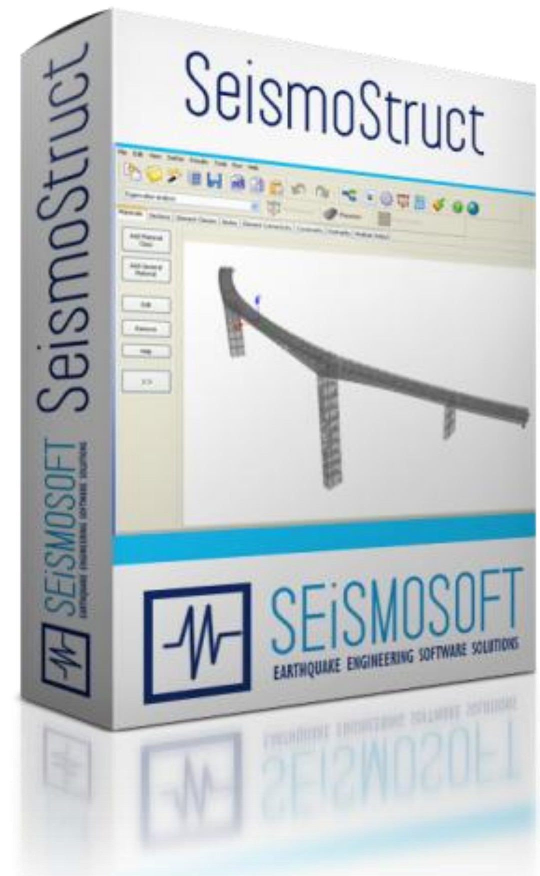
موسسه آموزشی و مهندسی ۸۰۸
آموزشهای تخصصی عمران و معماری

معرفی تغییرات نسخه ۲۰۲۰ نرم افزار SeismoStruct

محمدرضا سقراط

دکترای مهندسی عمران-مهندسی زلزله



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- معرفی مصالح تعریف شده در برنامه و تفاوت های موجود در ورژن ۲۰۲۰ با ورژنهای قبلی
- معرفی روشهای تحلیل در برنامه و تغییرات ایجاد شده در ورژن جدید مانند اضافه شدن آنالیز کمانش و تحلیل مقادیر ویژه در هر گام در تحلیل دینامیکی غیرخطی و یا پوش اور
- معرفی مقاطع اضافه شده در ورژن ۲۰۲۰ مانند مقاطع اضافه شده در سازه های بنایی
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- معرفی آیین نامه های سازه‌ای جدید
- مدلسازی یک سازه قاب خمشی و انجام آنالیز پوش آور

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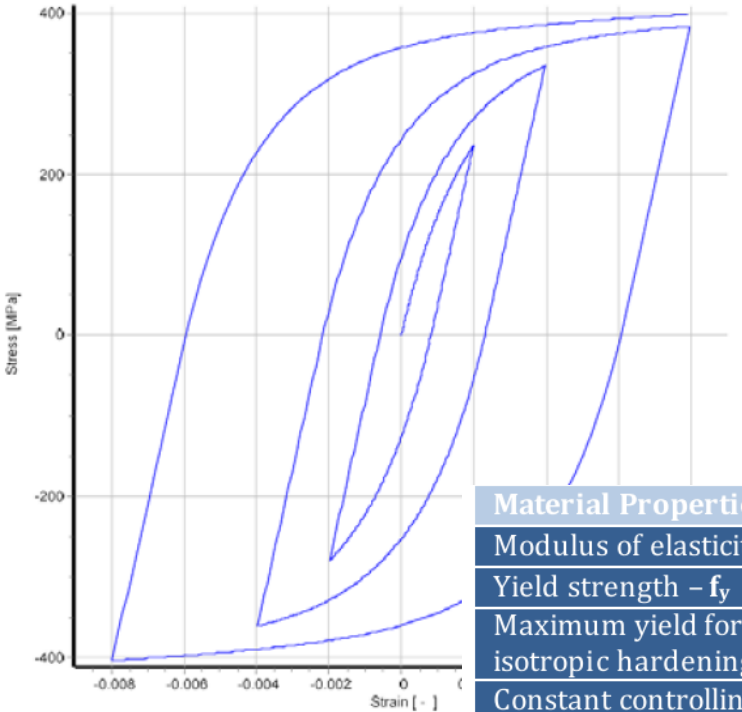
Materials

- Bilinear steel model - stl_bl
- Menegotto-Pinto steel model - stl_mp
- Bilinear steel model with isotropic strain hardening- stl_bl2
- Giuffre-Menegotto-Pinto Model with Isotropic Hardening – stl_gmp
- Ramberg-Osgood steel model - stl_ro
- Dodd-Restrepo steel model – stl_dr
- Monti-Nuti steel model - stl_mn
- Buckling Restrained steel brace model – stl_brb
- Mander et al. nonlinear concrete model - con_ma
- Trilinear concrete model - con_tl
- Chang-Mander nonlinear concrete model – con_cm
- Kappos and Konstantinidis nonlinear concrete model - con_hs
- Engineered cementitious composites material– con_ecc
- Kent-Scott-Park concrete model – con_ksp
- Trilinear masonry model - mas_tl
- Parabolic masonry model - mas_par
- Superelastic shape-memory alloys model - se_sma
- Trilinear FRP model - frp_tl

Buckling Restrained steel brace model - stl_brb

Stl_BRB is a uniaxial steel material model describing the behaviour of steel in Bucking Restrained Braces. The model has been presented by Zona et al. [2012]



Buckling restrained steel b

Material Properties	Typical values	Default values
Modulus of elasticity – $\mathbf{E_s}$	2.00E+08 - 2.10E+08 (kPa)	2.00E+08 (kPa)
Yield strength – $\mathbf{f_y}$	260000 - 420000 (kPa)	37000 (kPa)
Maximum yield force for fully saturated isotropic hardening for Tension – $\mathbf{f_{y,max\ T}}$	400000 - 600000 (kPa)	492000 (kPa)
Constant controlling elastic to plastic transition for tension– $\boldsymbol{\alpha_{,T}}$	0.2-1.2	0.9
Hardening ratio for tension – $\mathbf{b_{,T}}$	0.01-0.04	0.01
Constant controlling isotropic hardening for tension – $\boldsymbol{\delta_{,T}}$	0.1-0.8	0.2
Maximum yield force for fully saturated isotropic hardening for compression – $\mathbf{f_{y,max\ c}}$	400000 - 600000 (kPa)	581000
Constant controlling elastic to plastic transition for compression – $\boldsymbol{\alpha_{,C}}$	0.2-1.2	0.9
Hardening ratio for compression – $\mathbf{b_{,C}}$	0.01-0.04	0.01
Constant controlling isotropic hardening for compression – $\boldsymbol{\delta_{,C}}$	0.1-0.8	0.2
Specific weight - $\boldsymbol{\gamma}$	78 (kN/m^3)	78 (kN/m^3)

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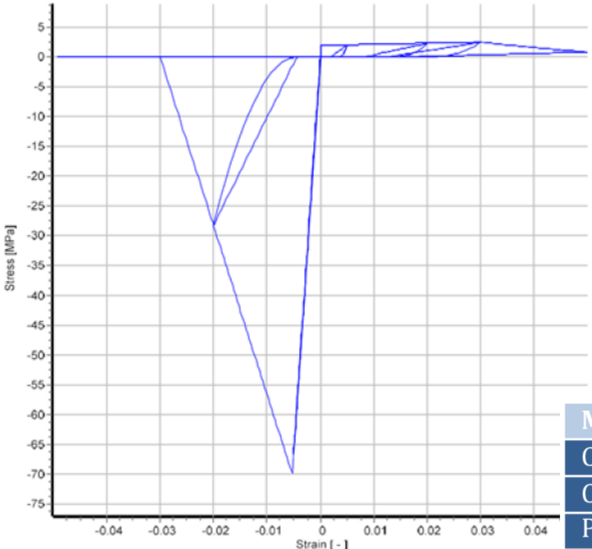
Engineered cementitious composites material – con_ecc

Con_ecc is a uniaxial generic material modeling the behavior of ductile fiber-reinforced cement-based composites as described by Han et al. [2003]. The model needs 13 variables for its definition.

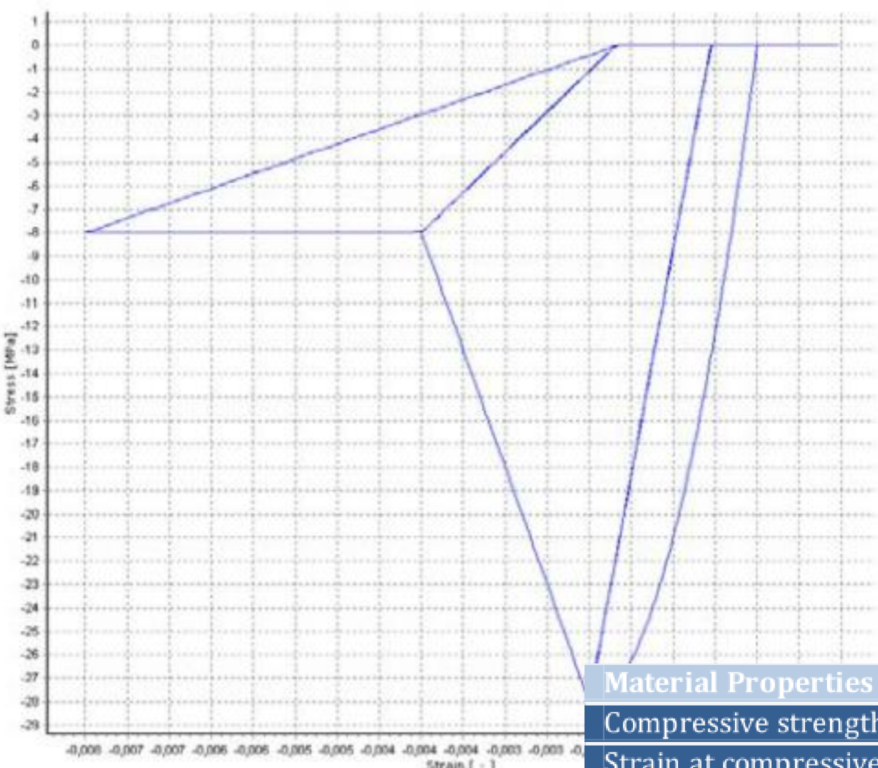


Ductile fiber – reinforcd cement based composite

Material Properties	Typical values	Default values
Cracking stress – $\sigma_{t,0}$	2.00E+03 – 4.00E+03 (kPa)	2.00E+03 (kPa)
Cracking strain – $\varepsilon_{t,0}$	0.000015 – 0.00025	0.00015
Peak tensile stress – $\sigma_{t,p}$	2.50E+03 – 6.00E+03 (kPa)	2.50E+03 (kPa)
Strain at peak tensile stress – $\varepsilon_{t,p}$	0.03-0.037	0.03
Ultimate tensile strain – $\varepsilon_{t,u}$	0.06	0.06
Compressive strength – $\sigma_{c,p}$	70.00E+03 – 80.00E+03 (kPa)	70.00E+03 (kPa)
Strain at compressive strength– $\varepsilon_{c,p}$	0.005-0.006	0.00525
Ultimate compressive strain – $\varepsilon_{c,u}$	0.01-0.03	0.03
Power for tensile unloading curve – a_t	≥1	5
Power for compressive unloading curve – a_c	≥1	2
Factor for tensile unloading curve – **bt**	>0	0.4
Factor for compressive unloading curve – **bc**	>0	0.3
Specific weight - γ	24 (kN/m^3)	24 (kN/m^3)

Kent-Scott-Park concrete model – *con_ksp*

The con_ksp is a simplified uniaxial concrete model with a stress-strain relationship described by Kent and Park [1971] and a cyclic behaviour proposed by Karsan and Jirsa [1969]. The model is characterized by zero tensile strength. Five variables are needed for the definition of the model.

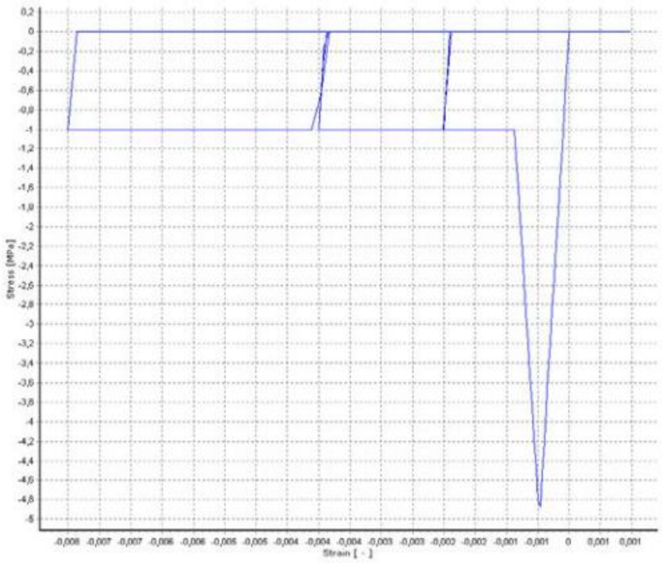


Kent – Scott – Park concret

Material Properties	Typical values	Default values
Compressive strength – σ_c	15000 - 45000 (kPa)	28000 (kPa)
Strain at compressive strength – ε_c	0.002 - 0.0022 [-]	0.002
Residual strength – σ_c	5000 - 15000 (kPa)	8000 (kPa)
Strain at residual strength– ε_c	-	0.004
Specific weight - γ	24 (kN/m^3)	24 (kN/m^3)

Trilinear masonry model - mas_tl

This is a simplified uniaxial trilinear material model that assumes no resistance to tension and features a residual strength plateau.



Trilinear masonry model

Material Properties	Typical values	Default values
Mean Compressive strength – f_{c1}	1000 - 10000 (kPa)	5000 (kPa)
Initial stiffness – E_1	5.0E+06 - 2.00E+07 (kPa)	1.05E+07 (kPa)
Post-peak stiffness – E_2	-2.50E+06 - -3.00E+07 (kPa)	-1.00E+07 (kPa)
Residual strength – f_{c2}	500 - 5000 (kPa)	1000 (kPa)
Specific weight – γ	24 (kN/m^3)	24 (kN/m^3)

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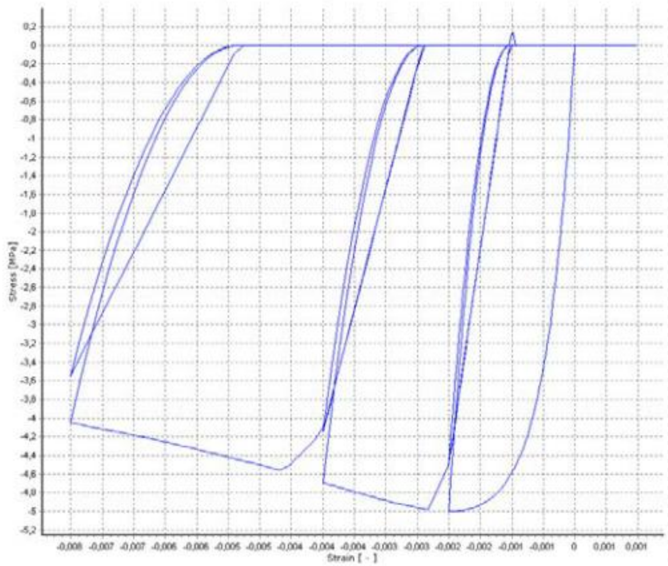
Parabolic masonry model - mas_par

This is a uniaxial nonlinear material model for masonry that is based on the hysteretic rules of the con_ma typical constant confinement concrete model.



Parabolic Masonry model

Material Properties	Typical values	Default values
Mean Compressive strength – f_c	1000 - 10000 (kPa)	5000 (kPa)
Mean Tensile strength – f_t	0 - 2000 (kPa)	500 (kPa)
Modulus of Elasticity – E_c	5000 - 20000 (MPa)	1.05E+07 (kPa)
Poisson Ratio	0.10-0.30 (-)	0.20 (-)
Strain at peak stress – ε_c	0.002 - 0.0022 (m/m)	0.002 (m/m)
Specific weight – γ	24 (kN/m^3)	24 (kN/m^3)

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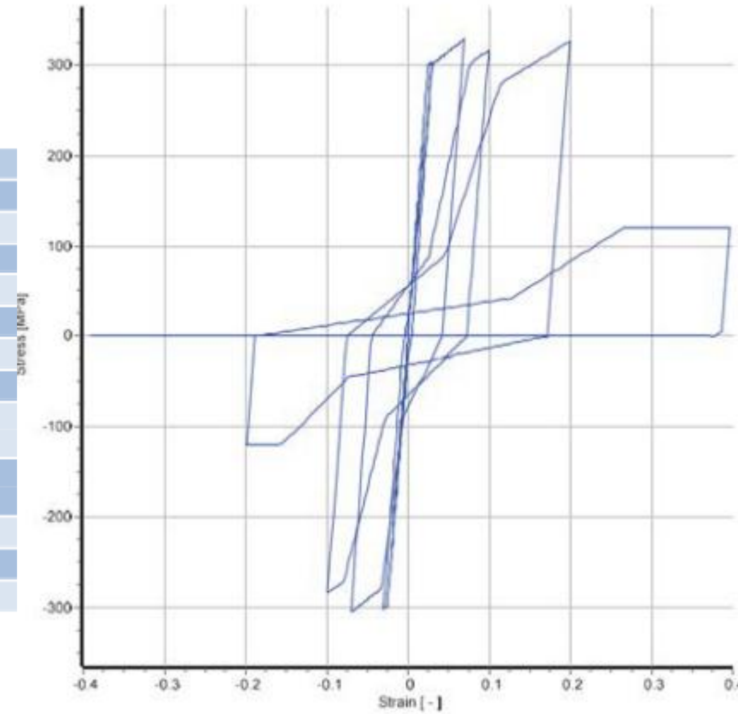
Generic Hysteretic material - hyst_mat

Hyst_mat is a generic uniaxial material model characterised by Pinching effect (controlled by a pinching factor with values from 0 for no pinching to 1 for high pinching) and by four deterioration modes including strength deterioration, peak stress deterioration, reloading and unloading stiffness deterioration. All deterioration modes are controlled by the deterioration factor (with values from 0 for no deterioration to 1 for high deterioration). An initial backbone curve consisting of the yielding point, the peak stress point and the reidual strength is initialy defined and deteriorates after each unloading incident.

Material Properties	Typical values	Default values
Modulus of Elasticity - E_s	-	2.00E+08 (kPa)
Yield Stress in positive direction – $\sigma_{y,pos}$	-	300E+03 (kPa)
Yield Stress in negative direction – $\sigma_{y,neg}$	-	300E+03 (kPa)
Peak Strain in positive direction – $\varepsilon_{peak,pos}$	-	300
Peak Stress in positive direction – $\sigma_{peak,pos}$	-	500 (kPa)
Peak Strain in negative direction – $\varepsilon_{peak,neg}$	-	300
Peak Stress in negative direction – $\sigma_{peak,neg}$	-	500 (kPa)
Residual Strength in positive direction – $\sigma_{Res,pos}$	-	120 (kPa)
Residual Strength in negative direction – $\sigma_{Res,pos}$	-	120 (kPa)
Pinching factor – **FPinch**	0-1	0.5
Deterioration factor – **FDet**	0-1	0.5
Specific weight - γ	-	24 (kN/m^3)



Generic Hysteretic material model

Sections

- Rectangular hollow section - rhs
- Circular solid section - css
- Circular hollow section - chs
- Symmetric I-or T- section - sits
- Asymmetric general-shape section - agss
- Double angle or channel shaped section – dacss
- Double I type1 section – di1
- Double I type2 section – di2
- Double I type1 section with top and bottom plates– di1tbp
- Double I type2 section with top and bottom plates – di2tbp
- Double I type1 section with web plates – di1wp
- Double I type2 section with web plates– di2wp
- Double I type1 section with top, bottom and web plates – di1tbwp
- Double I type2 section with top, bottom and web plates – di2tbwp
- Built up box double channel section – bbdc
- Built up box double channel section with connecting plate - bbdccp
- Built up box double channel section with top and bottom plates - bbdctbp
- Built up box double angle section – bbda
- Built up box double angle section with connecting plate – bbdacp

- I section with top and bottom plates – itbp
- I section with top, bottom and web plates – itbwp
- I section with top plate – itp
- I section with bottom plate – ibp
- I section reinforced with bottom I section – ibri
- I section reinforced with bottom T section – ibrt
- Star section composed from angle sections – sfa
- Double angle back-to-back section – dabtb
- Built up box formed by four angle sections – bbfa
- Double angle section placed along the diagonal – dadg
- Cruciform Section - cfs
- Reinforced concrete rectangular section - rcrs
- Reinforced concrete quadrilateral section - rcqs
- Reinforced concrete rectangular with rounded corners section - rcrrcs
- Reinforced concrete circular section - rccs
- Reinforced concrete Z-shaped column section – rczcs
- Reinforced concrete L-shaped column section – rclcs
- Reinforced concrete T-shaped column section – rctcs
- Reinforced concrete T-section - rcts

- Reinforced concrete asymmetric rectangular section - rcars
- Reinforced concrete rectangular wall section – rcrws
- Reinforced concrete rectangular no pseudo-columns wall section - rcbws
- Reinforced concrete U-shaped wall section - rcuws
- Reinforced concrete Z-shaped wall section - rczws
- Reinforced concrete L-shaped wall section - rclws
- Reinforced concrete rectangular hollow section - rcrhs
- Reinforced concrete rectangular with rounded corners hollow section - rcrrchs
- Reinforced concrete circular hollow section - rcchs
- Reinforced concrete box-girder section - rcbgs
- Reinforced concrete jacketed rectangular section – rcjrs
- Reinforced concrete jacketed rectangular with rounded corners section – rcjrrcs
- Reinforced concrete 3-side jacketed rectangular section – rcjrs3
- Reinforced concrete 2-side jacketed rectangular section – rcjrs2
- Reinforced concrete 1-side jacketed rectangular section – rcjrs1

- Reinforced concrete jacketed circular section – rcjcs
- Reinforced concrete jacketed Z-shaped column section – rcjzcs
- Reinforced concrete jacketed L-shaped column section – rcjlcs
- Reinforced concrete 3-side jacketed L-shaped column section – rcjlcs3
- Reinforced concrete jacketed T-shaped column section – rcjtcs
- Reinforced concrete 3-side jacketed T-shaped column section – rcjtcs3
- Reinforced concrete jacketed T-section – rcjts
- Reinforced concrete 3-side jacketed T-section – rcjts3
- Reinforced concrete 1-side jacketed T-section – rcjts1
- Reinforced concrete jacketed asymmetric rectangular section - rcjars
- Reinforced concrete 1-side jacketed asymmetric rectangular section – rcjars1
- Composite I-section - cpis
- Partially encased composite I-section - pecs
- Fully encased composite I-section - fecs
- Composite rectangular section - crs
- Composite circular section – ccs
- Masonry wall section – mws
- Masonry spandrel section - mss

Masonry wall section - mws

This is a section frequently adopted for the modelling of masonry members.

Materials and Dimensions

Two different materials can be defined:

- Reinforcement,
- Masonry

The required dimensions are as follows:

- Wall width (in-plane dimension). The default value is 2.0 m
- Wall thickness (out-of-plane dimension). The default value is 0.4 m
- Cover Thickness. The default value is 0.025 m

Reinforcement

Reinforcement bars can be defined in two different ways:

1. By editing the reinforcement pattern;
2. By entering the respective area and sectional coordinates (the latter being defined in the local coordinate system of the section).

Masonry spandrel section - mss

This is a section frequently adopted for the modelling of coupling beams bertween two wall elements.

Materials and Dimensions

Two different materials can be defined:

- Reinforcement,
- Masonry

The required dimensions are as follows:

- Spandrel height (in-plane dimension). The default value is 0.6 m
- Spandrel thickness (out-of-plane dimension). The default value is 0.4 m
- Cover Thickness. The default value is 0.025 m

Reinforcement

Reinforcement bars can be defined in two different ways:

1. By editing the reinforcement pattern;
2. By entering the respective area and sectional coordinates (the latter being defined in the local coordinate system of the section).

Cruciform Section – cfs

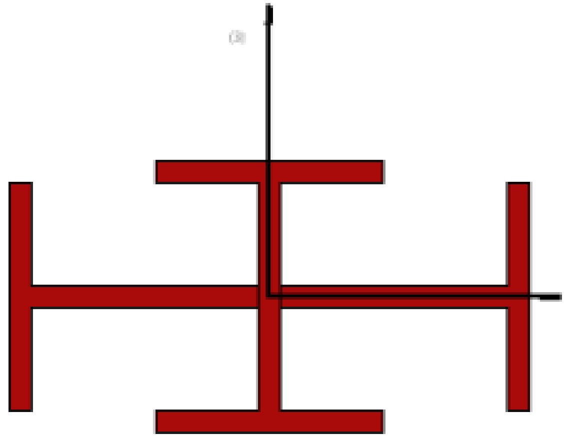
This is a section frequently adopted for the modelling an I section with two T sections connected symmetrically on its web forming a cross section with flanges.

<u>Materials and Dimensions</u>

Two different materials (steel) can be defined:

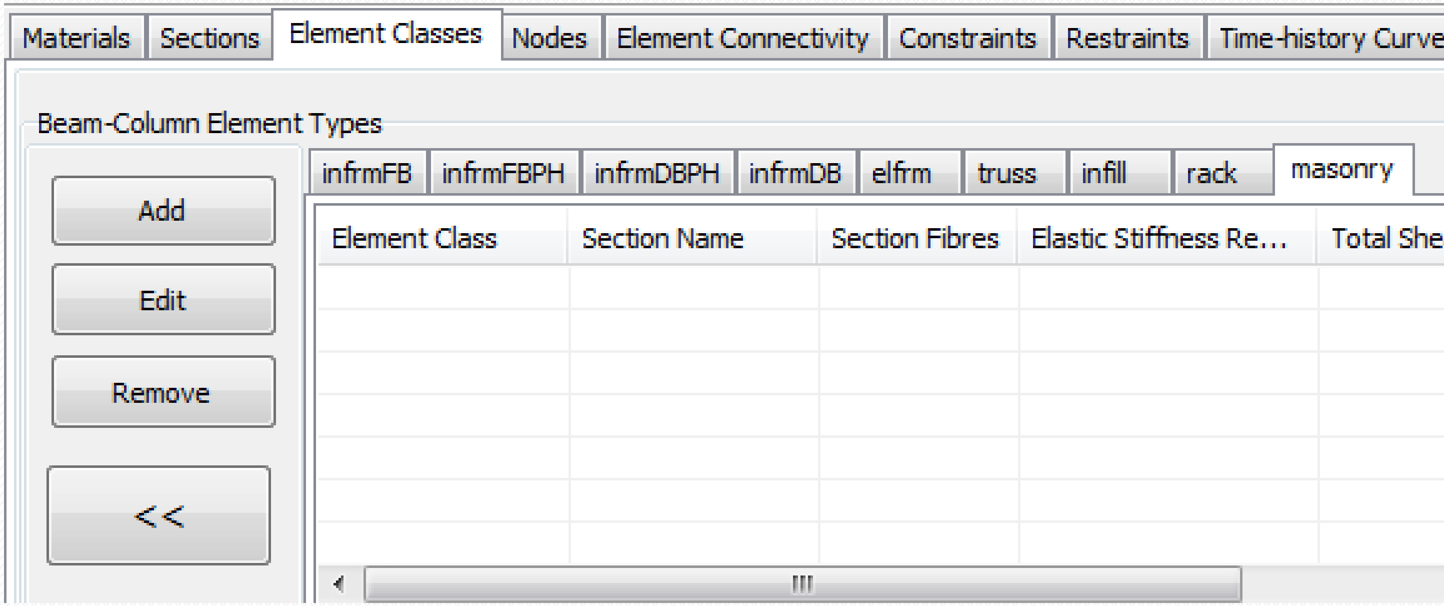
- Steel profile (I Section),
- Steel profile (Side T Sections).

The required dimensions are as follows:

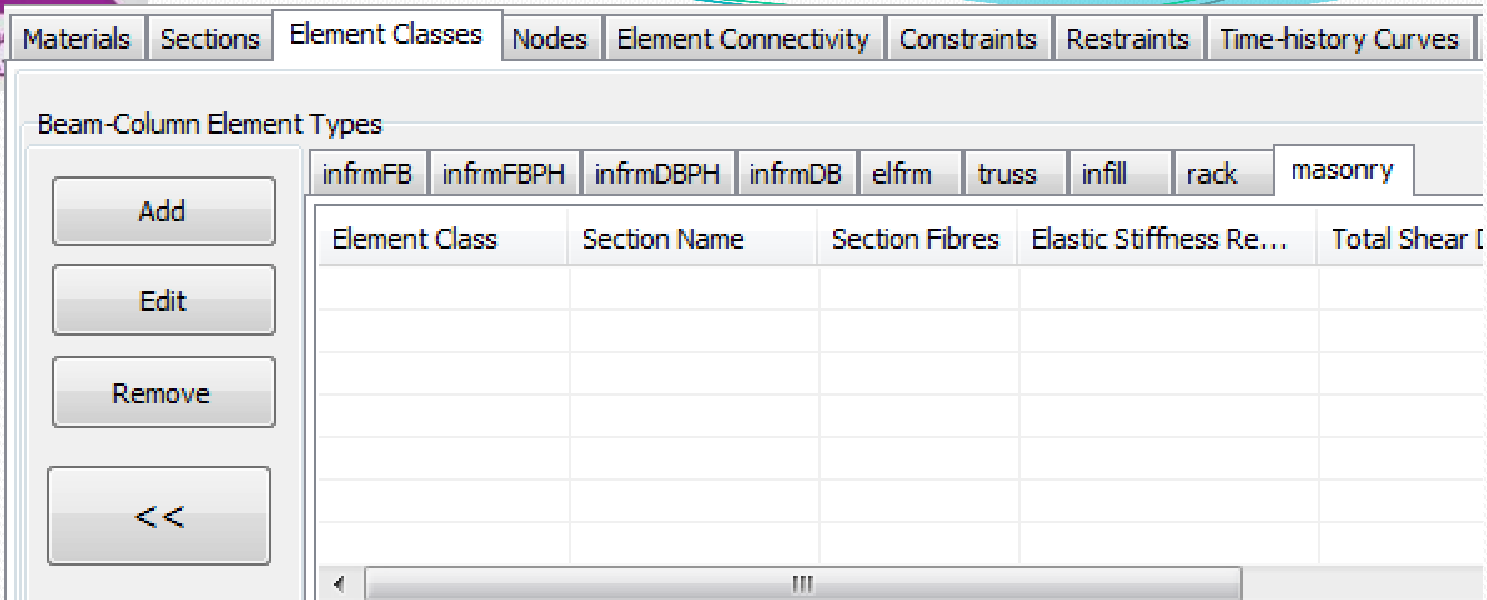


- I Section Web Height. The default value is 0.2 m
- I Section Web Thickness. The default value is 0.02 m
- I Section Flange Width. The default value is 0.2 m
- I Section Flange Thickness. The default value is 0.02 m
- T Section Web Height. The default value is 0.2 m
- T Section Web Thickness. The default value is 0.02 m
- T Section Flange Width. The default value is 0.2 m
- T Section Flange Thickness. The default value is 0.02 m

Element types



- Masonry Element
- 7-DOF Rack Element
- New infrmDBPH element type with better stability and automatic calculation of the plastic rotation capacity values according to ASCE 41-17



infrmFB: Inelastic force-based frame element
infrmFBPH: Inelastic plastic-hinge force-based frame element
infrmDBPH: Inelastic plastic-hinge displacement-based frame element
infrmDB: Inelastic displacement-based frame element
elfrm: Elastic frame element
truss: Inelastic truss element
infill: Inelastic infill panel element
rack: Rack element
masonry: Inelastic masonry frame element

Masonry element type

This element is combination of a 3D, force-based, plastic hinge element type employed in modelling mainly the bending behaviour of the masonry member (herein mentioned as the 'internal sub-element') with two links at the two edges that are employed to simulate the shear behaviour of the member (herein referred to as the 'external links' or the 'link sub-elements'). The internal sub-element and the external links are connected in series, ensuring equilibrium in bending moment and shear force. The only 'active' degrees-of-freedom of the link sub-elements are the two translational ones in the shear directions (in-plane and out-of-plane), whilst the other four DOFs (axial and 3 rotational) remain perfectly rigid links. Both masonry walls and spandrels can be accurately modelled with such configuration.

NOTE: The masonry element type can only be used with the mas_par and the mas_tl material models. Similarly, only the special masonry section types mws and mss can be employed. The reason for this, is that these can store parameters that are used for the automatic calculation of the member shear strength.

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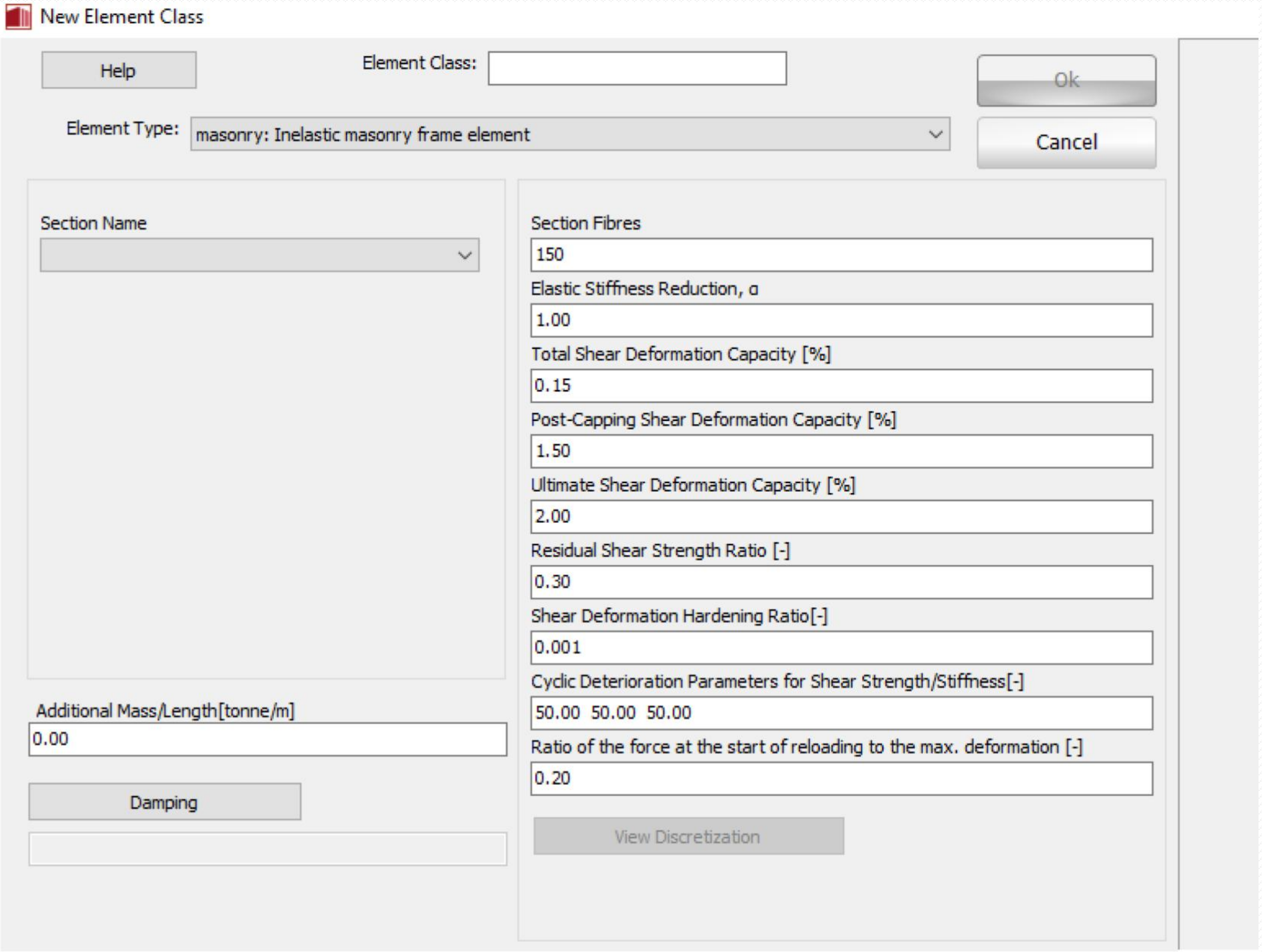
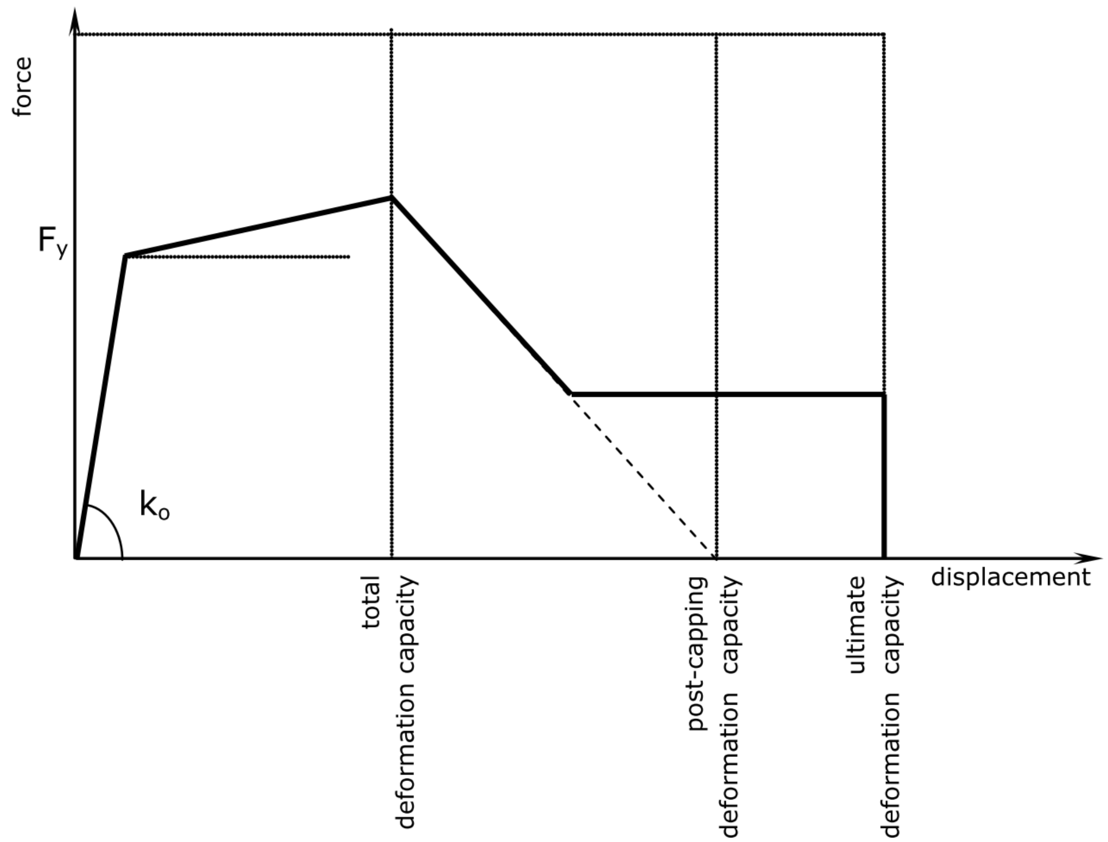


Definition of a new masonry element

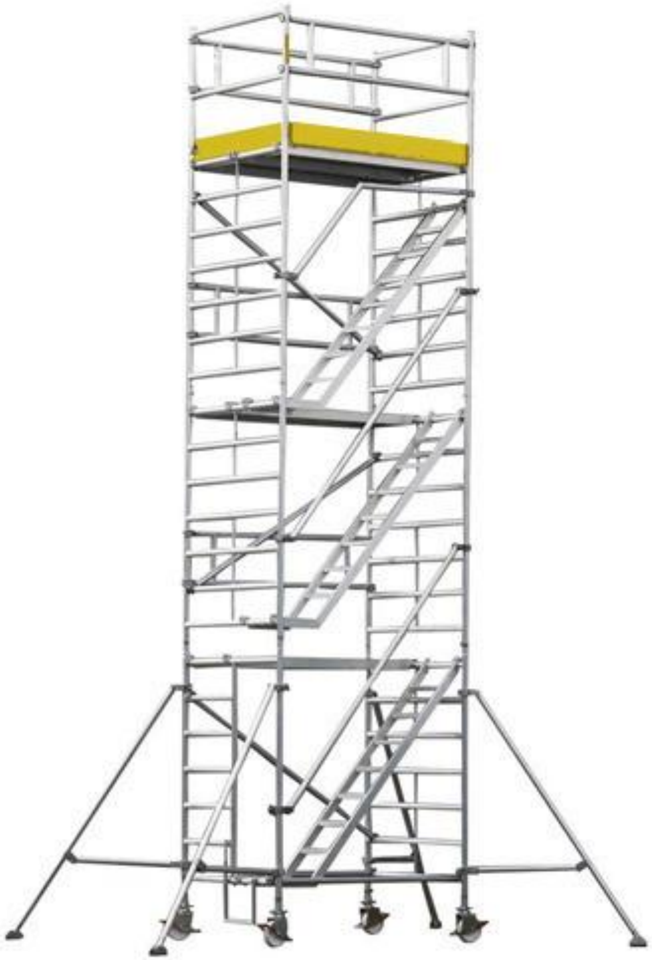


Shear force-deformation curve employed in the masonry model

RACK ELEMENT TYPE

This element is a 3D beam element with thin-walled, open, cross-sections. The element is characterized by seven degrees of freedom per node, so that to correctly estimate both the displacements and the internal stresses, including warping displacements and bi-moment stresses, and to correctly predict the flexural-torsional and lateral-torsional buckling, derived by the coupling between flexure and torsion. Furthermore, the model accounts for the eccentricity of the shear centre from section centroid, and it considers all the Wagner coefficients, which makes it suitable for use with non-symmetric cross-sections.

As a result, the formulation is ideal for the modelling of steel storage pallet racks (in the field of logistic), as well as scaffoldings, which are generally composed by uprights which have mono-symmetric lipped channel cross-sections.

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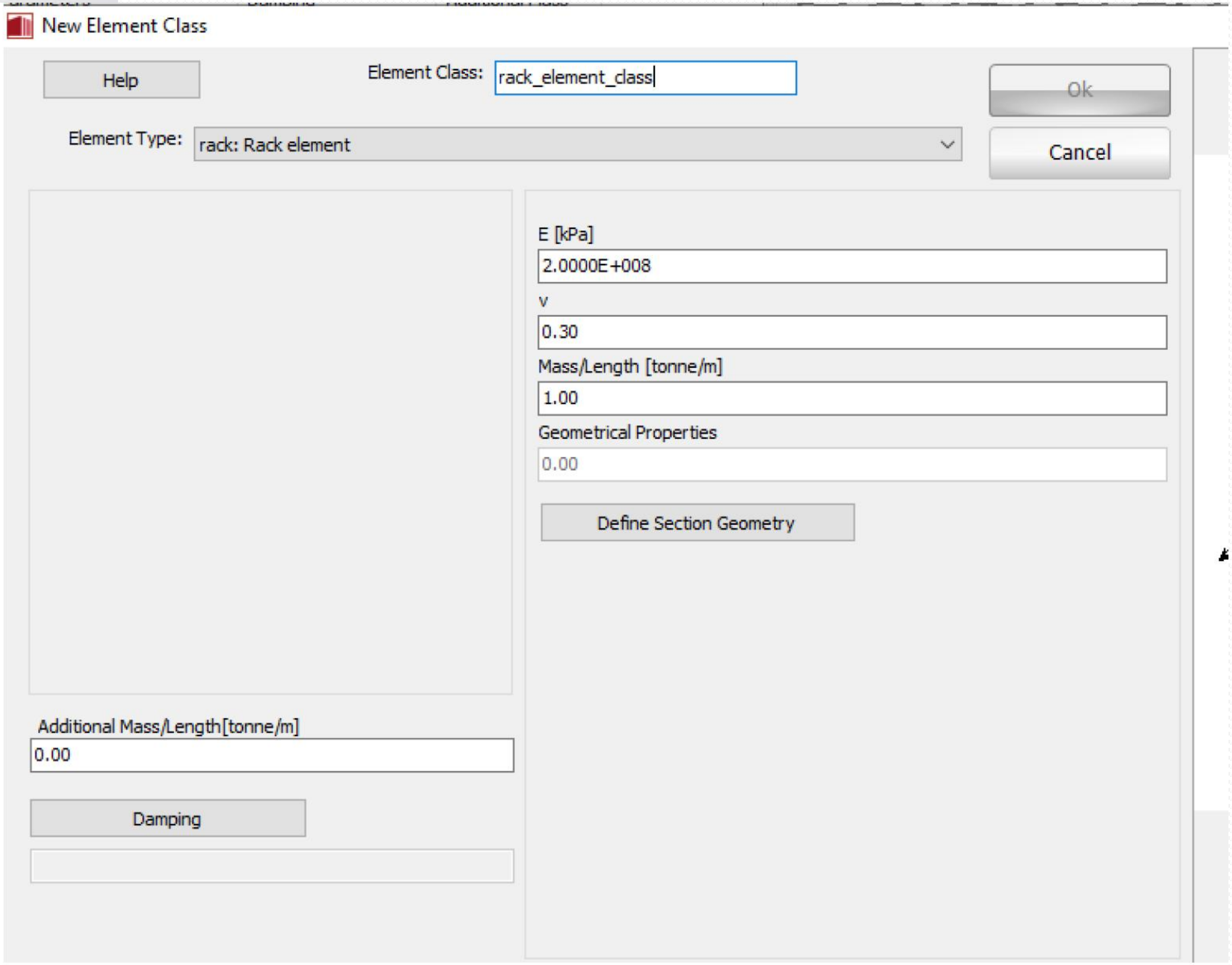
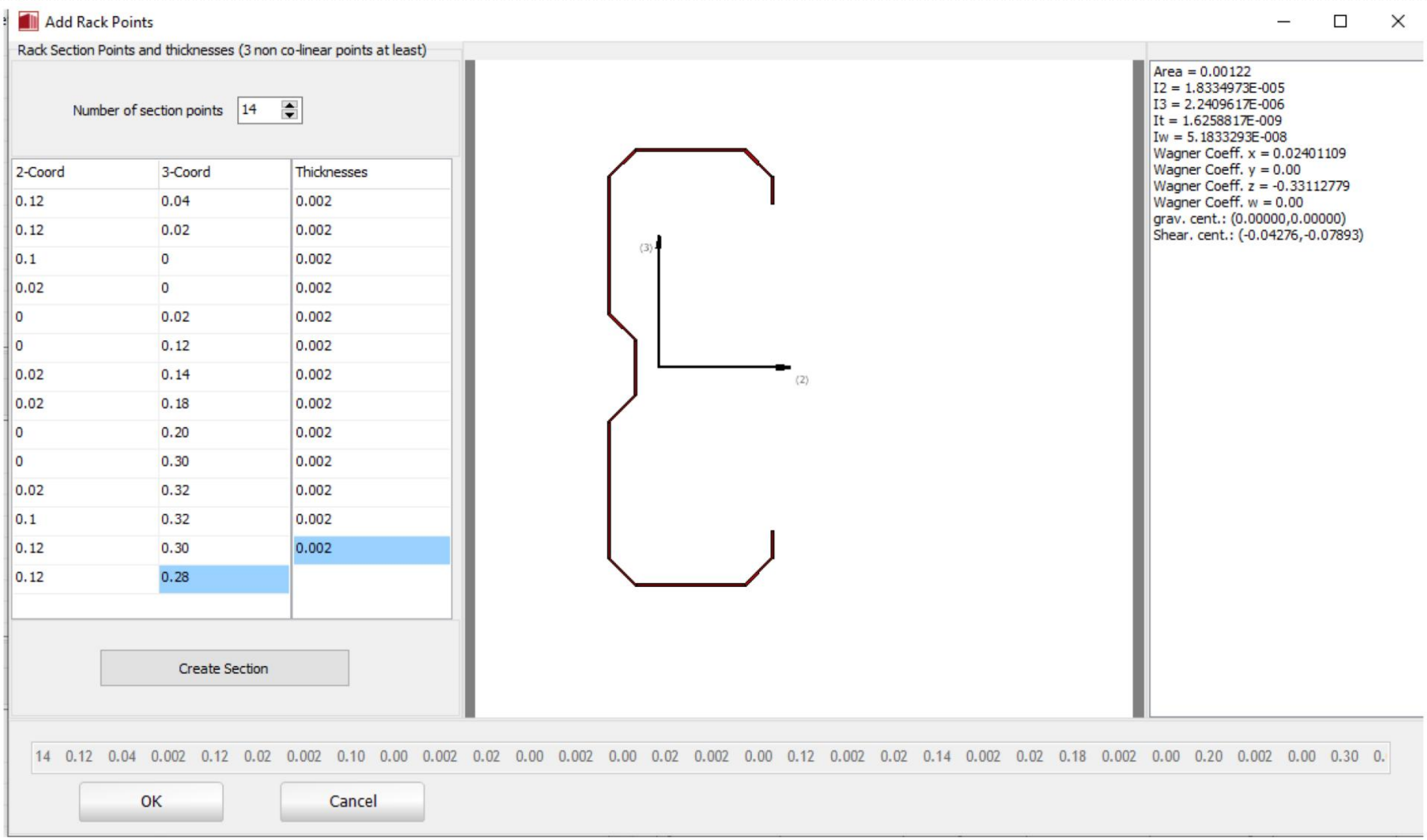
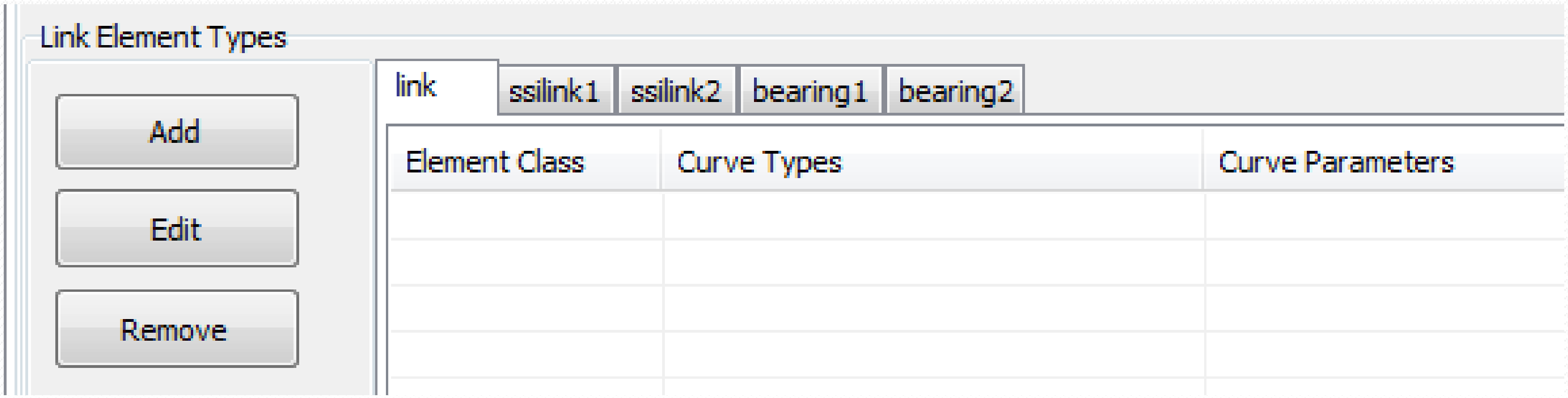


Definition of a new rack element



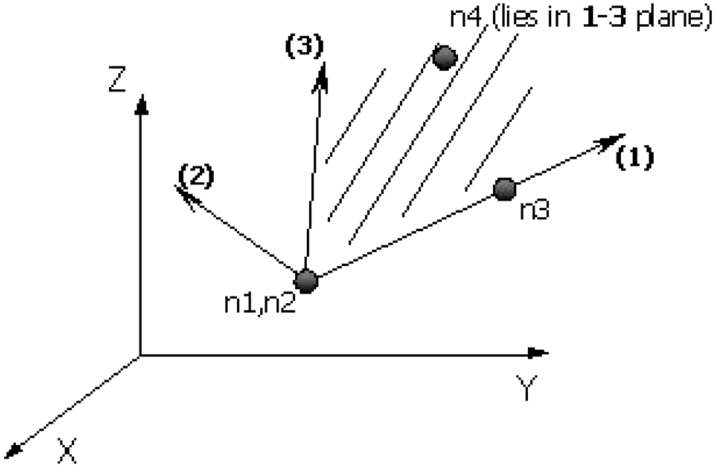
Definition of the section configuration for the rack element



Link elements link, SSI & SSI2, bearing1 & bearing2

Four nodes need to be defined for these element types. The first two are the end-nodes of the element and must be initially coincident since all link elements have an initial length equal to zero. The latter condition implies also that a third node is required to define local axis (1), noting that the orientation of this axis after deformation is determined by its initial orientation and the global rotation of the first node of the element. The fourth node is used to define local axes (2) and (3), following the convention described in global and local axes systems.

NOTE 1: Instead of the definition of a third and a fourth node, users may simply employ the keyword 'default', which implies that local axis-1 is along the X global axis and local axis-3 is along the Z global axis.



NOTE 2: Users are advised to make use of a non-structural node in the definition of the third and fourth element nodes.

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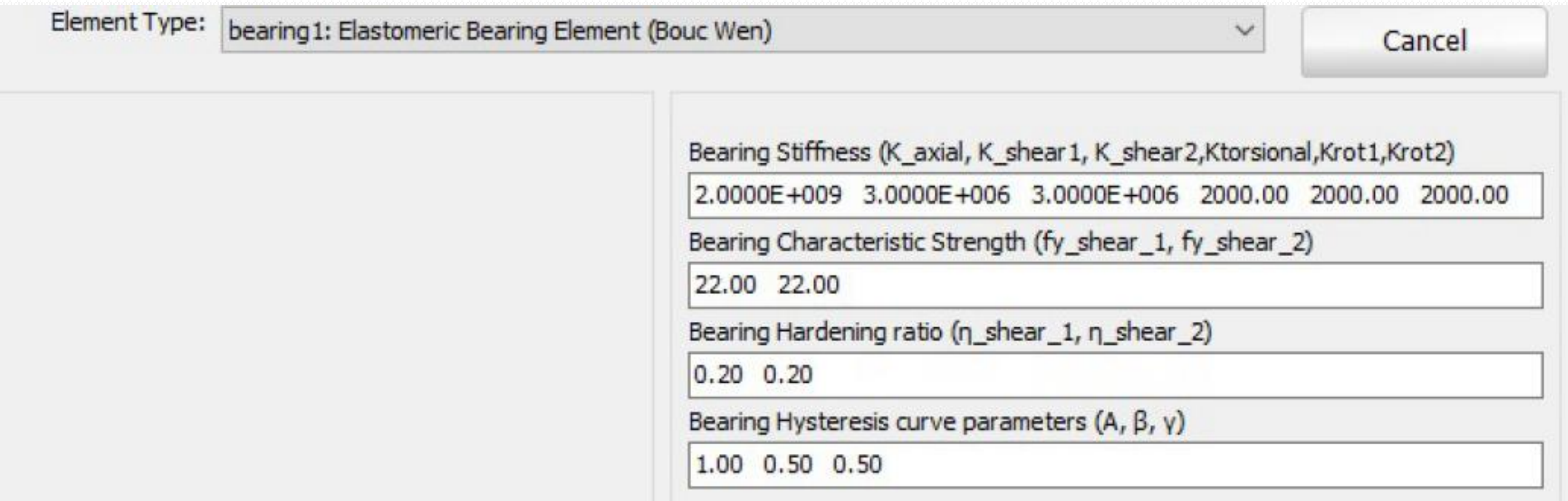
Elastomeric Bearing 1 Element (Bouc Wen) element type

Bearing 1 Elements are 3D elements with zero length used to model the behaviour of elastomeric bearings used in Seismic Isolation Applications. Bearing 1 Elements have coupled plasticity properties for the two shear directions (axes 2 and 3 in the local coordinate system of the bearing 1 element) while they are characterised by linear elastic behaviour for the remaining four deformation types. The behaviour in the shear directions is based on the hysteretic behaviour proposed by Wen [1976] and Park et al. [1986]. In the shear directions the force-deformation relationships follow the equations below:

$$f_2 = \eta\_shear\_2 \;\; K\_Shear\_2 \; u_2 + (1 - \eta\_shear\_2) \; Y_2 \; z_2$$

$$f_3 = \eta\_shear\_3 \;\; K\_Shear\_3 \; u_3 + (1 - \eta\_shear\_3) \; Y_3 \; z_3$$

where η_shear_2 and η_shear_3 are the ratios of the Post Yield Stiffness to the Elastic (Pre-Yielding) Stiffness of the bearing in each shear direction (Bearing Hardening Ratios), K_2 and K_3 are the elastic Stiffnesses of the bearing in each direction, Y_2 and Y_3 the yielding deformations in each shear direction while z_2 and z_3 are internal hysteretic variables.
Thirteen parameters are needed in order to describe the bearing 1 element behaviour:


Element Type:
bearing1: Elastomeric Bearing Element (Bouc Wen)
Cancel
Bearing Stiffness (K_axial, K_shear1, K_shear2,Ktorsional,Krot1,Krot2)
2.0000E+009 3.0000E+006 3.0000E+006 2000.00 2000.00 2000.00
Bearing Characteristic Strength (fy_shear_1, fy_shear_2)
22.00 22.00
Bearing Hardening ratio (η_shear_1, η_shear_2)
0.20 0.20
Bearing Hysteresis curve parameters (A, β, γ)
1.00 0.50 0.50

...t Properties	Typical values	Default values
Elastic Stiffness in the axial (local axis 1) direction - K_axial	2E+05 - 150E+05 [kN/m]	2E+06 [kNm]
Elastic Stiffness in the shear (local axis 2 and 3) directions - K_shear_1, K_Shear_2	3000 – 40000 [kN/m]	3000 [kNm]
Elastic Stiffnesses in the torsional and rotational degrees of freedom - Ktorsional, Krot1, Krot2	-	20000 [kNm]
Bearing Characteristic Strength for each shear direction – fy_shear_1, fy_shear_2	20-600 [kN]	22 [kN]
Bearing Hardening ratio for eachshear direction – η_shear_1, η_shear_2	0.1-0.8	0.2
Variable controlling the hysteresis Amplitude - A	1	1
First hysteretic shape variable - β	0.1-0.9	0.5
First hysteretic shape variable - γ	0.1-0.9	0.5

FRICTION PENDULUM BEARING/SYSTEM ELEMENT TYPE

Bearing 2 Elements are 3D elements with zero length used to model the behaviour of single friction pendulim bearings used in Seismic Isolation Applications. Bearing 2 Elements have coupled plasticity properties for the two shear directions (axes 2 and 3 in the local coordinate system of the element) while they are characterised by linear elastic behaviour for the remaining four deformation types. The friction model described by Constantinou et al. [1999] is utilised for calculating the friction coefficient of the friction pendulum bearing sliding surface. The friction coefficient is calculated according to the following equation:

$$\mu = ffast\_1 - (ffast\_1 - fslow\_1)exp(-rate\_1|v|)$$

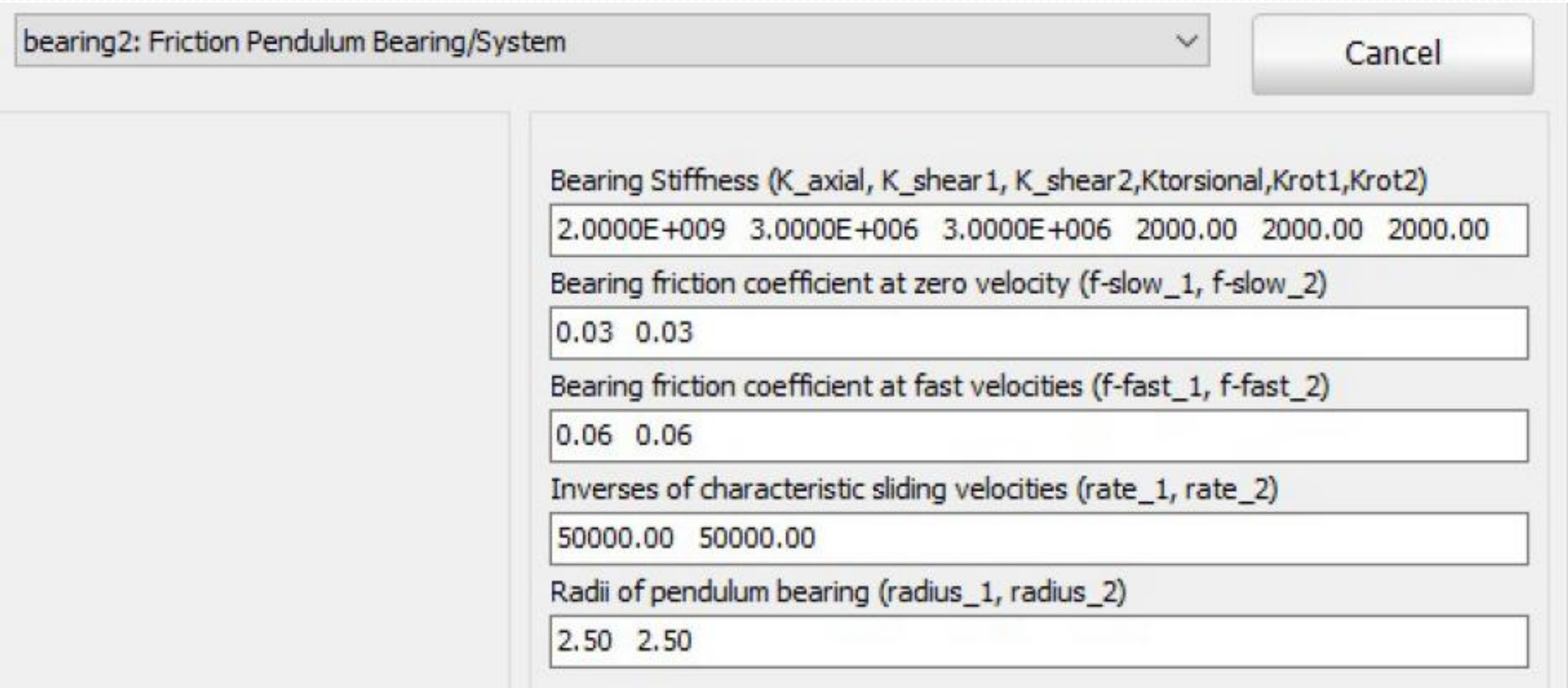
where ffast_1 and fslow_1 are the bearing friction coefficients at fast and slow velocities respectively, v is the bearing velocity anf rate_1 is the rate controlling the transition from low to high velocities.

The Bearing 2 element behaves elasticaly in the shear directions, with a stiffness equal to the elastic stiffness provided by the user, until the yielding limit defined by the yield strength which is calculated according to the following equation

$$Q_{yield} = \mu P$$

where P is the total vertical load on the bearing. Plastic deformations after the yielding point are computed using a Return-Mapping Algorithm as described for hardening models by Simo and Hughes [1998]. The post-yielding stiffness is equal to P/R where R is the radius of curvature of the friction pendulum and P is the total vertical load on the bearing.

Fouteen parameters are needed in order to describe the bearing 1 element behaviour:


bearing2: Friction Pendulum Bearing/System
Cancel
Bearing Stiffness (K_axial, K_shear1, K_shear2,Ktorsional,Krot1,Krot2)
2.0000E+009 3.0000E+006 3.0000E+006 2000.00 2000.00 2000.00
Bearing friction coefficient at zero velocity (f-slow_1, f-slow_2)
0.03 0.03
Bearing friction coefficient at fast velocities (f-fast_1, f-fast_2)
0.06 0.06
Inverses of characteristic sliding velocities (rate_1, rate_2)
50000.00 50000.00
Radii of pendulum bearing (radius_1, radius_2)
2.50 2.50

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Element Properties	Typical values	Default values
Elastic Stiffness in the axial (local axis 1) direction - K_axial	1E+06 - 30E+06 [kN/m]	2E+06 [kNm]
Elastic Stiffness in the shear (local axis 2 and 3) directions - K_shear_1, K_Shear_2	500 – 20000 [kN/m]	3000 [kNm]
Elastic Stiffnesses in the torsional and rotational degrees of freedom - Ktorsional, Krot1, Krot2	-	20000 [kNm]
Bearing Friction Coefficient at slow velocities –fslow_1, fslow_2	0.02-0.05	0.03
Bearing Friction Coefficient at fast velocities –ffast_1, ffast_2	0.04-0.1	0.06
Inverse of characteristic sliding velocities – rate_1, rate_2	50-100 [s/m]	50 [s/m]
Curvature radii of the friction pendulum – radius_1, radius_2	2-7 [m]	2,50 [m]

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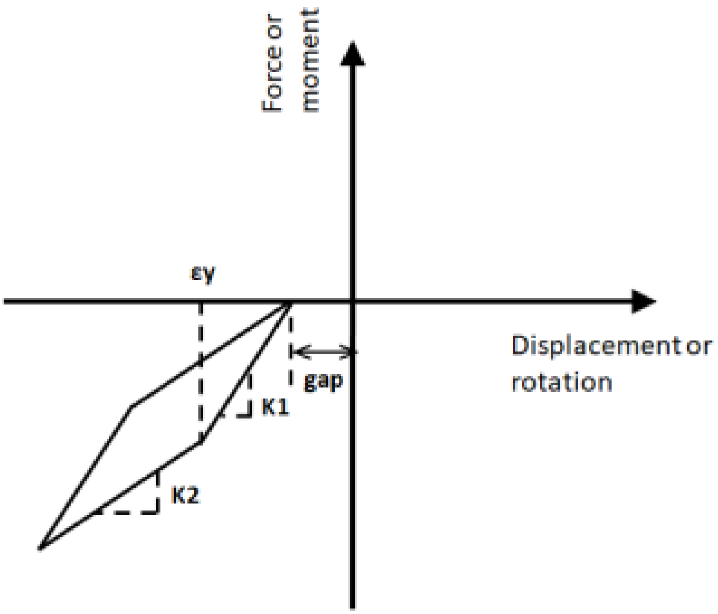
LINK ELEMENT TYPE

- Linear symmetric curve - lin_sym
- Linear asymmetric curve - lin_asm
- Bilinear symmetric curve - bl_sym
- Bilinear asymmetric curve - bl_asm
- Bilinear kinematic hardening curve - bl_kin
- Trilinear symmetric curve - trl_sym
- Trilinear asymmetric curve - trl_asm
- Quadrilinear symmetric curve - quad_sym
- Quadrilinear asymmetric curve - quad_asm
- Pinched asymmetric curve - pinched_asm
- Modified Ibarra-Medina-Krawinkler Deterioration curve with Bilinear Hysteretic Response – MIMK_bilin
- Modified Ibarra-Medina-Krawinkler Deterioration Model with Peak-Oriented Hysteretic Response – MIMK_peak
- Modified Ibarra-Medina-Krawinkler Deterioration Model with Pinched Hysteretic Response – MIMK_Pinched
- Nonlinear elastic curve - Non_lin_Elast

- Plastic curve – plst
- Simplified bilinear Takeda curve – Takeda
- Asymmetric bilinear Takeda curve – Takeda_asm
- Ramberg Osgood curve - Ramberg_Osgood
- Modified Richard-Abbott curve - Richard_Abbott
- Soil-structure interaction curve - ssi_py
- Gap-hook curve - gap_hk
- Multi-linear curve – multi_lin
- Smooth curve – smooth
- Viscous Damper – vsc_dmp
- Bouc Wen curve - Bouc_Wen
- Elastic – Perfectly plastic Gap curve - gap_elpl
- Impact response curve - pound_hz
- Self Centering Brace response curve - scb
- Generic Hysteretic Curve - gen_hyst

Impact response curve – pound_hz

The Impact response curve is described by Muthukumar and DesRoches [2006] and is based on the Herz Law. The model can be used for modeling impact and pounding events between oscillating structures. The model is characterised by compressive stress after the closing of the initial gap between the two members included in the pounding event. Four parameters have to be defined for the model:



Material Properties	Typical values	Default values
Initial Stiffness – **K1**	-	30(-)
Post Yield Stiffness – **K2**	-	2.6 (-)
Yield displacement– **εy**	-	0.02 (-)
Gap length– **gap**	-	0.005 (-)

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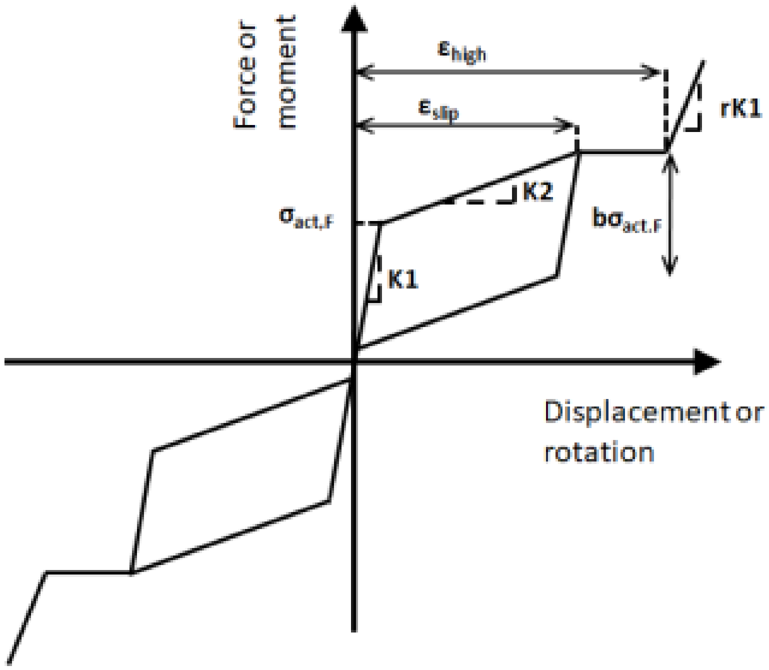
Self Centering Brace response curve- SCB

The Self Centering Brace (SCB) response curve is a uniaxial response curve modeling the behaviour of Self Centering Energy dissipating Braces. The model is described by Christopoulos et al. (2008). The SCB model resembles ordinary Flag-shaped models giving also the opportunity to incude two additional characteristics consisting of a non-recoverable slip of an external friction fuse available by the brace structure, and a steep increase of the stiffness of the brace at high deformations. In total seven parameters have to be defined for the model:



Material Properties	Typical values	Default values
Initial Stiffness – **K1**	100000 – 2000000 (kN/m)	200000 (kN/m)
Post-Yield Stiffness – **K2**	10000- 50000 (kN/m)	12000 (kN/m)
Activation Force– $\boldsymbol{\sigma_{act,F}}$	200 – 2000 (kN)	200 (kN)
Ratio of Forward to Reverse activation force – **b**	-	0.5
Slip displacement – $\boldsymbol{\varepsilon_{slip}}$	0-0.01(m)	0 (m)
Displacement at the start of stiffness increase – $\boldsymbol{\varepsilon_{,high}}$	-	0.3 (m)
Ratio of Increased Stiffness to Initial Stiffness – **r**	1.5-2	1.5

Other links

- pound_hz: Pounding Hertz model (curve for modeling impact/pounding events)
- scb: Self Centering Brace curve for the modeling of Self Centering Braces
- vsc_dmp: Viscous Damper curve to model Viscous Dampers
- Bouc_Wec: Bouc-Wec type of model
- gen_hyst: Generic Hysteretic Material (pinching with 4 deterioration modes)
- gap_elpl: elastic-perfectly plastic gap material
- quad_sym & quad_asm: Symmetric and asymmetric multilinear curves with degradation

Element Classes

Beam-Column element types

Inelastic force-based frame element type - infrmFB

Inelastic force-based plastic hinge frame element type - infrmFBPH

Inelastic displacement-based plastic hinge frame element type - infrmDBPH

Inelastic displacement-based frame element type - infrmDB

Elastic frame element - elfrm

Inelastic truss element - truss

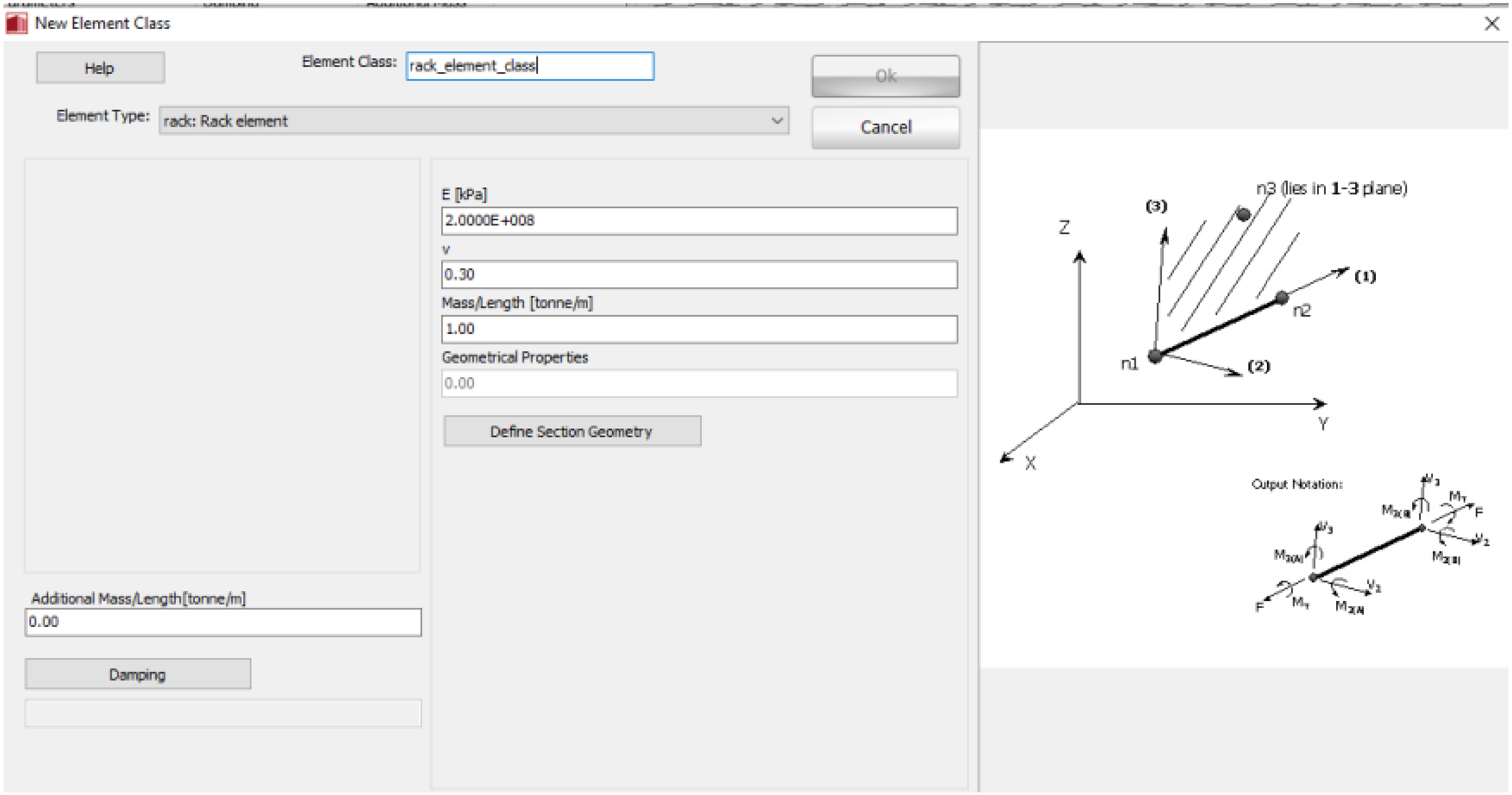
Inelastic infill panel element type

Rack element type

This element is a 3D beam element with thin-walled, open, cross-sections. The element is characterized by seven degrees of freedom per node, so that to correctly estimate both the displacements and the internal stresses, including warping displacements and bi-moment stresses, and to correctly predict the flexural-torsional and lateral-torsional buckling, derived by the coupling between flexure and torsion. Furthermore, the model accounts for the eccentricity of the shear centre from section centroid, and it considers all the Wagner coefficients, which makes it suitable for use with non-symmetric cross-sections.

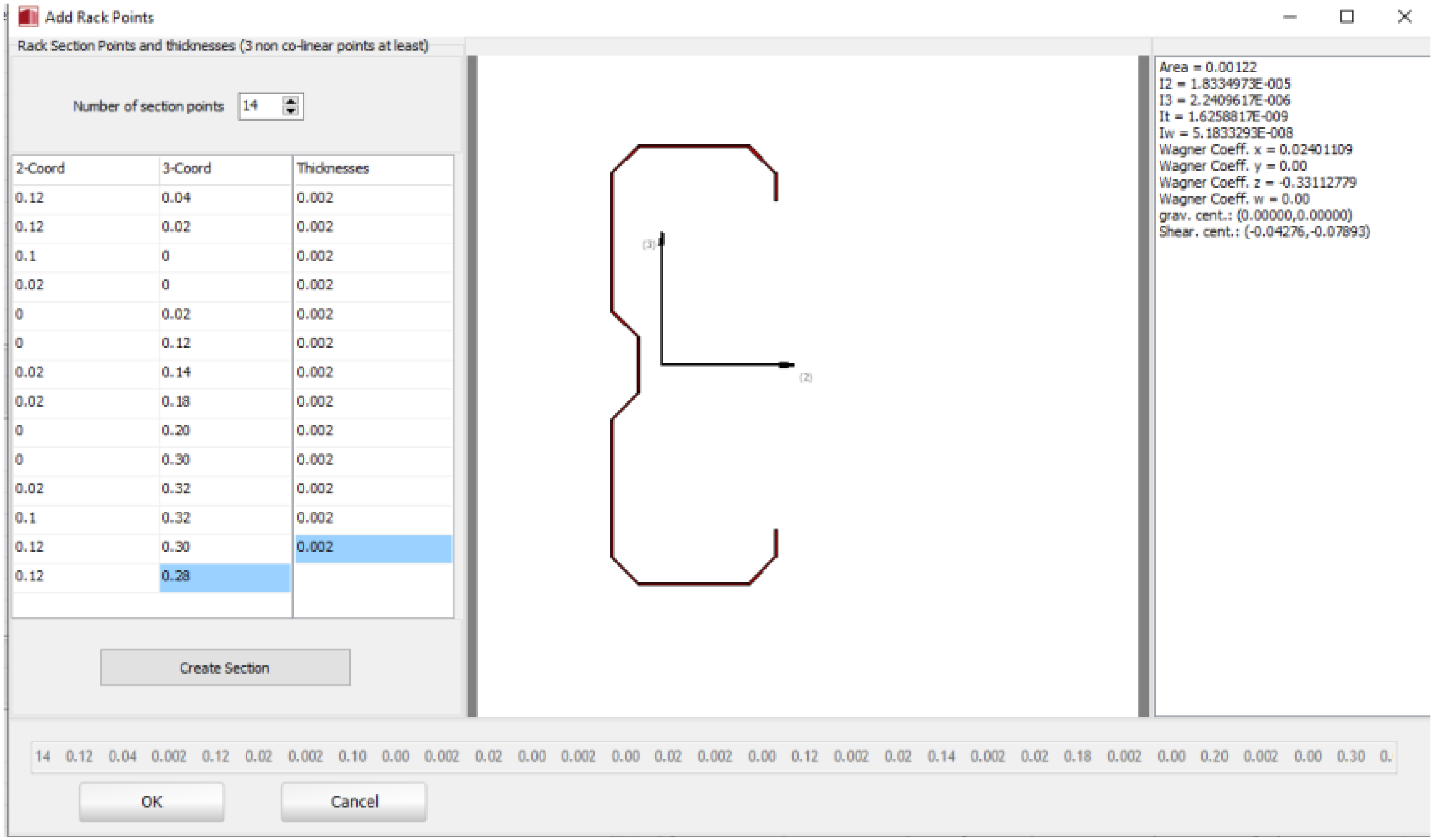
As a result, the formulation is ideal for the modelling of steel storage pallet racks (in the field of logistic), as well as scaffoldings, which are generally composed by uprights which have mono-symmetric lipped channel cross-sections.

The rack element can be fully defined, if the (elastic) material properties (modulus of elasticity and Poisson ratio) and the section configuration are provided. The former are given on the main dialog box of the rack element class (see figure below).

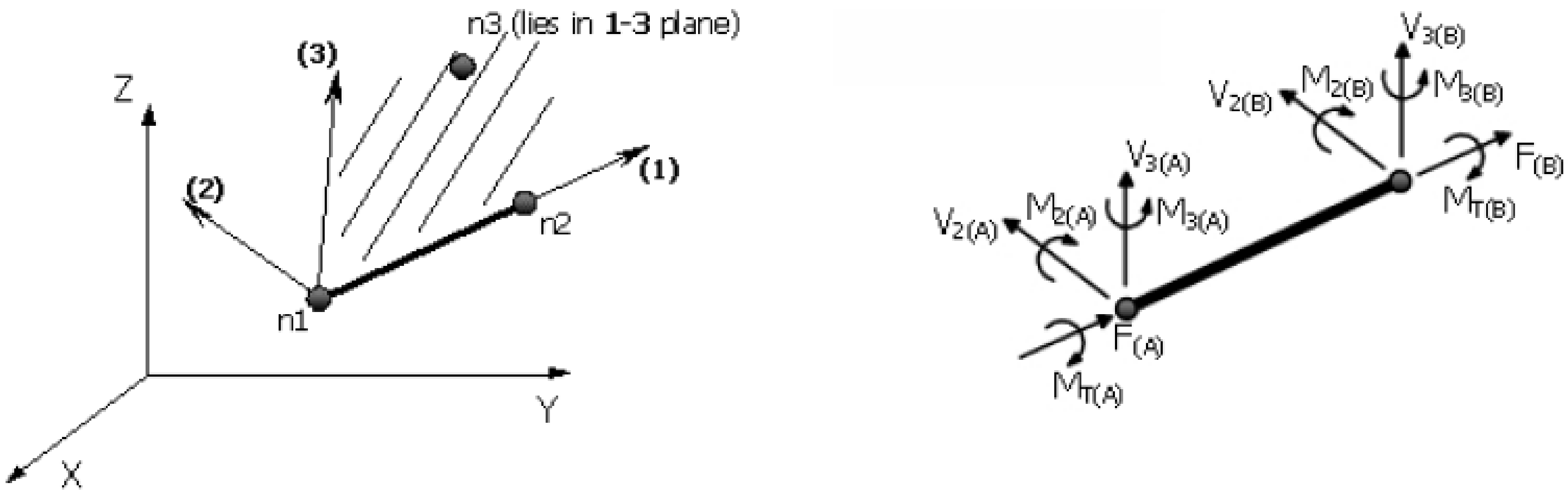


Definition of a new rack element

The section geometry can be defined in a special dialog box. Any thin-walled open section configuration can be modelled, and different thicknesses may be assigned at the different parts of the section. After the user defines the coordinates of the corner points of the section, and clicks on the Create Section button, the section is shown on the screen and the elastic section properties, the Wagner coefficients and the position of the shear centre are automatically calculated.



Definition of the section configuration for the rack element



Local Axes and Output Notation for rack elements

ANALYSIS TYPES

- Eigenvalue analysis
- Static Time-History analysis
- Static analysis
- Dynamic Time-History analysis
- Static Pushover analysis
- Incremental Dynamic analysis
- Static Adaptive Pushover analysis
- Response Spectrum analysis-

BUCKLING ANALYSIS

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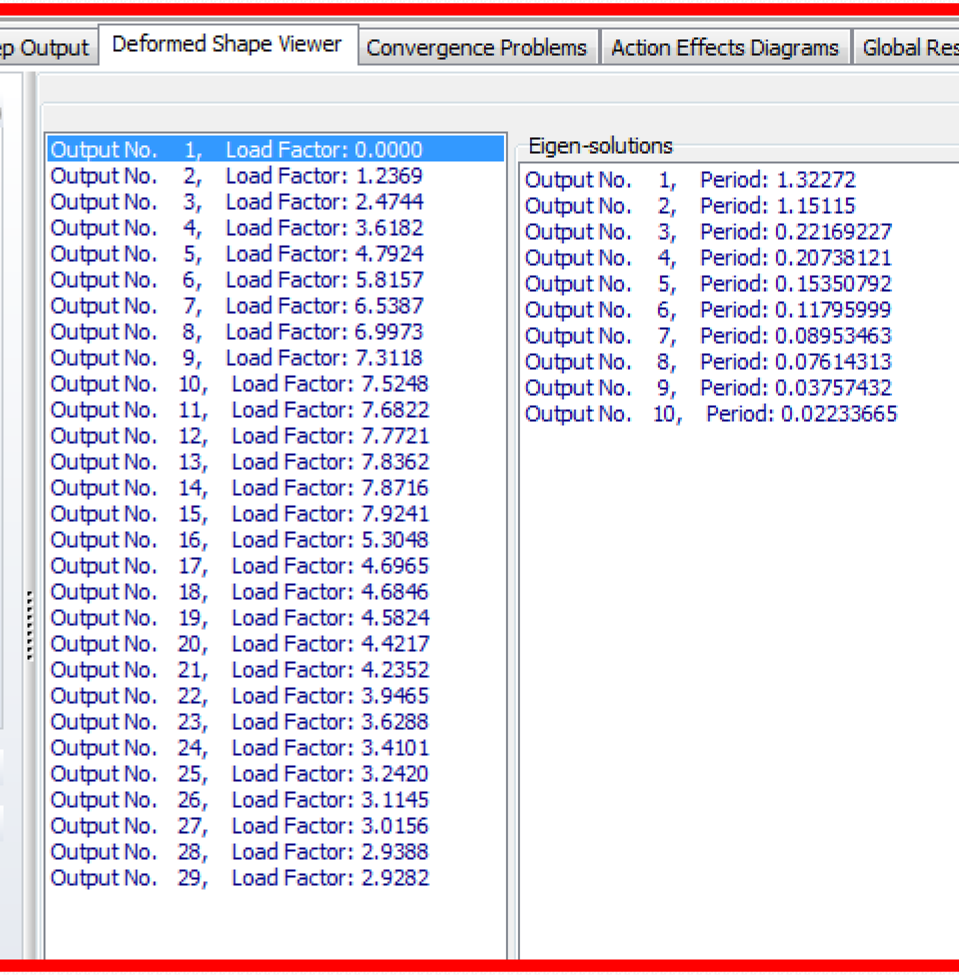
BUCKLING ANALYSIS

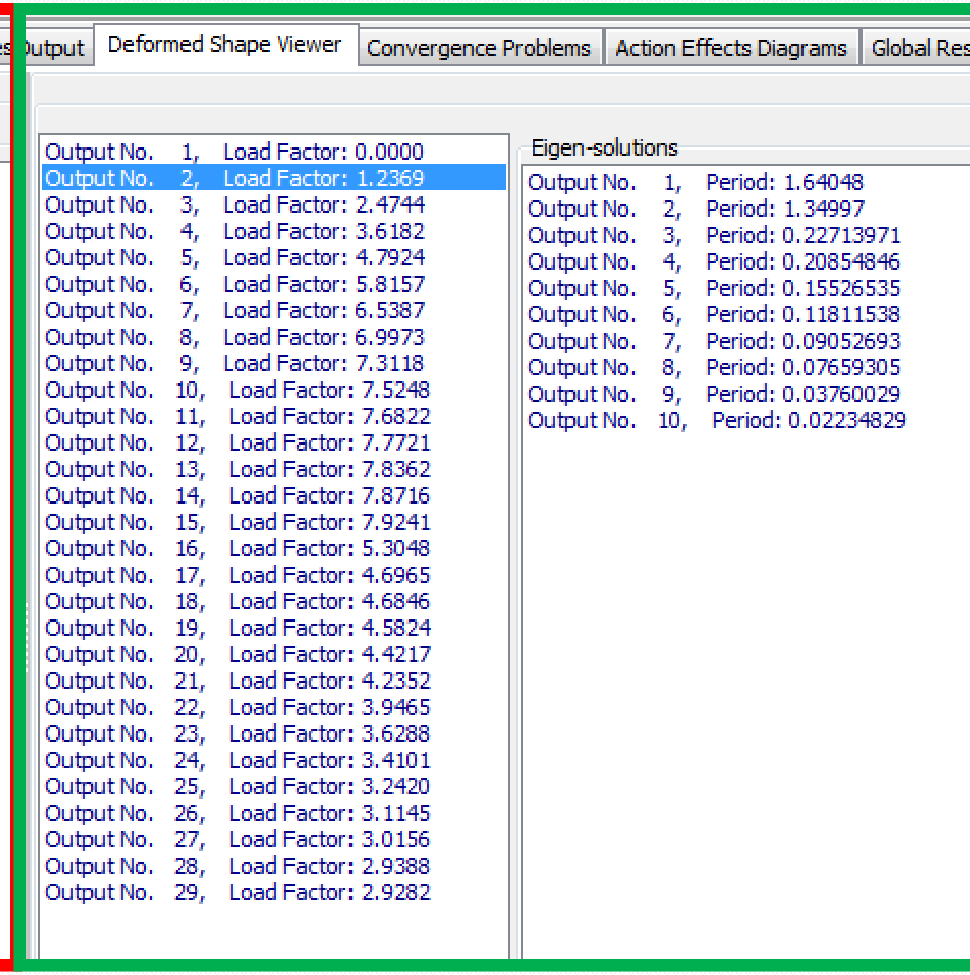
In general, in order to identify the limit point that recognises the transition from a stable to an unstable structure, an incremental analysis should be performed. The incremental analysis considers both geometric and material nonlinearities. In some cases, i.e. slender steel structures, the stability is governed by the geometric nonlinearities. Hence, neglecting nonlinear material behaviour and assuming the relative distribution of internal force equal at all ratios of the applied load, a buckling analysis can be performed in place of the incremental one. Besides at these two assumptions, the element geometric stiffness matrices are linear functions of their end forces. Hence, these hypothesis permit to write the global stiffness equation in the form of a generalised eigenvalue problem in which the equation of equilibrium at the critical state is

$$[K_E + \lambda_i K_G]d_i = 0$$

K_E is the linear elastic stiffness matrix. K_G is the geometric stiffness matrix which represents the change in stiffness that results from changes in geometry as the applied loading is increased. It is computed for a reference loading pattern P_{ref} which corresponds to the base state of structure with preloads. λ_i is a vector of load factors (eigenvalues) with respect to P_{ref} and d_i is the buckling mode shape (eigenvectors), where i refers to the ith buckling mode. The lowest value of λ_i yields the elastic critical load vector λ_{min} P_{ref}. Commonly, it is easier to solve this kind of problem than to solve an incremental analysis.

Eigenvalues Analysis





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