ACCOUNTING FOR IMPERFECTIONS AS PER EUROCODE 2 AND EUROCODE 3 IN ETABS 2016 AND SAP2000 V19

I. Overview

Eurocode 2 and Eurocode 3 requires frame and member imperfections (global and local) be accounted for either in the analysis stage or in the design stage.

Accounting global and local imperfections in the analysis stage of a numerical model can be done by either:

- Modeling the global and/or local imperfections explicitly in the model
- Applying horizontal UDL along vertical member length for local imperfection (applicable for EC3 only)
- Applying Equivalent Horizontal Force (EHF) at each stories for global imperfection.
- Using the buckling mode shapes as initial geometry (applicable for EC3 only)

Accounting global and/or local imperfections in the analysis stage by modelling the frames and members unstraight or crooked, although can be done, will be impractical and time consuming. Similarly, applying horizontal UDL on vertical members to account for local imperfection is only practical when checking the effect on a single vertical element. These methods will not be discussed here.

In EC2 and EC3, global imperfections can be accounted for in the design stage as ECC (eccentricity) on isolated vertical members. Local imperfections in EC3 are accounted for in the design stage by using the relevant buckling shape factors in member buckling check.

This technical note discusses on how to account for imperfections in ETABS 2016 and SAP2000 v19 models.

II. Accounting for Global Imperfection in Design Stage as ECC

The most convenient way to account for global imperfections is to consider it as an ECC in design of isolated vertical members. This method is available in ETABS 2016 and SAP200o v19 for EC2 design, but not currently available for EC3 design. This will be included for EC3 design in the future software release/updates.

The inclination value will be calculated as:

$\theta_i = \theta_0 \, \alpha_h \, \alpha_m$

where

θ_0 = is the basic value given as 0.005 (= 1/200) ; [Please check your NDP value]
$\alpha_m = 1$

$\alpha_h = 2 \div \surd l$; $2/3 \leq \alpha_h \leq 1$

l = actual length of the vertical member

The eccentricity due to imperfection will be calculated as:

$e_i = \theta_i \, l_0/2$

The eccentricity e_i shall be equal or bigger than code specified minimum eccentricity:

$e_{min} = h/30 \geq 20$ mm

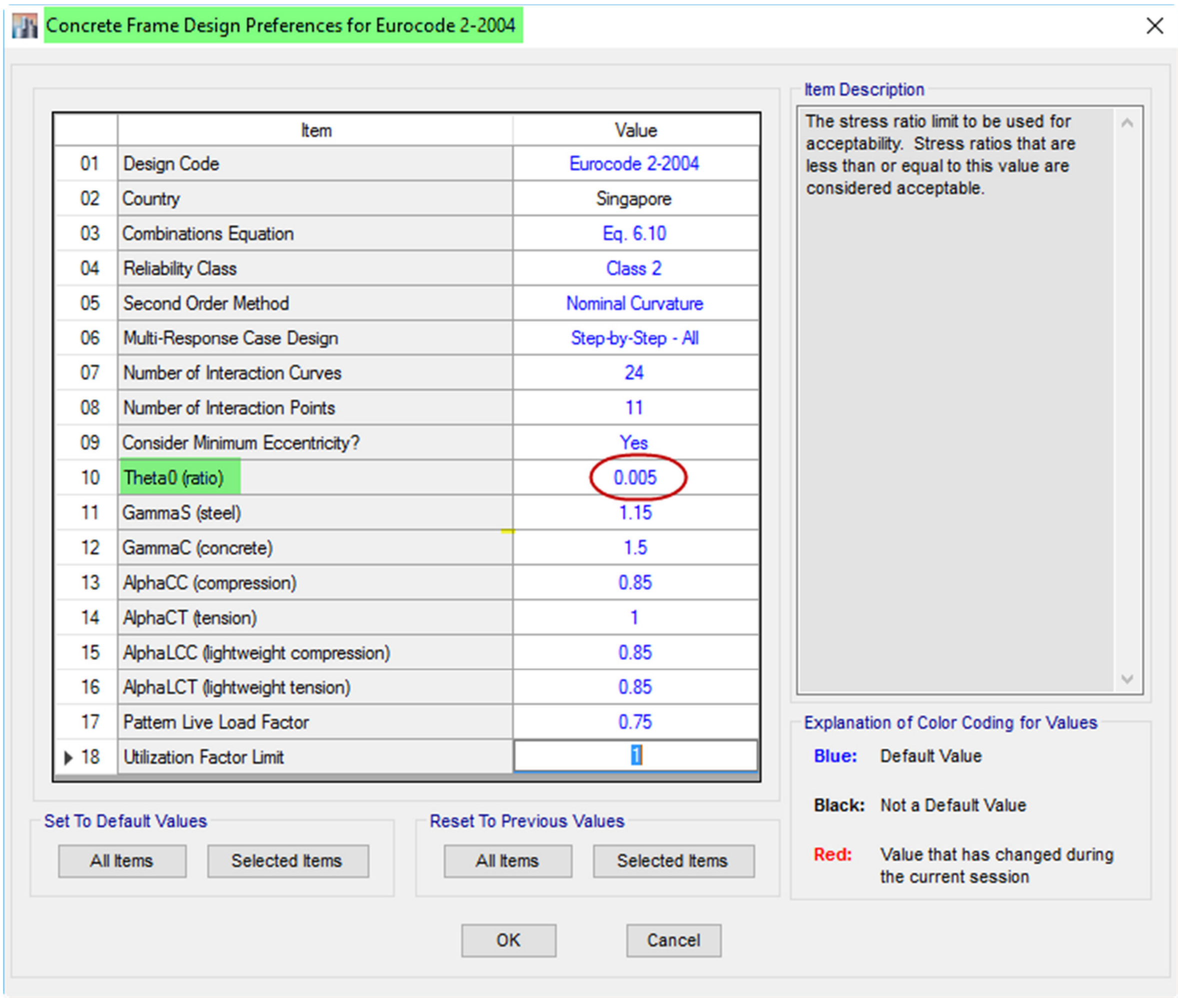
The resulting geometric imperfection moments, M_{i2} and M_{i3}, will be added to analysis first order moments in a single direction at a time:

$M_{i2} = e_{i2} \, N_{Ed}$

$M_{i3} = e_{i3} \, N_{Ed}$

Setting **Concrete Frame Design Preference** to account for global imperfection in ETABS 2016 and SAP2000 v19 for EC2 column design:

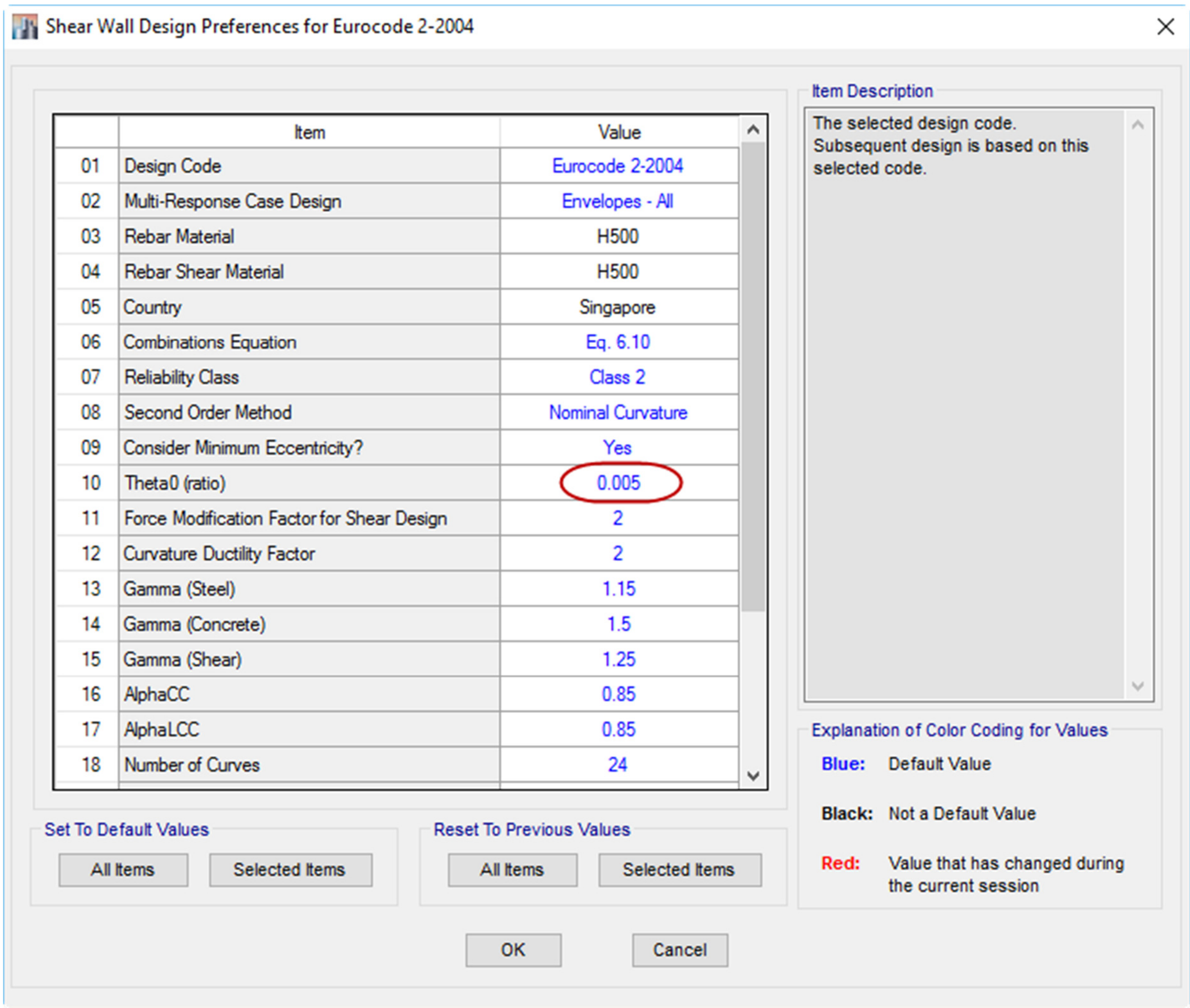
Go to **Design>Concrete Frame Design>View/Revise Preferences…** and specify Theta0 value.



	Item	Value
01	Design Code	Eurocode 2-2004
02	Country	Singapore
03	Combinations Equation	Eq. 6.10
04	Reliability Class	Class 2
05	Second Order Method	Nominal Curvature
06	Multi-Response Case Design	Step-by-Step - All
07	Number of Interaction Curves	24
08	Number of Interaction Points	11
09	Consider Minimum Eccentricity?	Yes
10	Theta0 (ratio)	0.005
11	GammaS (steel)	1.15
12	GammaC (concrete)	1.5
13	AlphaCC (compression)	0.85
14	AlphaCT (tension)	1
15	AlphaLCC (lightweight compression)	0.85
16	AlphaLCT (lightweight tension)	0.85
17	Pattern Live Load Factor	0.75
18	Utilization Factor Limit	1

Setting **Shear Wall Design Preference** to account for global imperfection in ETABS 2016 and SAP2000 v19 for EC2 wall design:

Go to **Design>Shear Wall Design>View/Revise Preferences…** and specify Theta0 value.



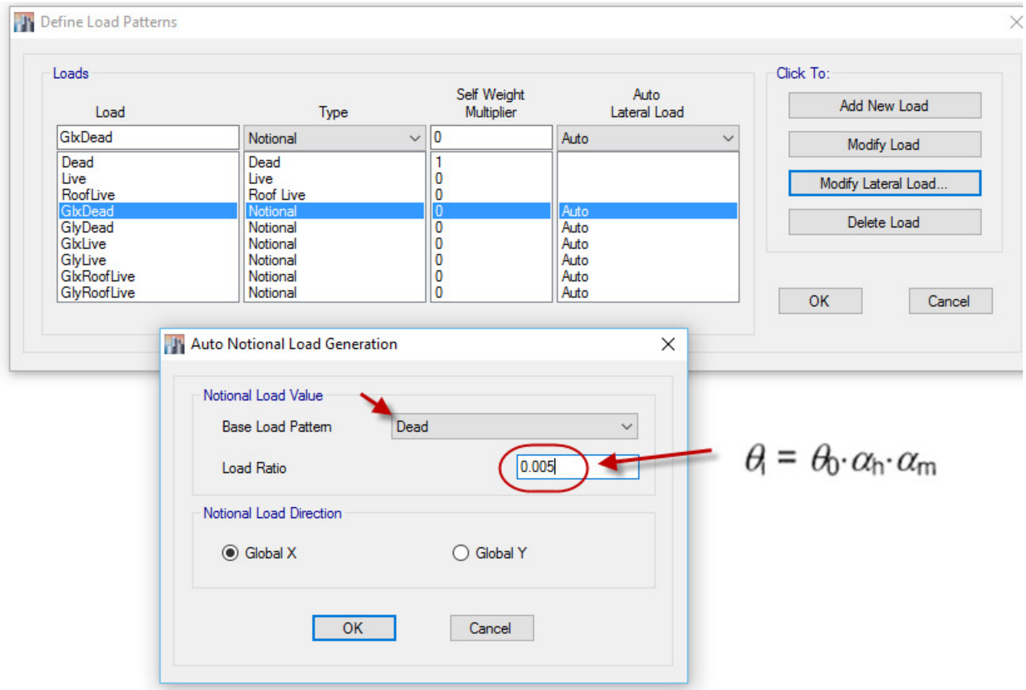
III. Accounting for Global Imperfection in Analysis Stage as EHF

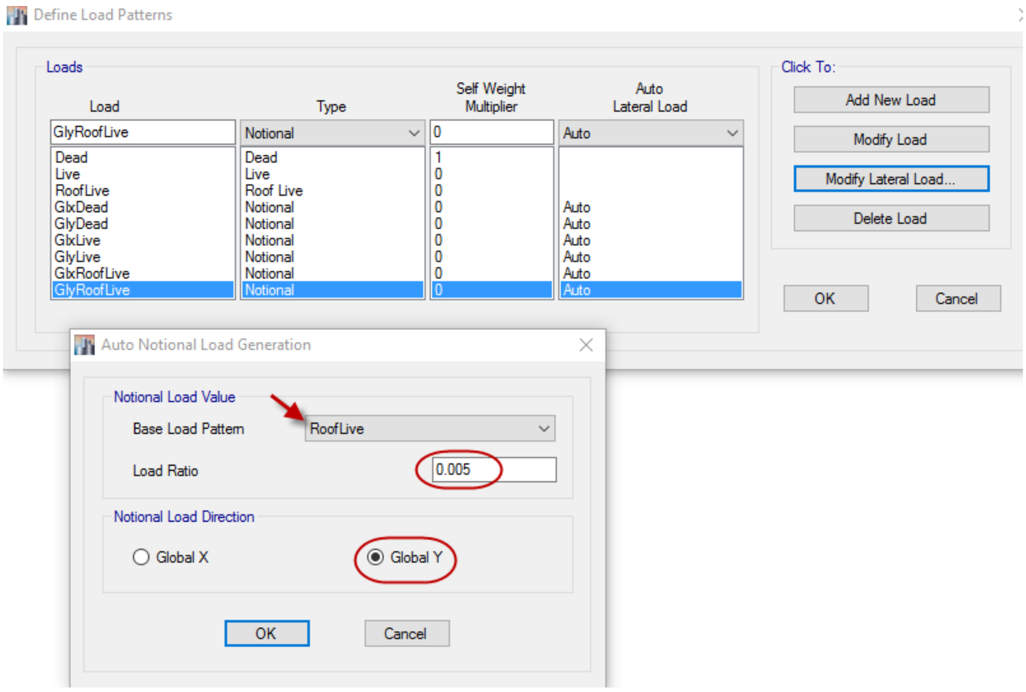
Global imperfection can be accounted for in analysis stage as EHF defined as notional loads in ETABS 2016 and SAP2000 v19.

This is done by defining notional load patterns for all dead load and live load pattern in both x and y directions.

Click on **Define>Load Patterns…**

Add a new notional load pattern for each dead and live load patterns for x and y directions. Specify the Load Ratio as the value of calculated θ_i, or conservatively use 0.0005.





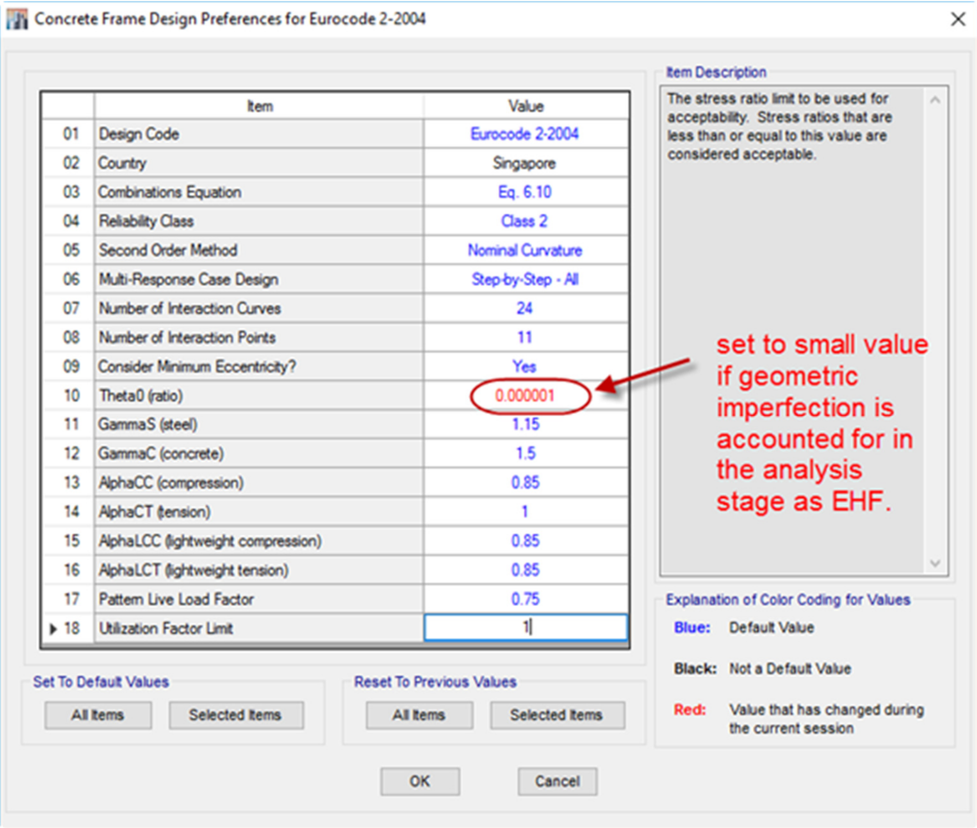
The value of Load Ratio to be used in defining the EHFs is equal to inclination θ_i from equation below:

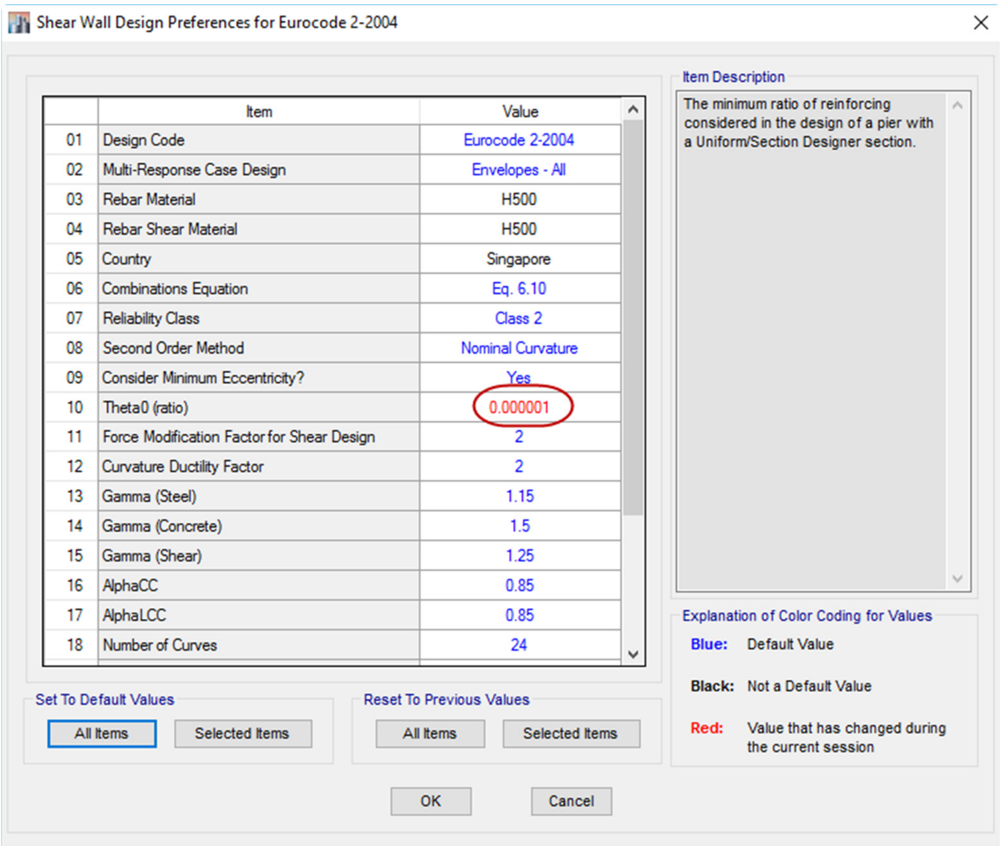
$\theta_i = \theta_0 \alpha_h \alpha_m$

where

θ_0 = is the basic value given as 0.005 (= 1/200) ; [Please check your NDP value]
α_m = number of vertical members contributing to the horizontal force
$\alpha_h = 2 \div \sqrt{l}$; $2/3 \leq \alpha_h \leq 1$
l = height of building

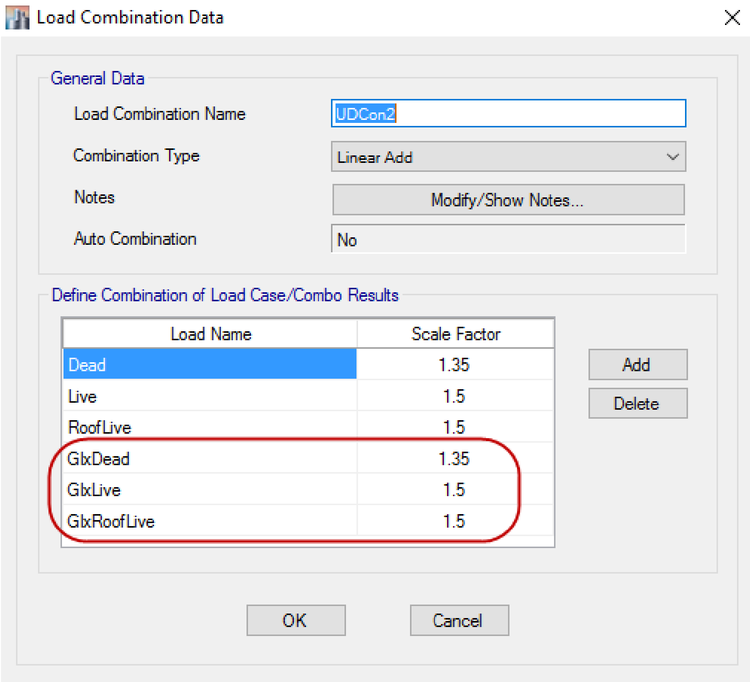
If designing for EC2, set the Theta0 value in the concrete frame and shear wall design preferences with very small value (i.e., 0.000001). This is done so that the effect of global imperfection will not be accounted twice. This step is not required if designing for EC3.

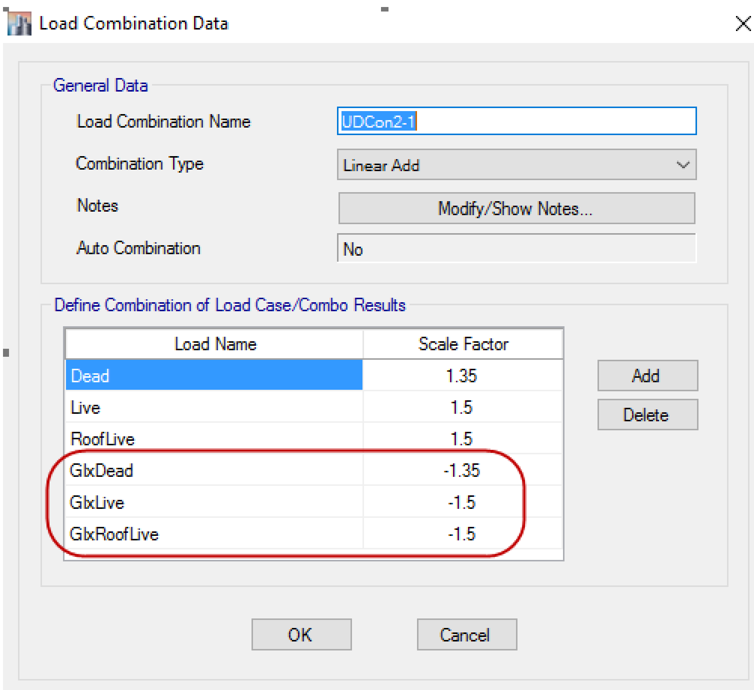




The design load combinations should also be updated to include these effects.

Click on **Define>Load Combinations...** and modify/create design load combinations to include the generated EHF load cases with the appropriate load factors. Use positive and negative values for opposing directions.





IV. Accounting for Local Imperfection in Design Stage

ETABS 2016 and SAP2000 v19 automatically accounted for local imperfection effect by the relevant buckling and imperfection factors as per EC3 in the design for buckling resistance of vertical steel members. This is done automatically and no additional input is required.

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \le 1 \qquad (6.61)$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \le 1 \qquad (6.62)$$

The reduction factor, χ for the relevant buckling mode is taken as:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \le 1.0 \qquad \text{(EC3 6.3.1.2(1))}$$

where the factor, Φ and the non-dimensional slenderness, $\bar{\lambda}$ are taken as:

$$\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] \qquad \text{(EC3 6.3.1.2(1))}$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}, \quad \text{for Class 1, 2 and 3 cross-sections} \qquad \text{(EC3 6.3.1.3(1))}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{\sqrt{A_{eff}/A}}{\lambda}, \quad \text{for Class 4 cross-sections} \qquad \text{(EC3 6.3.1.2(1))}$$

The reduction factor χ_{LT} is taken as:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \le 1.0 \qquad \text{(EC3 6.3.2.2(1))}$$

where the factor, Φ, and the non-dimensional slenderness, $\bar{\lambda}_{LT}$ are taken as:

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2\right] \qquad \text{(EC3 6.3.2.2(1))}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} \qquad \text{(EC3 6.3.2.2(1))}$$

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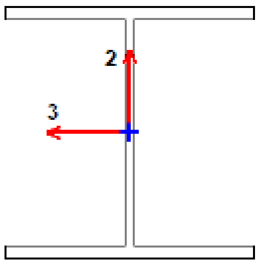
ETABS 2015 Steel Frame Design

Eurocode 3-2005 Steel Section Check (Strength Summary)

SAMPLE EC3 CALCULATION SUMMARY



Element Details (Part 1 of 2)

Level	Element	Unique Name	Length (mm)	Location (mm)	Combo	Design Type	Element Type
Story3	C7	19	4000	3596.8	UDStlS10-1	Column	DCL MRF

Element Details (Part 2 of 2)

Section	Classification	Rolled
UKC305X305X97	Class 3	Yes

Design Parameters

National Annex	Combination Equation	Analysis Type	Reliability
Singapore	Eq. 6.10	Method 2 (Annex B)	Class 2

Design Code Parameters

γ_{M0}	γ_{M1}	γ_{M2}	A_n/A_g	LLRF	PLLF	Stress ratio Limit
1	1	1.1	1	1	0.75	0.95

Section Properties

A (cm²)	I_{yy} (cm⁴)	i_{yy} (mm)	$W_{el,yy}$ (cm³)	$A_{v,y}$ (cm²)	$W_{pl,yy}$ (cm³)	I_{yz} (cm⁴)	I_t (cm⁴)
123	22249	134.5	1445.2	30.5	1592	0	91.2

I_{zz} (cm⁴)	i_{zz} (mm)	$W_{el,zz}$ (cm³)	$A_{v,z}$ (cm²)	$W_{pl,zz}$ (cm³)	I_w (cm⁶)	h (mm)
7308	77.1	478.7	95.6	726	1562218	307.9

A_{eff} (cm²)	e_{Ny} (mm)	e_{Nz} (mm)	$W_{eff,yy}$ (cm³)	$W_{eff,zz}$ (cm³)
123	0	0	1445.2	478.7

Material Properties

E (MPa)	f_y (MPa)	f_u (MPa)
210000	275	430

Stress Check Forces and Moments

Location (mm)	N_{Ed} (kN)	$M_{Ed,yy}$ (kN-m)	$M_{Ed,zz}$ (kN-m)	$V_{Ed,z}$ (kN)	$V_{Ed,y}$ (kN)	T_{Ed} (kN-m)
3596.8	-121	-71	35	36	-17	0

SAMPLE EC3 CALCULATION SUMMARY

Demand/Capacity (D/C) Ratio 6.2.1(7), 6.2.9.2(1)

D/C Ratio = $N_{Ed}/N_{Rd} + M_{y,Ed}/M_{y,Rd} + M_{z,Ed}/M_{z,Rd}$
0.477 = 0.032 + 0.263

Basic Factors

Buckling Mode	K Factor	L Factor	L Length (mm)	L_{cr}/i
Major (y-y)	1.834	0.899	3596.8	49.035
Major Braced	1	0.899	3596.8	26.743
Minor (z-z)	1.327	0.899	3596.8	61.899
Minor Braced	1	0.899	3596.8	46.663
LTB	1.327	0.899	3596.8	61.899

Axial Force Design

	N_{Ed} Force kN	$N_{c,Rd}$ Capacity kN	$N_{t,Rd}$ Capacity kN	$N_{byy,Rd}$ Major kN	$N_{bzz,Rd}$ Minor kN
Axial	-121	3383	3383	2890	2424

$N_{pl,Rd}$ kN	$N_{u,Rd}$ kN	$N_{cr,T}$ kN	$N_{cr,TF}$ kN	A_n/A_g Unitless
3383	4327	8984	8984	1

Design Parameters for Axial Design

	Curve	α	N_{cr} (kN)	λ	Φ	χ	$N_{bd,Rd}$ (kN)
Major (y-y)	b	0.34	10602	0.565	0.722	0.854	2890
MajorB (y-y)	b	0.34	35645	0.308	0.566	0.961	2890
Minor (z-z)	c	0.49	6654	0.713	0.88	0.717	2424
MinorB (z-z)	c	0.49	11708	0.537	0.727	0.822	2424
Torsional TF	c	0.49	8984	0.614	0.79	0.777	2629

Moment Designs

	M_{Ed} Moment kN-m	$M_{Ed,span}$ Moment kN-m	$M_{c,Rd}$ Capacity kN-m	$M_{v,Rd}$ kN-m	$M_{n,Rd}$ kN-m	$M_{b,Rd}$ Capacity kN-m
Major (y-y)	-71	-71	397	397	397	384
Minor (z-z)	35	35	132	132	132	

Moment Designs

	Section	Flange	Web	ε (Unitless)	α (Unitless)	ψ (Unitless)
Compactness	Class 3	Class 3	Class 1	0.924	0.58	-0.928

	Curve	α_{LT}	λ_{LT}	Φ_{LT}	χ_{LT}	C_1	M_{cr} (kN-m)
LTB	a	0.21	0.35	0.577	0.965	2.7	3236

	C_{my}	C_{mz}	C_{mLT}	k_{yy}	k_{yz}	k_{zy}	k_{zz}
Factors	0.4	0.4	0.4	0.403	0.406	0.992	0.406

Shear Design

	V_{Ed} Force (kN)	$V_{c,Rd}$ Capacity (kN)	T_{Ed}/Torsion (kN-m)	Stress Ratio	Status Check
Major (z)	36	484	0	0.075	OK
Minor (y)	17	1517	0	0.011	OK

V. Using Buckling Mode Shape as Modified Geometry to Account for Global and/or Local Imperfection in Analysis Stage

As per EC3, the assumed shape of global imperfections and local imperfections may also be derived from the elastic buckling mode of a structure in the plane of buckling considered.

The deformed geometry of any Buckling Analysis load case can then be used as modified geometry of the structure to account for global and/or local imperfections.

This method is useful for modelling non-building structures (e.g., domes, etc.) wherein modelling global imperfection as EHF may not be applicable anymore.

This method requires engineering judgment and experience in selecting which buckling load case and mode to use. This may also mean that a single structure will have to be saved in multiple models with different deformed geometry for analysis and design.

After running the buckling analysis load cases, click on **Analyze>Modify Undeformed Geometry…**

Select the buckling load case and one of its mode shape to be used to modify the initial geometry.