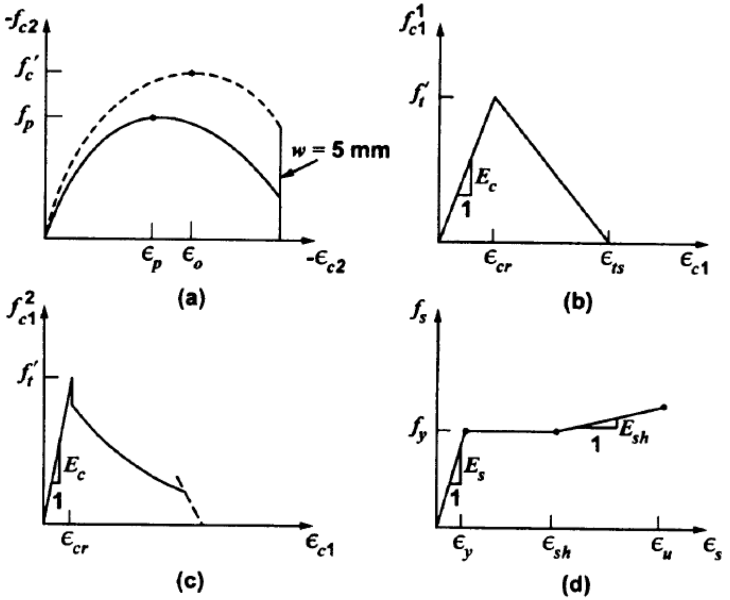


NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

- constitutive relationships for the cracked concrete and reinforcement

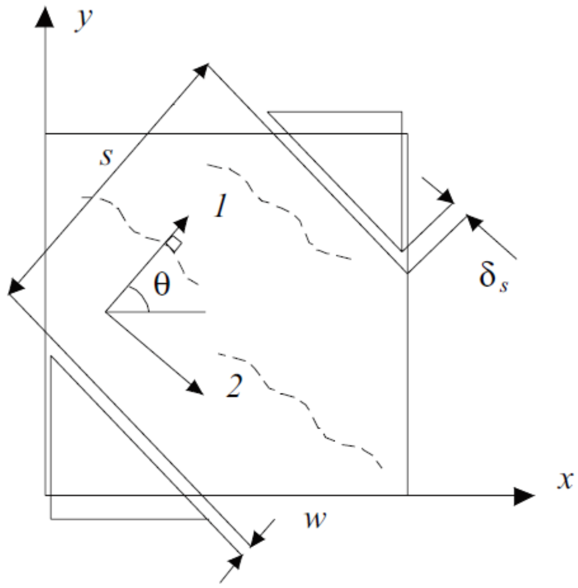


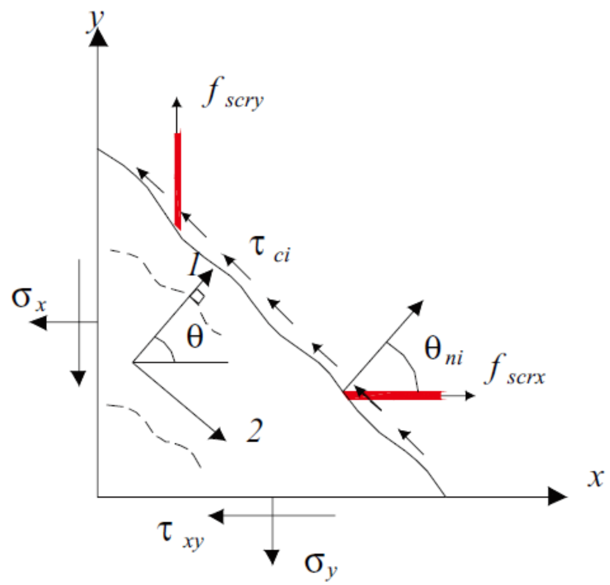
Constitutive Relations: (a) Compressive Softening Model; (b) Tension Softening Model; (c) Tension Stiffening Model; (d) Reinforcing Steel Response

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

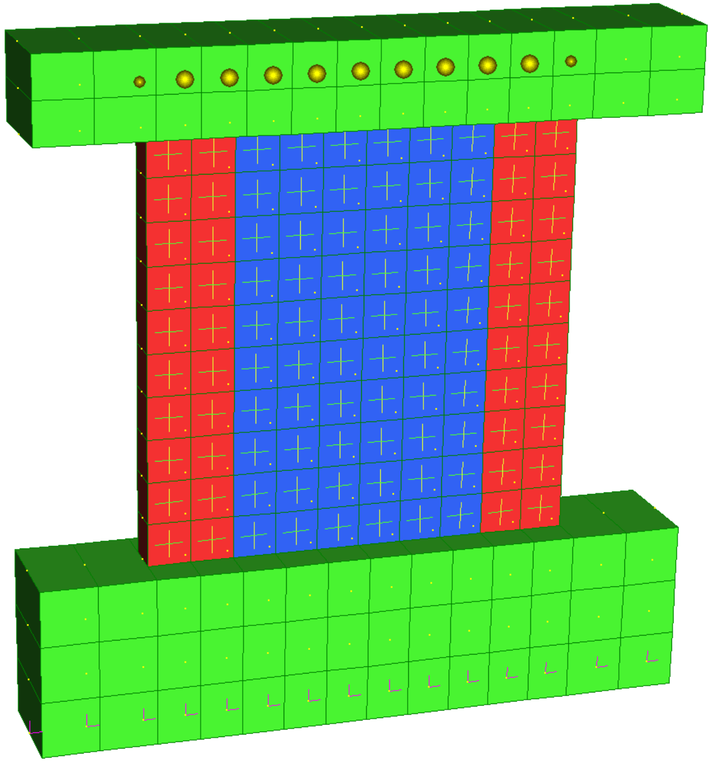
- compatibility relationships for crack shear slip





NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**



Model with a wall of dimensions 75x75 cm and 7 cm thick, at the top of which non-structural masses are applied for a total of 46,000 kg.
The wall will be loaded by lateral forces lying on its plane, according to the X axis of the global model.
Area 1 (central plates):
- d.6/8 cm in dir. of local axis x (horiz. reinf.)
- d.8/6 cm in dir. of local axis y (vert. reinf.)

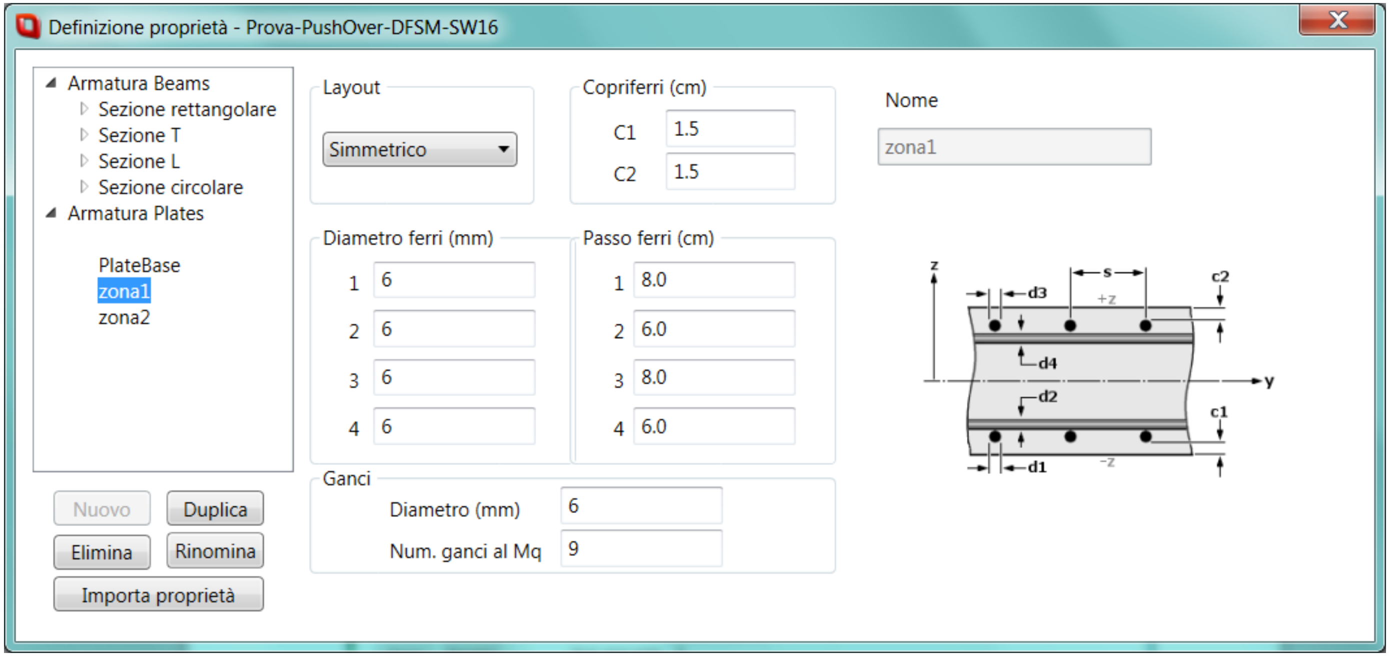
Area 2 (wall end plates):
- d.6/8 cm in dir. of local axis x (horiz. reinf.)
- d.8/5 cm in dir. of local axis y (vert. reinf.)
- d.4/5 cm in dir. of local axis y (stirrups)

Concrete cover = 15 mm

PUSHOVER ANALYSIS CASE STUDY

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**



In the sub menu *Plate reinforcement* of EasyOver you can create and define properties for the reinforcement of plate elements. The screen is quite similar to the one in Straus7 for Plate RC properties.

NONLINEARITY OF THE STRUCTURE

- Nonlinear behavior of reinforced concrete shear walls

Out-of-plane stresses and deformation, due to the confinement of lateral expansion by out-of-plane reinforcement (No. hooks / sq)

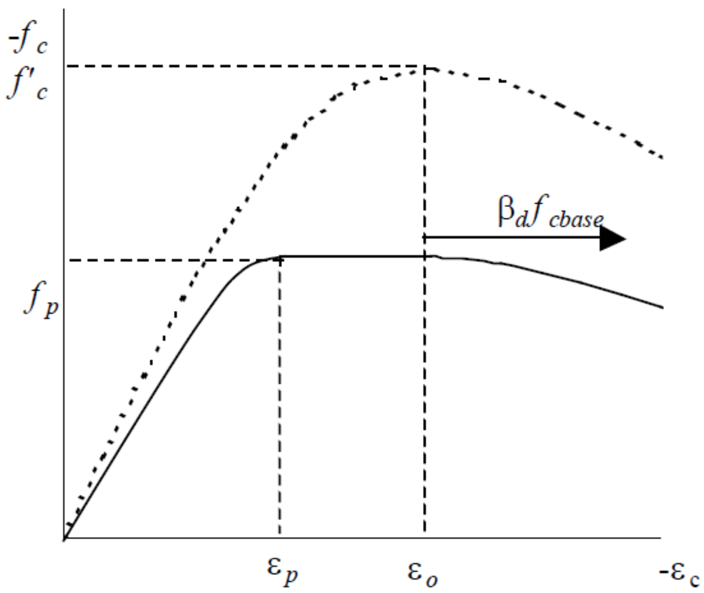
$$\varepsilon_{cz} = \frac{E_c}{E_c + \rho_z \cdot E_{sz}} \left(- \nu_{12} \frac{f_{c2}}{\bar{E}_{c2}} - \nu_{21} \frac{f_{c1}}{\bar{E}_{c1}} \right)$$

$$\varepsilon_{cz} = - \frac{\rho_z \cdot f_{z,yield}}{E_c} - \nu_{12} \frac{f_{c2}}{\bar{E}_{c2}} - \nu_{21} \frac{f_{c1}}{\bar{E}_{c1}}$$

$$f_{sz} = E_s \varepsilon_{cz} \leq f_{z,yield} \qquad f_{cz} = -\rho_z \cdot f_{sz}$$

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**



$$f_p = \beta_d \beta_l f'_c$$

$$\varepsilon_p = \beta_d \beta_l \varepsilon_o$$

$$\beta_d \rightarrow compression\ softening$$
$$\beta_l \rightarrow confinement$$

NONLINEARITY OF THE STRUCTURE

- Nonlinear behavior of reinforced concrete shear walls

ACI STRUCTURAL JOURNAL TECHNICAL PAPER
Title no. 87-S3

Behavior of Reinforced Concrete Structural Walls: Strength, Deformation Characteristics, and Failure Mechanism

Ioannis D. Lefas, Michael D. Kotsovos, and Nicholas N. Ambraseys

Table 1 — Properties of reinforcement bars

Type	Yield strength f_{sy}, MPa	Ultimate strength f_{su}, MPa
8 mm high-tensile bar	470	565
6.25 mm high-tensile bar	520	610
4 mm mild-steel bar	420	490

1 mm = 0.0394 in.; 1 MPa = 145 psi.

Model with a wall of dimensions 75x75 cm and 7 cm thick, at the top of which non-structural masses are applied for a total of 46,000 kg.
The wall will be loaded by lateral forces lying on its plane, according to the X axis of the global model.
Area 1 (central plates):
- d.6/8 cm in dir. of local axis x (horiz. reinf.)
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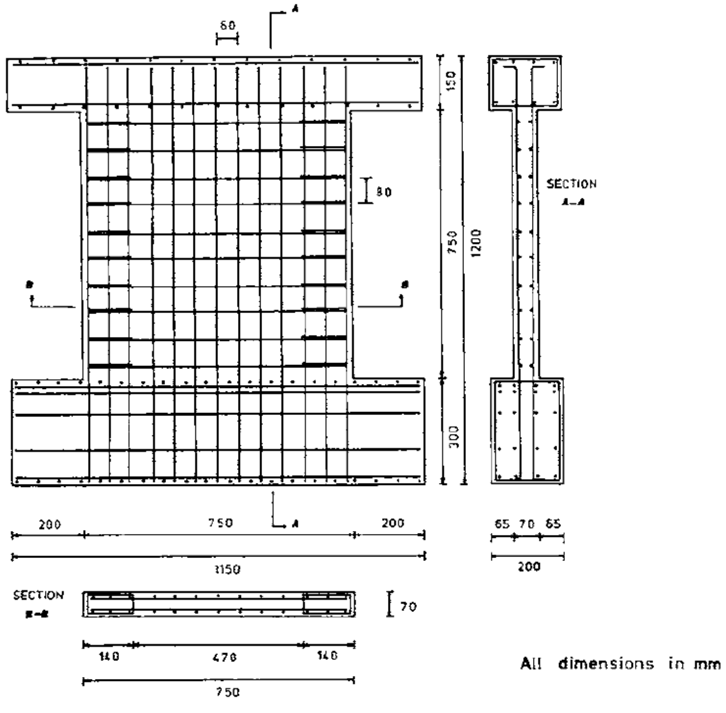
- d.4/5 cm in dir. of local axis y (stirrups)

Concrete cover = 15 mm

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**



(a)

Model with a wall of dimensions 75x75 cm and 7 cm thick, at the top of which non-structural masses are applied for a total of 46,000 kg.
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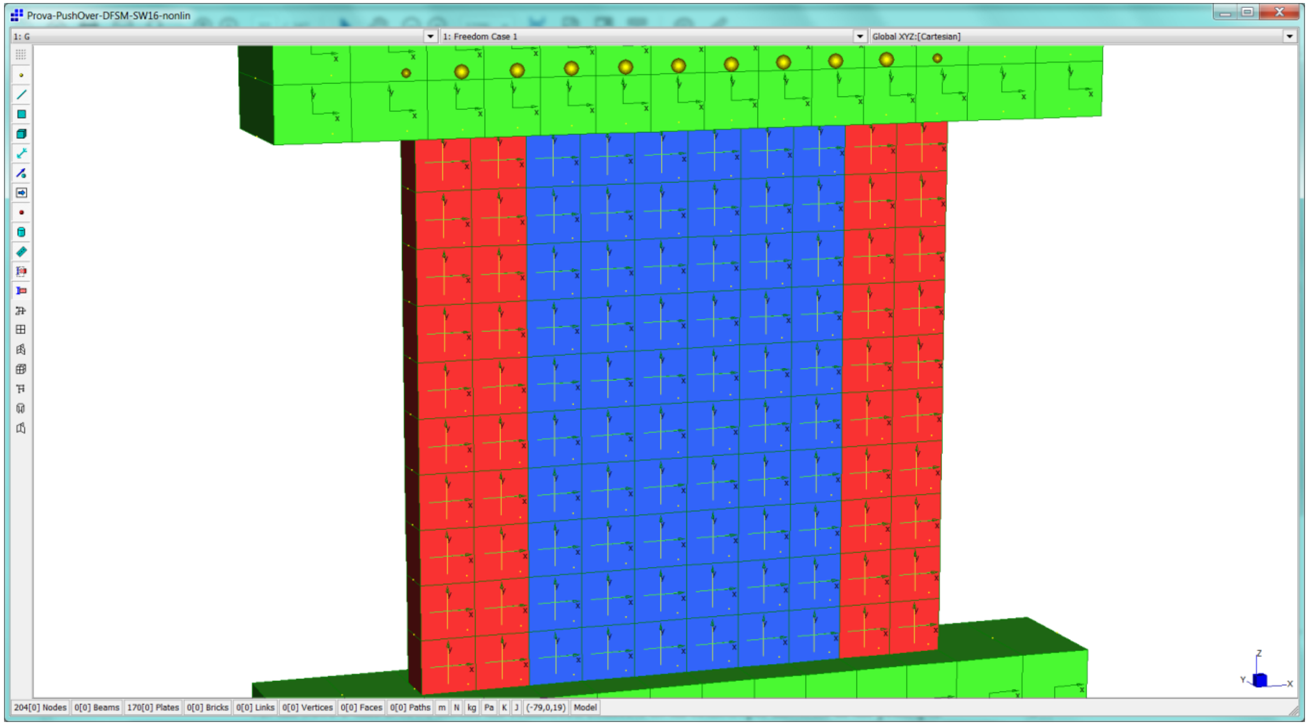
Concrete cover = 15 mm

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

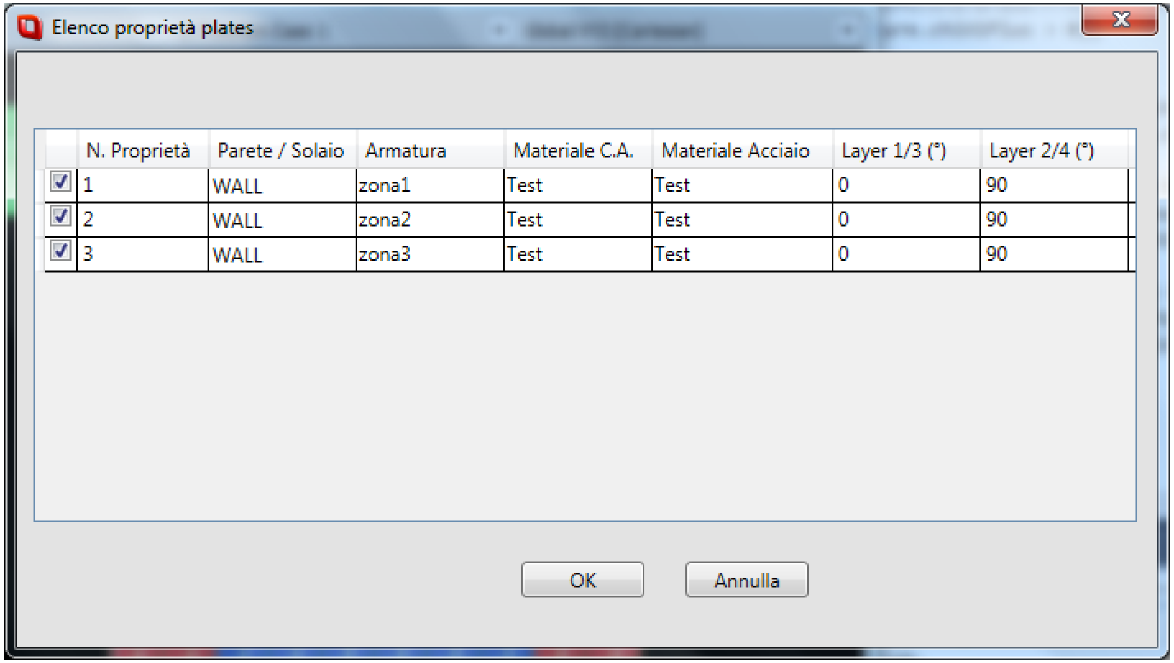
It is important to correctly orient the various plate elements in the Straus7 model.



NONLINEARITY OF THE STRUCTURE

- Nonlinear behavior of reinforced concrete shear walls

You can assign to the various Straus7 plate properties the properties for the reinforcement defined in the menu just seen, as well as the concrete and steel material.



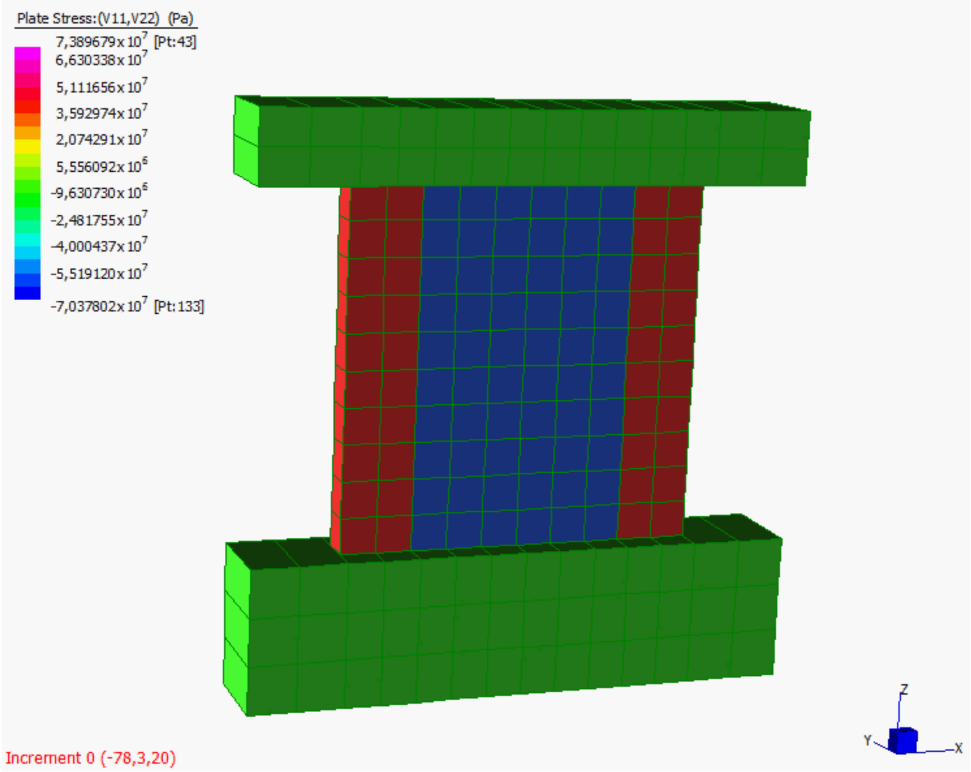
Elenco proprietà plates

	N. Proprietà	Parete / Solaio	Armatura	Materiale C.A.	Materiale Acciaio	Layer 1/3 (°)	Layer 2/4 (°)
☑	1	WALL	zona1	Test	Test	0	90
☑	2	WALL	zona2	Test	Test	0	90
☑	3	WALL	zona3	Test	Test	0	90

PUSHOVER ANALYSIS
CASE STUDY

NONLINEARITY OF THE STRUCTURE

- Nonlinear behavior of reinforced concrete shear walls

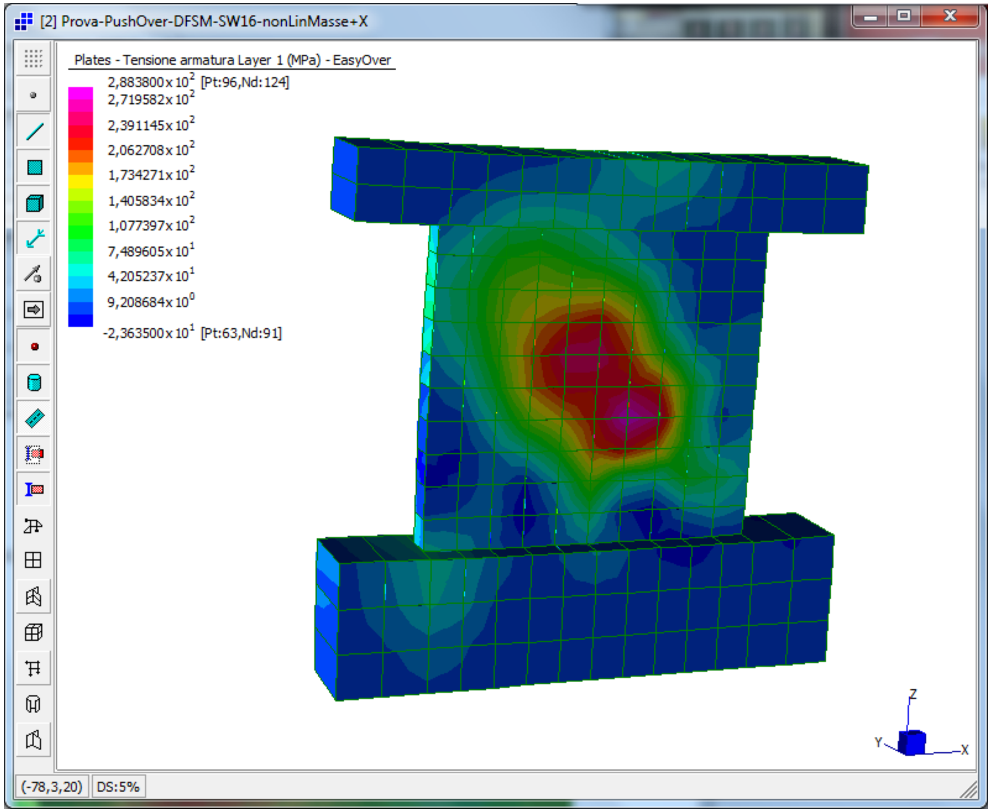


PUSHOVER ANALYSIS CASE STUDY

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

Stress in the transverse reinforcement (layers 1 and 3). Maximum stress of 288 MPa (tensile stress).

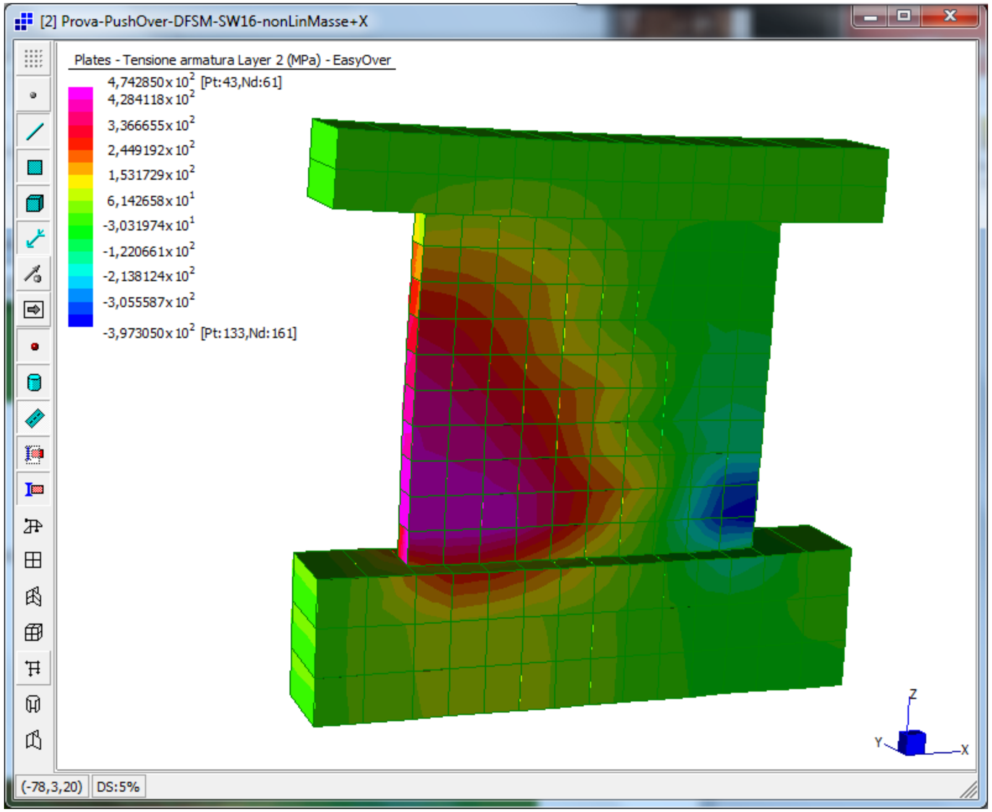


PUSHOVER ANALYSIS
CASE STUDY

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

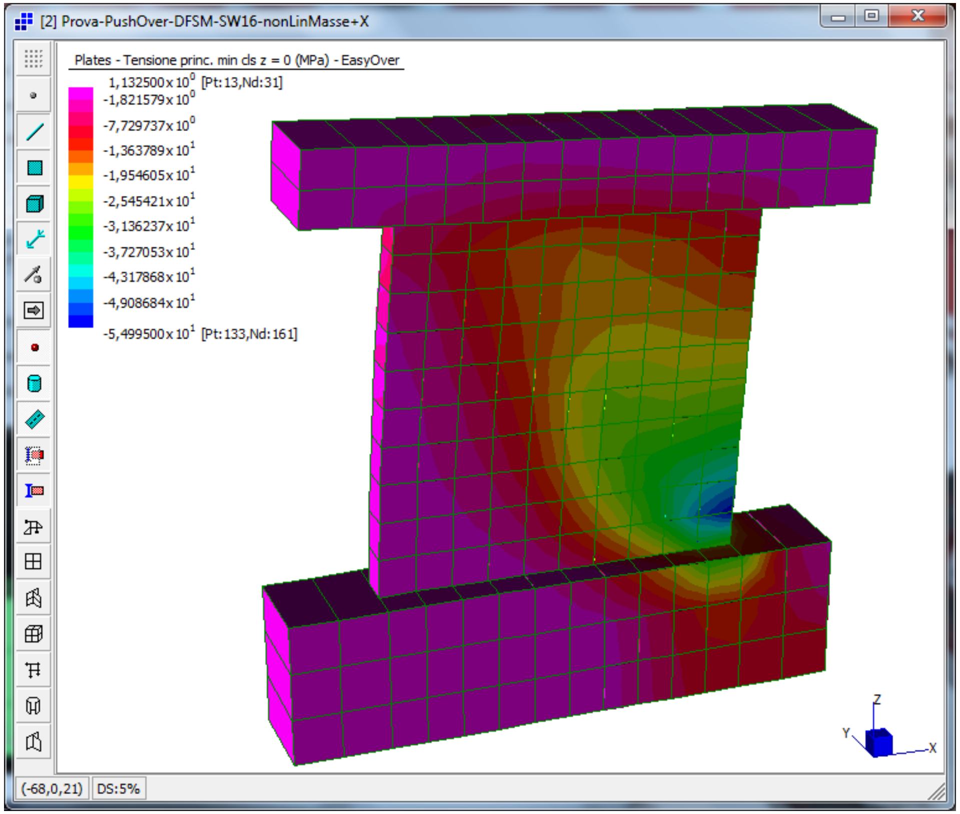
Stress in the vertical reinforcement (layers 2 and 4).
Maximum stress of 474 MPa (tensile stress).



NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

Compressive stress in the concrete. Maximum stress of 55 MPa.



PUSHOVER ANALYSIS CASE STUDY

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

Maximum base shear equal to 354.903 N
Failure by web crushing, which occurs simultaneously with the yield of the reinforcement.

PUSHOVER ANALYSIS
CASE STUDY

NONLINEARITY OF THE STRUCTURE

- Nonlinear behavior of reinforced concrete shear walls

Comparison with the experimental results obtained from the test campaign carried out from Lefas et al. (1990): good correspondence

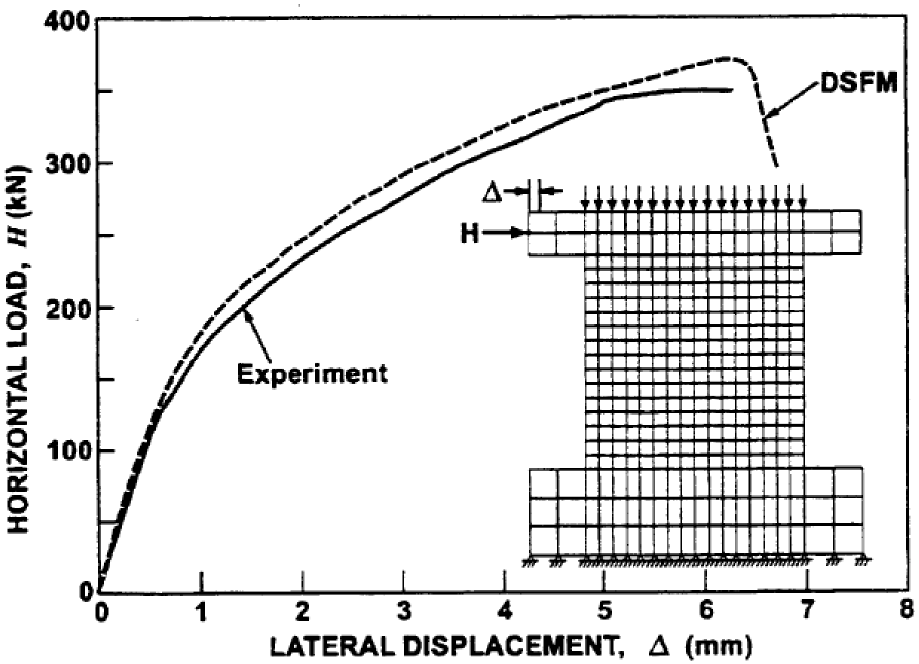


FIG. 11. Results of Finite-Element Analysis of Wall SW16

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

Comparison with analytical models for calculation of shear strength.

STRUT WITH cotθ = 1		BISKINIS WITH $\mu^{PL}_{\Delta} = 5$		BISKINIS WITH $\mu^{PL}_{\Delta} = 0$	
b (mm)	70	b (mm)	70	b (mm)	70
h (mm)	750	h (mm)	750	h (mm)	750
c (mm)	15	c (mm)	15	c (mm)	15
f_c (N/mm^2)	49	x (mm)	525,00	x (mm)	525,00
f_y (N/mm^2)	470	L_v (mm)	825	L_v (mm)	825
$A_{stirrups}$ (mm^2)	56,52	f_c (N/mm^2)	49	f_c (N/mm^2)	49
s (mm)	67	f_y (N/mm^2)	470	f_y (N/mm^2)	470
ν	0,50	μ^{pl}_{Δ}	5,00	μ^{pl}_{Δ}	-
N (N)	460.000	A_{long} (mm^2)	301,44	A_{long} (mm^2)	301,44
ω_{sw}	0,11531	$A_{stirrups}$ (mm^2)	56,52	$A_{stirrups}$ (mm^2)	56,52
σ_{cp} (N/mm^2)	8,76	s (mm)	67	s (mm)	67
α_c	1,18	N (N)	460.000	N (N)	460.000
cotθ*	1,00	ρ_{tot}	0,00574	ρ_{tot}	0,00574
cotθ	1,00	ρ_w	0,01205	ρ_w	0,01205
V_{Rsd} (N)	262.274	A_c (mm^2)	51.450	A_c (mm^2)	51.450
V_{Rcd} (N)	670.055	V_N (N)	62.727	V_N (N)	62.727
V_R (N)	**262.274**	k	0,75	k	1,00
		V_C (N)	27.296	V_C (N)	27.296
		V_W (N)	262.274	V_W (N)	262.274
		$V_{R,max}$ (N)	274.967	$V_{R,max}$ (N)	392.810
		V_R (N)	**274.967**	**V_R (N)**	**352.297**

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

Comparison with analytical models for calculation of shear strength. EC8 - Part 3

(2) The shear strength of a concrete wall, V_R, may not be taken greater than the value corresponding to failure by web crushing, $V_{R,max}$, which under cyclic loading may be calculated from the following expression (with units: MN and meters):

$$V_{R,max} = \frac{0{,}85\left(1-0{,}06\min\left(5;\ \mu_\Delta^{pl}\right)\right)}{\gamma_{el}}\left(1+1{,}8\min\left(0{,}15;\frac{N}{A_c f_c}\right)\right)\left(1+0{,}25\max\left(1{,}75;\ 100\rho_{tot}\right)\right)\left(1-0{,}2\min\left(2;\frac{L_V}{h}\right)\right)\sqrt{f_c}\,b_w z \quad \text{(A.15)}$$

where γ_{el} = 1,15 for primary seismic elements and 1,0 for secondary seismic ones, f_c is in MPa, b_w and z are in meters and $V_{R,max}$ in MN, and all other variables are as defined in **(1)**.

The shear strength under cyclic loading as controlled by web crushing prior to flexural yielding is obtained from expression (A.15) for μ_Δ^{pl}=0.

PUSHOVER ANALYSIS
CASE STUDY

NONLINEARITY OF THE STRUCTURE

- **Nonlinear behavior of reinforced concrete shear walls**

Comparison with analytical models for calculation of shear strength.

VARIABLE STRUT INCLINATION METHOD			CIRC. 617/2009	
b (mm)	70		b (mm)	70
h (mm)	750		h (mm)	750
c (mm)	15		c (mm)	15
f_c (N/mm^2)	49		f_c (N/mm^2)	49
f_y (N/mm^2)	470		f_y (N/mm^2)	470
$A_{stirrups}$ (mm^2)	56,52		A_{long} (mm^2)	301,44
s (mm)	67		$A_{stirrups}$ (mm^2)	56,52
ν	0,50		s (mm)	67
N (N)	460.000		N (N)	460.000
ω_{sw}	0,11531		ρ_l	0,00586
σ_{cp} (N/mm^2)	8,76		σ_{cp} (N/mm^2)	8,76
α_c	1,18		α_c	1,18
$\cot\theta^*$	2,03		k	1,52
$\cot\theta$	2,03		v_{min} (N/mm^2)	0,460
V_{Rsd} (N)	531.685		V_{Rc} (N)	96.410
V_{Rcd} (N)	531.685		V_{Rsd} (N)	262.274
V_R (N)	**531.685**		$V_{Rd,max}$ (N)	670.055
			V_R (N)	**358.684**

NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEAR ANALYSIS OF MIXED WALL-FRAME STRUCTURES

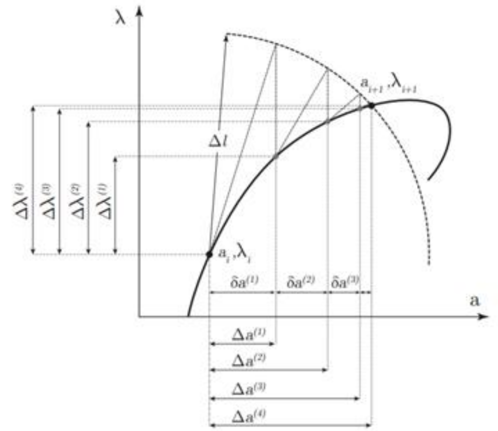
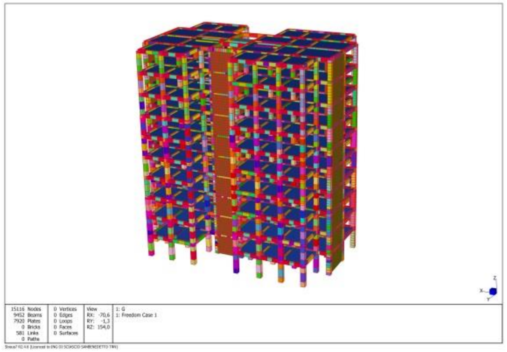


Figure 2.4: A schematic representation of the Arc-Length method iterations. a denotes a normalized displacement whereas λ the load incrementation parameter. The increment is defined by the radius of the circle Δl and the next point is the point of intersection between the path and the circle.

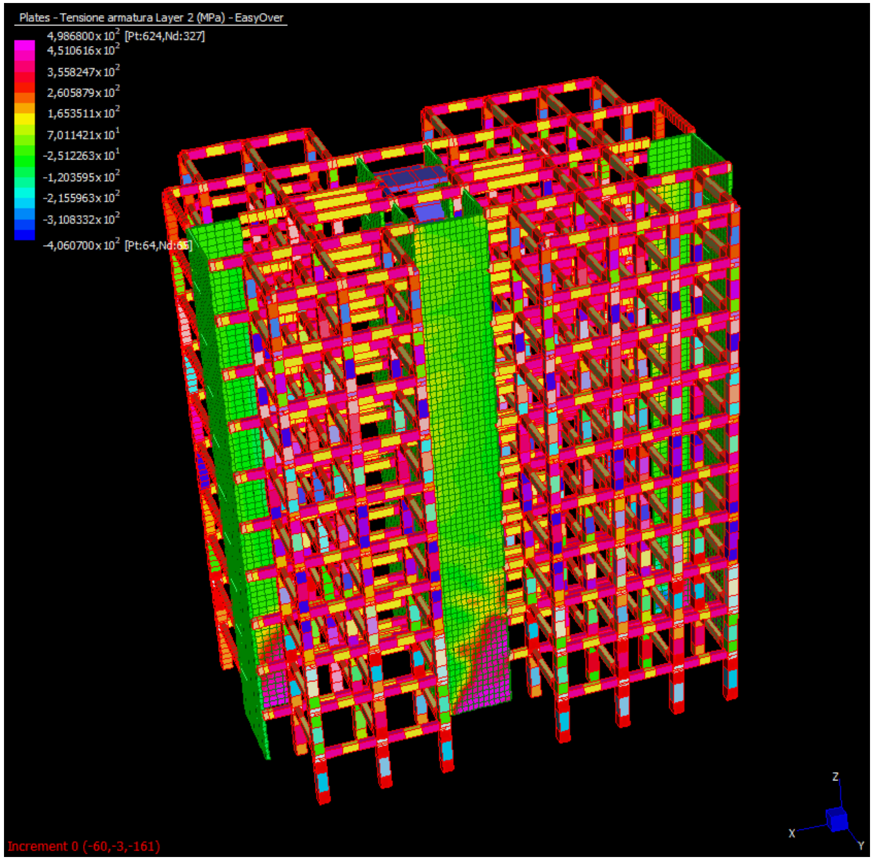


NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

ENGIN SOFT

HSH

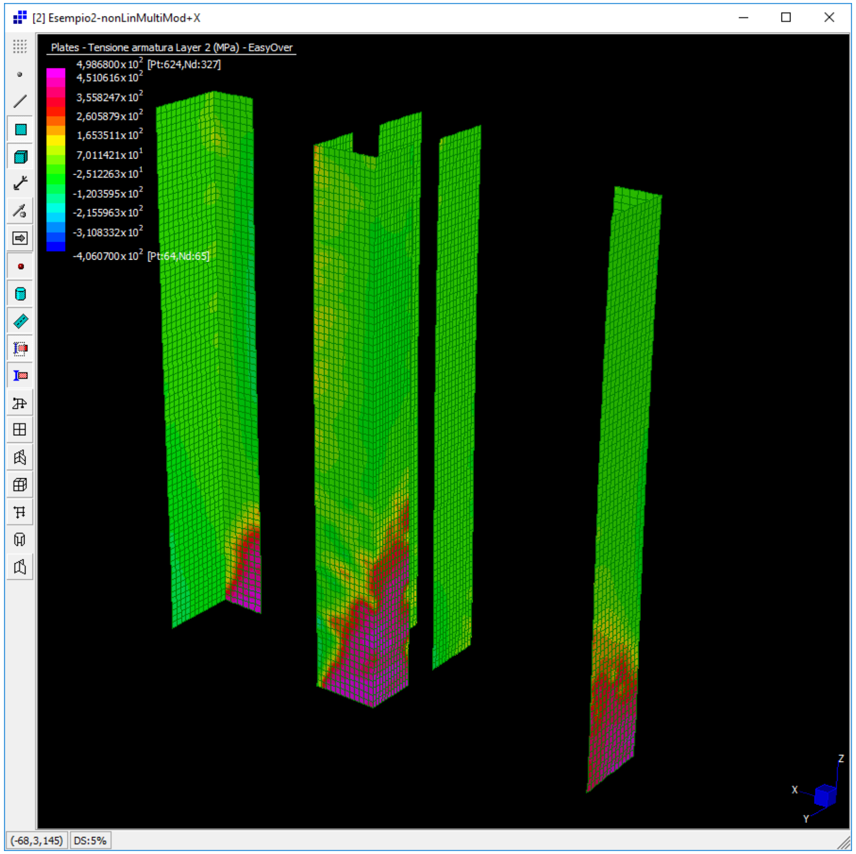
NONLINEAR ANALYSIS OF MIXED WALL-FRAME STRUCTURES



NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEAR ANALYSIS OF MIXED WALL-FRAME STRUCTURES

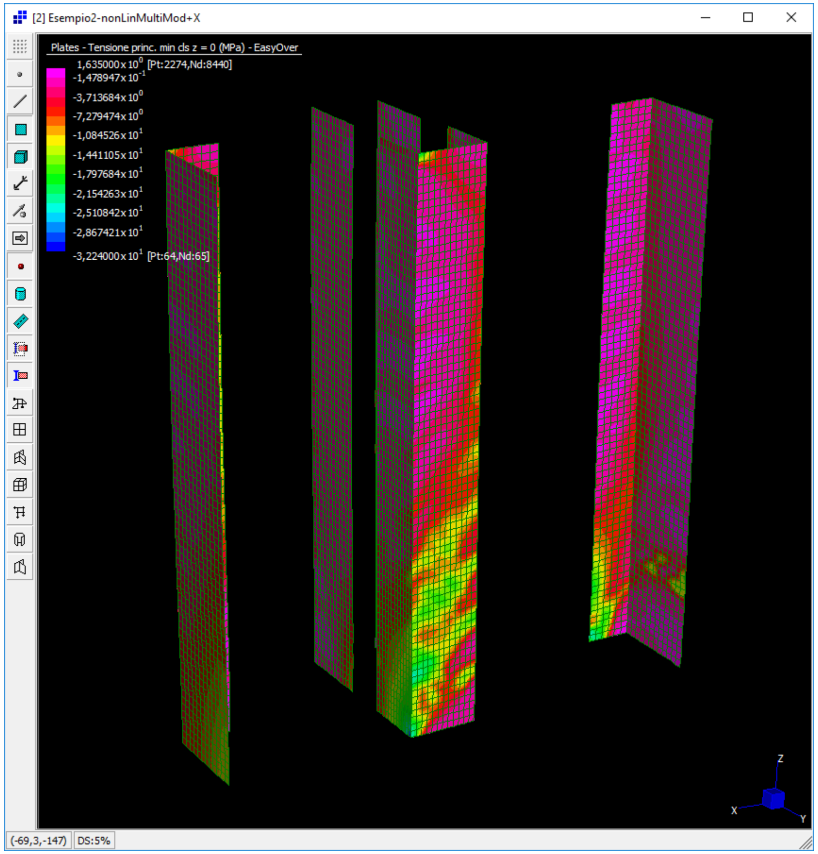
Stress in the vertical reinforcement (layers 2 and 4).



NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

NONLINEAR ANALYSIS OF MIXED WALL-FRAME STRUCTURES

Compressive stress in the concrete.



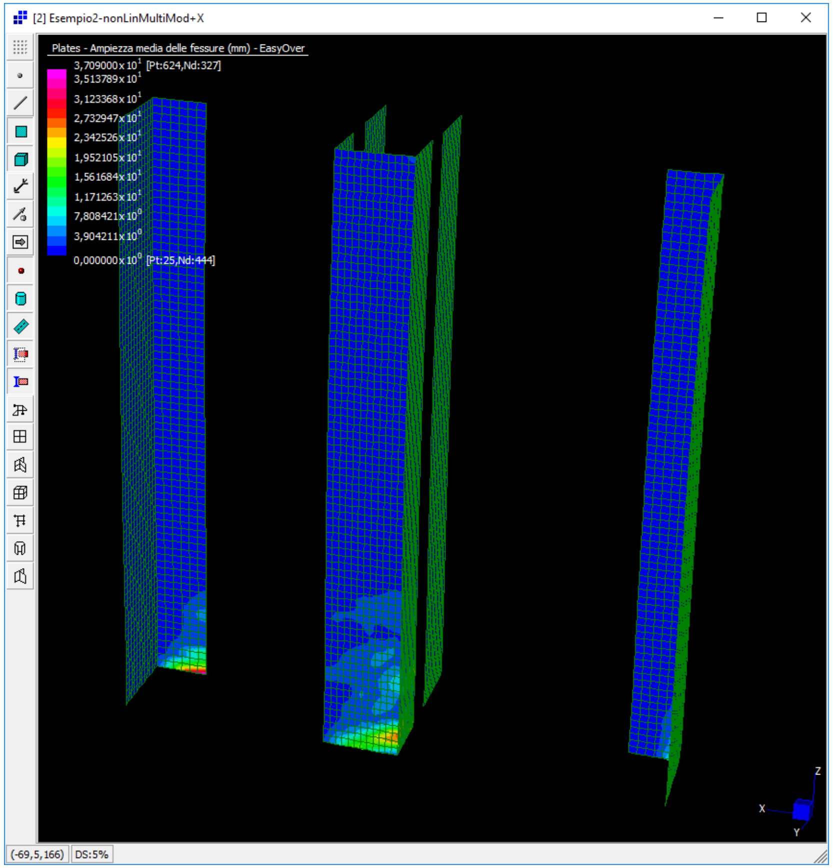
NONLINEAR BEHAVIOR OF REINFORCED CONCRETE SHEAR WALLS

ENGIN SOFT

HSH

NONLINEAR ANALYSIS OF MIXED WALL-FRAME STRUCTURES

Average width of the cracks.



LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

- distribution proportional to the fundamental mode in the direction considered (belonging to Group 1):

$$F_i = \frac{m_i \cdot \phi_{fond,i}}{\sum_{j=1}^{N} m_j \cdot \phi_{fond,j}} V_b$$

- distribution calculated by combining modal responses from a response spectrum analysis of the building (belonging to Group 1):

$$F_i = \sqrt{\sum_n \sum_m \rho_{n,m} \cdot F_{i,n} \cdot F_{i,m}}$$

where

$$F_{ij} = \Gamma_j \cdot \phi_{i,j} \cdot M_i \cdot S_a\left(T_j, \xi\right)$$

$$\rho_{n,m} = \frac{8\xi^2 \beta_{i,j}^{3/2}}{\left(1+\beta_{i,j}\right) \cdot \left[\left(1-\beta_{i,j}\right)^2 + 4 \cdot \xi^2 \cdot \beta_{i,j}\right]}$$

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

- analysis of different modes of vibration in the case of irregular buildings and calculation of the load profile corresponding to the equivalent modal shape (Valles, 1996) (belonging to Group 2):

$$F_i = \frac{m_i \cdot \phi_{eq,i}}{\sum_{j=1}^{N} m_j \cdot \phi_{eq,j}} V_b \quad dove \quad \phi_{eq,i} = \sqrt{\sum_{m=1}^{N} \left(\phi_i^m \cdot \Gamma_m\right)^2}$$

- distribution proportional to the masses (belonging to Group 2):

$$F_i = \frac{m_i}{\sum_{j=1}^{N} m_j} V_b$$

- adaptive load distribution (belonging to Group 2), continuously updated during the analysis, to reflect the progressive stiffness degradation of the structure. This distribution considers the contribution of multiple modes of vibration, updated at each load step (*Adaptive Analysis* module):

$$F_i = \sqrt{\sum_n \sum_m \rho_{n,m} \cdot F_{i,n} \cdot F_{i,m}}$$

where

$$F_{ij} = \Gamma_j \cdot \phi_{i,j} \cdot M_i \cdot S_a\left(T_j, \xi\right)$$

$$\rho_{n,m} = \frac{8\xi^2 \beta_{i,j}^{3/2}}{\left(1+\beta_{i,j}\right)\cdot\left[\left(1-\beta_{i,j}\right)^2 + 4\cdot\xi^2\cdot\beta_{i,j}\right]}$$

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

where

$$F_{ij} = \Gamma_j \cdot \phi_{i,j} \cdot M_i \cdot S_a(T_j, \xi)$$

$$\rho_{n,m} = \frac{8\xi^2 \beta_{i,j}^{3/2}}{(1+\beta_{i,j}) \cdot \left[(1-\beta_{i,j})^2 + 4 \cdot \xi^2 \cdot \beta_{i,j}\right]}$$

being

i = node index; j, n, m = index of vibration mode

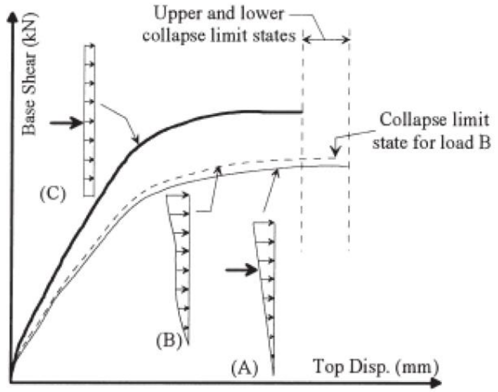
ξ = damping factor for vibration modes

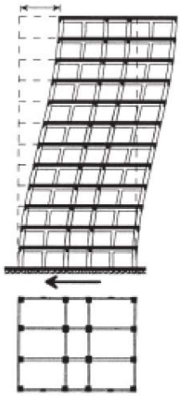
$\beta_{i,j} = T_j/T_i$ = inverse ratio between the periods of each i-j pair of modes

- adaptive load distribution (belonging to Group 2), continuously updated during the analysis, in equilibrium with the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (*Adaptive Analysis* module)

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

The resultant of the uniform distribution corresponds to the lowest point of application and therefore the maximum resistance and the lower yield and limit state of collapse displacements.
The resultant of the triangular distribution is applied to the highest point and presents, on the other hand, the lower resistance and the greater yield and the collapse limit state displacements.
The choice of considering two different distributions arises from the consideration that the distribution of the lateral forces should approximate the distribution of the forces of inertia during the earthquake. Comparisons with nonlinear dynamic analyses have shown that force distributions proportional to first vibrational mode better capture the dynamic response as long as the structure remains in the elastic field, whereas, when large deformations are reached, the dynamic response can be better represented by distributions of forces proportional to the masses.





LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

In the adaptive analysis, the loads applied to the structure are updated at each stage, according to the secant stiffness of the elements (beam, plates and bricks) at the stage considered. The procedure can be summarized in the following steps:

- calculation of modal vectors: $F_{ij} = \Gamma_j \cdot \phi_{i,j} \cdot M_i \cdot S_a(T_j, \xi)$

  being
  i = node index; j, n, m = index of vibration mode

  M_i mass of the i-th node of the structure
  $\phi_{i,j}$ = displacement of the i-th node in the j-th mode
  Γ_j is the "modal participation factor" defined by the relation:

  $$\Gamma_j = \frac{\phi_j^T M \tau}{\phi_j^T M \phi_j^T}$$ where τ it is the drag vector corresponding to the direction of the considered seismic action

  T_j period of the j-th vibration mode
  ξ = damping factor for vibration modes
  S_a = horizontal elastic acceleration response spectrum

- combination of modes and calculation of incremental load vector:

  $$F_i = \sqrt{\sum_n \sum_m \rho_{n,m} \cdot F_{i,n} \cdot F_{i,m}}$$

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

where

$$\rho_{n,m} = \frac{8\xi^2 \beta_{i,j}^{3/2}}{(1+\beta_{i,j})\cdot\left[(1-\beta_{i,j})^2 + 4\cdot\xi^2\cdot\beta_{i,j}\right]}$$

being

i = node index; j, n, m = index of vibration mode

$\beta_{i,j} = T_j/T_i$ = inverse ratio between the periods of each i-j pair of modes

- normalization of the incremental load vector:

$$\overline{F} = \frac{F}{\sum F_i}$$

- increase of the incremental load vector:

$$P_k = P_{k-1} + \Delta\lambda_k P_0 \overline{F_k}$$

being

k = load step index

PUSHOVER ANALYSIS
LATERAL LOAD DISTRIBUTIONS

ENGIN SOFT HSH

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

- adaptive load distribution (belonging to Group 2), continuously updated during the analysis, in equilibrium with the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (*Adaptive Analysis* module)

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An adaptive modal pushover procedure for asymmetric-plan buildings

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LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

The maximum induced modal forces and torques in each mode, because of excitation in one direction (the *y*-direction) is computed by Eqs. (11)–(13). These loads are applied in the mass center of the stories:

$$F_{x_{ij}} = \Gamma_{y_j}\phi_{x_{ij}} m_{x_i} S_{a_{yj}} \tag{11}$$

$$F_{y_{ij}} = \Gamma_{y_j}\phi_{y_{ij}} m_{y_i} S_{a_{yj}} \tag{12}$$

$$T_{\theta_{ij}} = \Gamma_{y_j}\phi_{\theta_{ij}} I_{\theta_i} S_{a_{yj}} \tag{13}$$

where i is the story number, j is the mode number, m_{x_i} is the translational mass of the ith story in x direction, m_{y_i} is the translational mass of the ith story in y direction, I_{θ_i} is the rotational mass of the ith story and $S_{a_{yj}}$ is the spectral acceleration in y direction corresponding to the jth mode.

In the well-known spectral-dynamic-analysis (SDA) the maximum total responses of the structure is approximated by the combination of responses due to maximum modal forces [Eqs. (11)–(13)]. Chopra and Goel [25] have extended this concept to nonlinear analysis by proposing the MPA procedure.

profile of the structure. The story drift of 3D structures consist of the translational and torsional relative displacements and these parameters are directly related to the story shears and total story torques, therefore in the proposed STA procedure the load vector associated with the combined modal story shear and torque profile is selected as the load pattern which would be the best representative of the damage in structures. The process of defining the load pattern at each step of the STA procedure is illustrated through a sample nine-story building in Fig. 1. This sample building is asymmetric about *x*-axis and *y*-axis (two horizontal directions) which is subjected to horizontal ground motion in one direction (the *y*-direction).

At each step, based on the instantaneous structural properties, the modal story forces in two translational (the *x* and *y*) directions and modal torque applied in the mass centers of stories, are calculated for each considered modes by Eqs. (11)–(13) [Fig. 1(a)].

The story shears in translational directions and story torque associated with each mode are calculated by Eqs. (14)–(16) [Fig. 1(b), (c) and (d)]. Then the combined modal story shears and combined

PUSHOVER ANALYSIS
LATERAL LOAD DISTRIBUTIONS

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

modal story torques are calculated using the complete-quadratic-combination (CQC) or square-root-of-the-sum-of-the-squares (SRSS) rule [Eqs. (17)–(19), Fig. 1(e), (f) and (g)].

$$SS_{x_{ij}} = \sum_{k=i}^{n} F_{x_{kj}} \tag{14}$$

$$\Rightarrow \quad CSS_{x_i} = \sqrt{\sum_{j=1}^{m} SS_{x_{ij}}^2} \tag{17}$$

$$SS_{y_{ij}} = \sum_{k=i}^{n} F_{y_{kj}} \tag{15}$$

$$\Rightarrow \quad CSS_{y_i} = \sqrt{\sum_{j=1}^{m} SS_{y_{ij}}^2} \tag{18}$$

$$ST_{\theta_{ij}} = \sum_{k=i}^{n} T_{\theta_{kj}} \tag{16}$$

$$\Rightarrow \quad CST_{\theta_i} = \sqrt{\sum_{j=1}^{m} ST_{\theta_{ij}}^2} \tag{19}$$

where $SS_{x_{ij}}$ and $SS_{y_{ij}}$ are the story shears in floor i associated with mode j in x and y directions, respectively. $ST_{\theta_{ij}}$ is the story torque in floor i associated with mode j. CSS_{x_i} and CSS_{y_i} are the combined modal story shears in floor i associated with all the modes considered. CST_{θ_i} is the combined modal story torque in floor i associated with all the modes considered.

The lateral forces (in two translational directions) and torques required to generate CSS_{x_i} and CSS_{y_i} (the combined modal story shears) and CST_{θ_i} profiles are assumed as the load pattern. The required story forces in each direction and the required story torque are calculated by subtracting the combined modal story shears and combined modal story torques of consecutive stories using Eqs. (20)–(22) [Fig. 1(h), (i) and (j)]:

$$\begin{cases} F_{x_i} = CSS_{x_i} - CSS_{x_{i-1}} & i < n \\ F_{x_n} = CSS_{x_n} & i = n \end{cases} \tag{20}$$

$$\begin{cases} F_{y_i} = CSS_{y_i} - CSS_{y_{i-1}} & i < n \\ F_{y_n} = CSS_{y_n} & i = n \end{cases} \tag{21}$$

$$\begin{cases} T_{\theta_i} = CST_{\theta_i} - CST_{\theta_{i-1}} & i < n \\ T_{\theta_n} = CST_{\theta_n} & i = n \end{cases} \tag{22}$$

By applying these loads (Eqs. (20)–(22)) in the mass center of each story, the resulted story shear and torque would be equal to the combined modal story shears and torque in each story. Consequently, the

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

induced drift in each story would be equal to the combined modal story drift in linear-elastic structures. Therefore the authors believe that these loads (Eqs. (20)–(22)) could be an efficient load pattern in each step of pushover analysis of nonlinear structures.

The components of this load pattern are normalized with respect to the summation of the force components (base shear) in the earthquake excitation direction (the *y*-direction) using Eqs. (23)–(25). The incremental applied load profile at each step is computed by Eqs. (26)–(28):

$$\overline{F}_{x_i} = \frac{F_{x_i}}{\sum F_{y_i}} \tag{23}$$

$$\overline{F}_{y_i} = \frac{F_{y_i}}{\sum F_{y_i}} \tag{24}$$

$$\overline{T}_{\theta_i} = \frac{T_{\theta_i}}{\sum F_{y_i}} \tag{25}$$

$$\Delta\overline{F}_{x_i} = \Delta V_{b_y} \times \overline{F}_{x_i} \tag{26}$$

$$\Delta\overline{F}_{y_i} = \Delta V_{b_y} \times \overline{F}_{y_i} \tag{27}$$

$$\Delta\overline{T}_{\theta_i} = \Delta V_{b_y} \times \overline{T}_{\theta_i} \tag{28}$$

where ΔV_{b_y} is the incremental amount of the base shear in the earthquake excitation direction (the *y*-direction).

Even though the quadratic combination rule (e.g. SRSS) is used in defining the load pattern, the calculated applied forces or torque in each story could be negative whenever the value of the combined modal shears or torque in one story is less than the one in the upper story [Fig. 1(e), (f) and (g)]. Therefore, this method is able to simulate the force or torque distribution profiles with reversal signs as observed in dynamic analysis [Fig. 1(h), (i) and (j)]. So the effects of the reversal signs in the higher modes are also simulated in the proposed load pattern.

PUSHOVER ANALYSIS
LATERAL LOAD DISTRIBUTIONS

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER

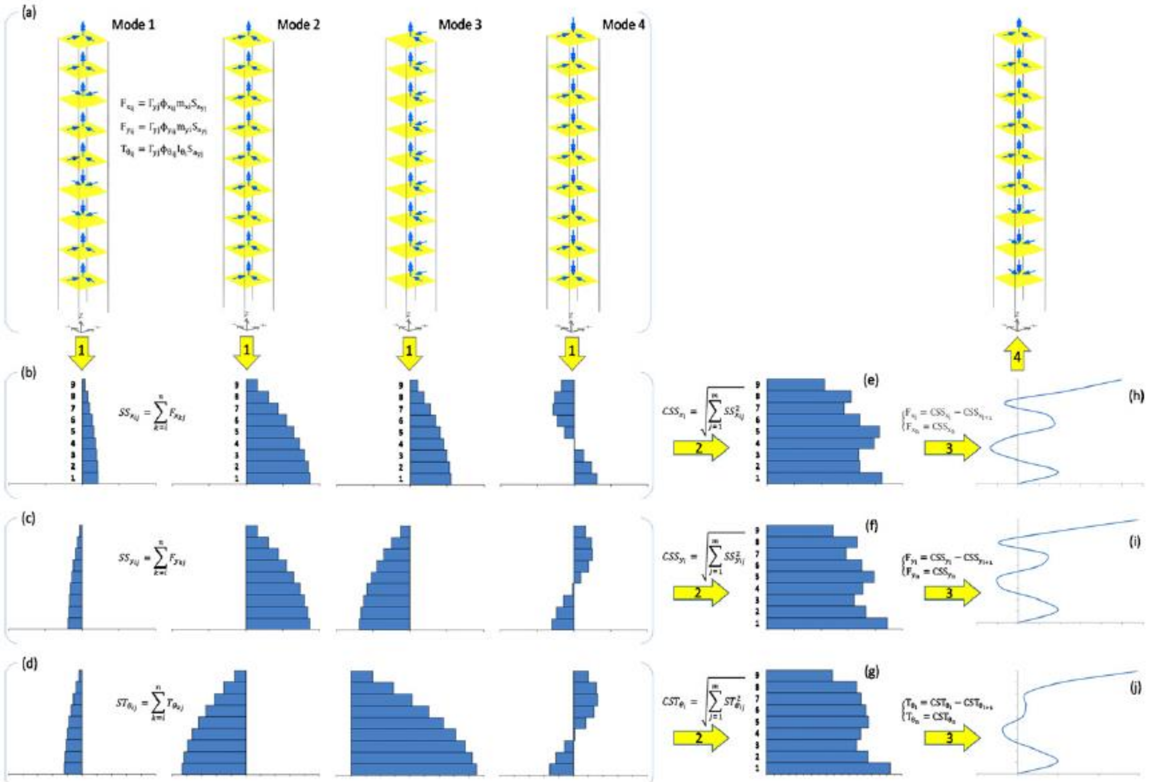
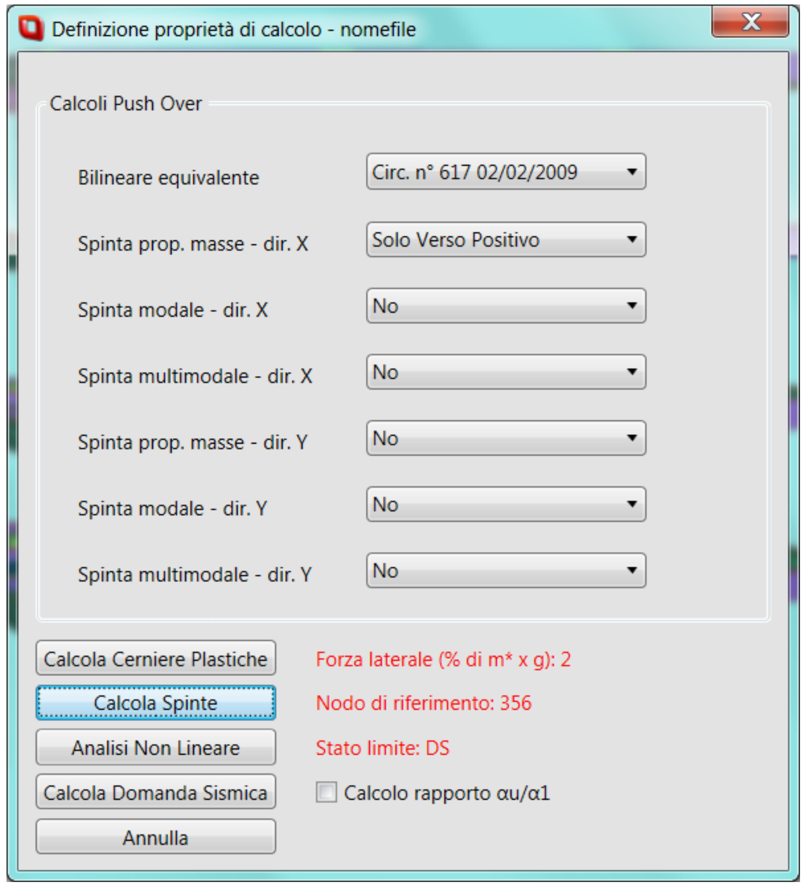


Fig. 1. The process of defining the load pattern at each step of the STA procedure. (a) Modal story forces in each mode. (b–d) Modal story shears and torques. (e–g) Combined modal story shear and torques. (h–j) Component of the proposed load pattern (x, y and θ direction) in each story.

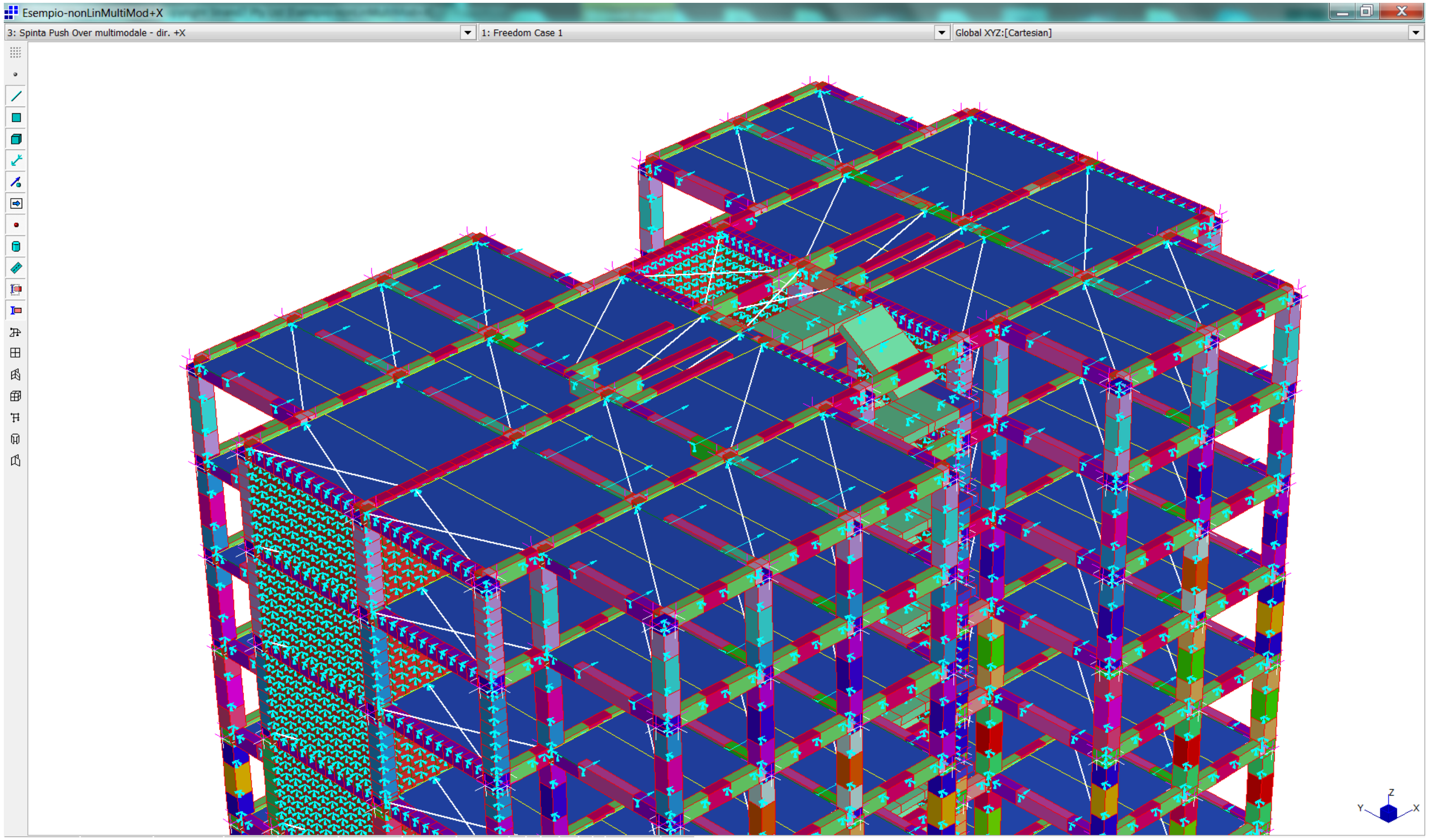
PUSHOVER ANALYSIS
LATERAL LOAD DISTRIBUTIONS

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER



PUSHOVER ANALYSIS
LATERAL LOAD DISTRIBUTIONS

LATERAL LOAD DISTRIBUTIONS ADOPTED IN EASYOVER



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