Tekla Structural Designer 2018i

Reference Guides (Eurocode)

September 2018
(6.1.06)

Table of Contents

Analysis Verification Examples

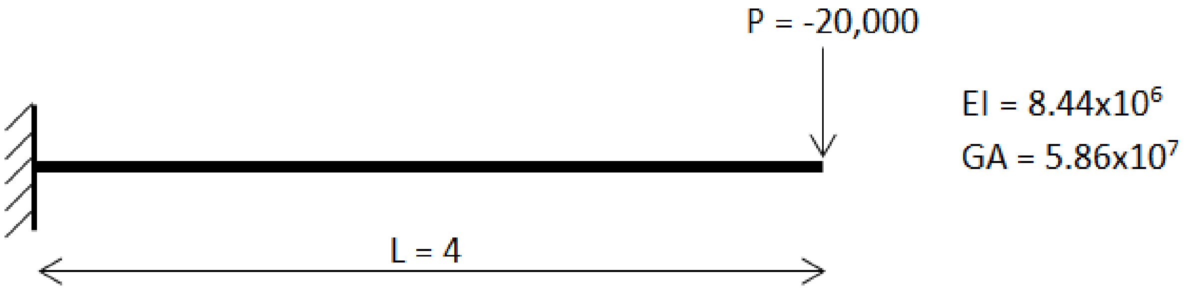
A small number of verification examples are included in this section. Our full automatic test suite for the Solver contains many hundreds of examples which are run and verified every time the Solver is enhanced.

These verification examples use SI units unless otherwise stated.

1st Order Linear - Simple Cantilever

Problem Definition

A 4 long cantilever is subjected to a tip load of 20,000.



Assumptions

Flexural and shear deformations are included.

Key Results

Result	Theoretical Formula	Theoretical Value	Solver Value	% Error
Support Reaction	-P	20,000	20,000	0%
Support Moment	PL	-80,000	-80,000	0%
Tip Deflection	$\frac{PL^3}{3EI} + \frac{PL}{GA}$	-0.0519	-0.0519	0%

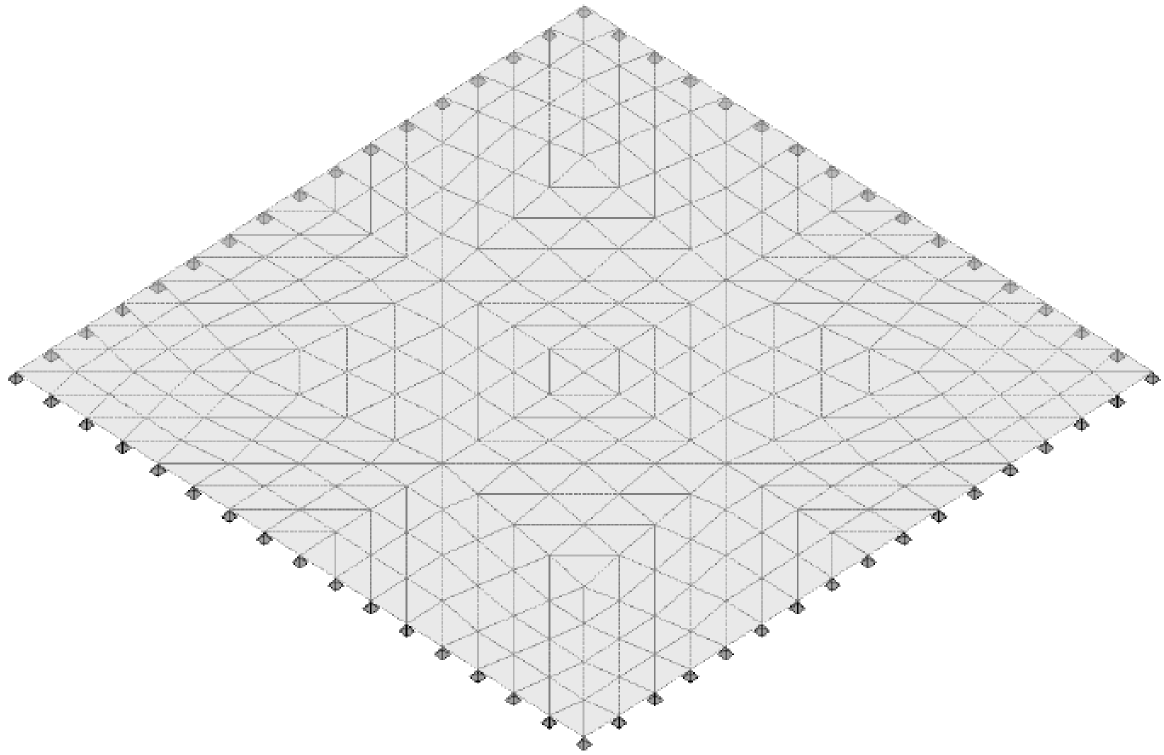
Conclusion

An exact match is observed between the values reported by the solver and the values predicted by beam theory.

1st Order Linear - Simply Supported Square Slab

Problem Definition

Calculate the mid span deflection of an 8x8 simply supported slab of 0.1 thickness under self-weight only. Take material properties $E=2x10^{11}$, $G=7.7x10^{10}$ and $\rho=7849$.



Assumptions

A regular triangular finite element mesh is used with sufficient subdivision. Flexural and shear deformation is included, and the material is assumed to be isotropic.

Key Results

The mid-span deformation is calculated using Navier's Method.

Result	Theoretical Value	Comparison 1	Solver Value	% Error

Mid-span deflection	7.002×10^{-3}	6.990×10^{-3}	7.031×10^{-3}	0.43%
Mid Span Moment	23616	23708	23649	0.14%

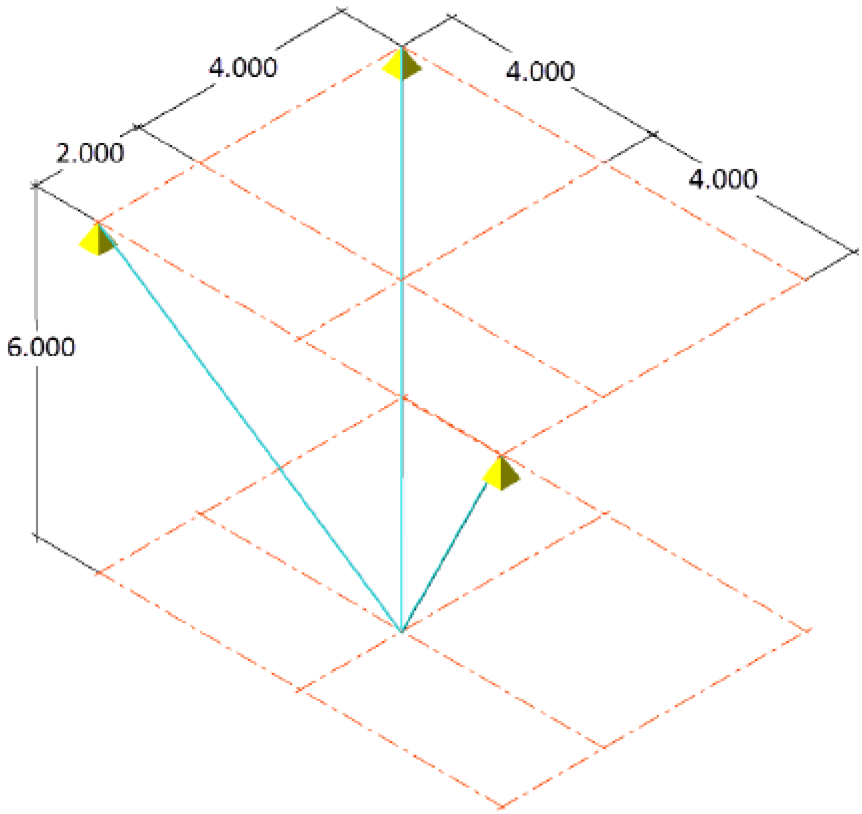
Conclusion

An acceptable match is observed between the theoretical values and the solver results. An acceptable match is also observed between the solver results and those obtained independently.

1st Order Linear - 3D truss

Problem Definition

Three truss members with equal and uniform EA support an applied load of -50 applied at the coordinate (4, 2, 6). The start of each truss member is fixed and are located at (0, 0, 0), (8, 0, 0) and (0, 6, 0) respectively. Calculate the axial force in each element.



Key Results

The results for this problem are compared against those published by Beer and Johnston and against another independent analysis package

Result	Beer and Johnston	Comparison 1	Solver Value	% Error
(0, 0, 0) - (4, 2, -6)	10.4	10.4	10.4	0%
(8, 0, 0) - (4, 2, -6)	31.2	31.2	31.2	0%
(0, 6, 0) - (4, 2, -6)	22.9	22.9	22.9	0%

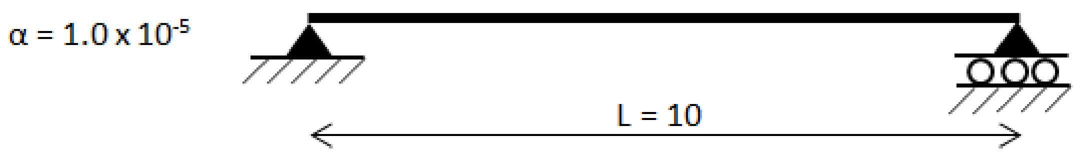
Conclusion

An exact match is observed between the values reported by the solver those reported by Beer and Johnston.

1st Order linear - Thermal Load on Simply Supported Beam

Problem Definition

Determine the deflection, *U*, due to thermal expansion at the roller support due to a temperature increase of 5. The beam is made of a material with a thermal expansion coefficient of 1.0 x 10^{-5}.



Assumptions

The roller pin is assumed to be frictionless.

Key Results

Result	Theoretical Formula	Theoretical Value	Solver Value	% Error
Translation at roller	$U = \Delta T \times \alpha \times L$	5 x 10^{-4}	5 x 10^{-4}	0.0%

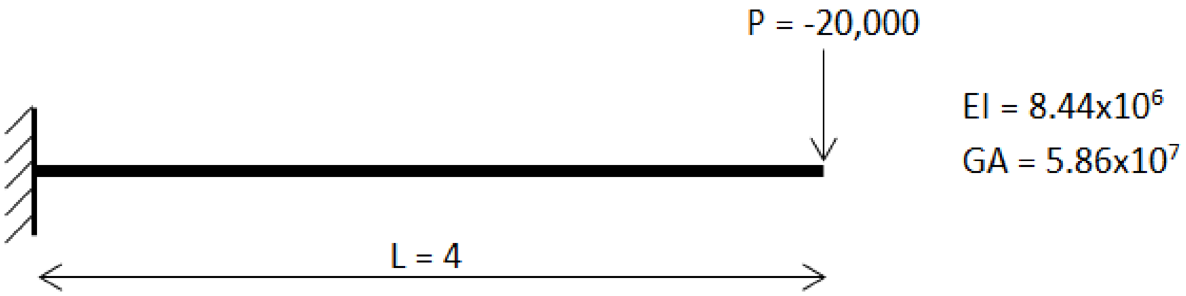
Conclusion

An exact match is shown between the theoretical result and the solver result.

1st Order Nonlinear - Simple Cantilever

Problem Definition

A 4 long cantilever is subjected to a tip load of 20,000.



Assumptions

Flexural and shear deformations are included.

Key Results

Result	Theoretical Formula	Theoretical Value	Solver Value	% Error
Support Reaction	-P	20,000	20,000	0%
Support Moment	PL	-80,000	-80,000	0%
Tip Deflection		-0.0519	-0.0519	0%

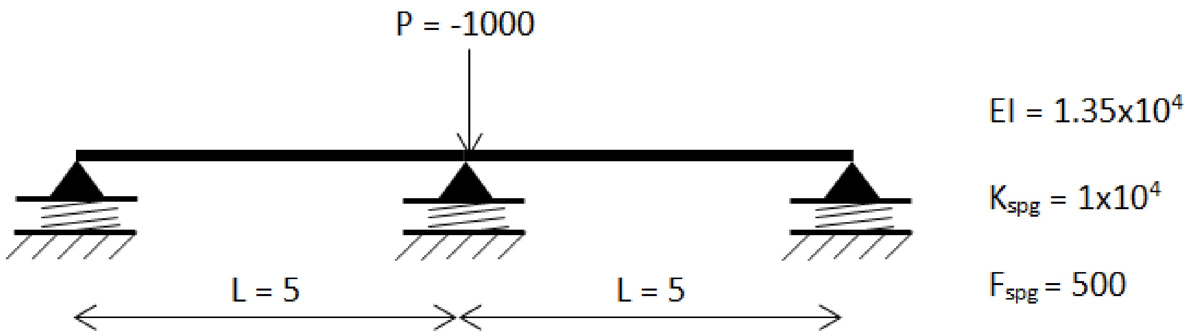
Conclusion

An exact match is observed between the values reported by the solver and the values predicted by beam theory.

1st Order Nonlinear - Nonlinear Supports

Problem Definition

A 10 long continuous beam is simply supported by three translational springs as shown. All springs have a maximum resistance force of 500. Calculate the reaction forces and deflection at each support.



Assumptions

Axial and shear deformations are ignored.

Key Results

Result	Comparison 1	Solver Value
LHS Reaction	250	250
Centre Reaction	500	500
RHS Reaction	250	250
LHS Displacement	-0.025	-0.025
Centre Displacement	-0.797	-0.797
RHS Displacement	-0.025	-0.025

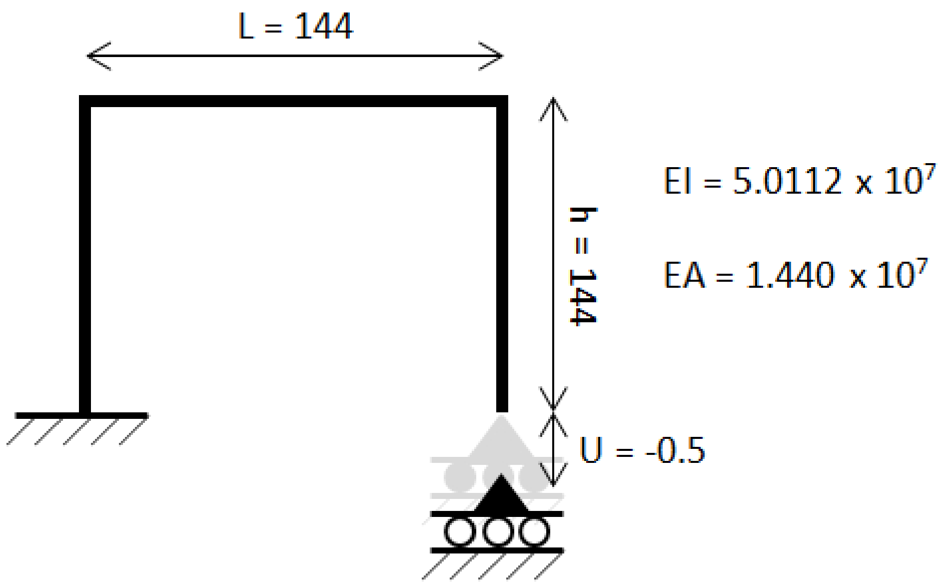
Conclusion

An exact match is shown between the solver and the independent analysis package.

1st Order Nonlinear - Displacement Loading of a Plane Frame

Problem Definition

Calculate the reaction forces of the plane moment frame shown below with the applied displacement U.



Assumptions

All elements are constant and equal EI. Axial and shear deformations are ignored; to achieve the former analytically the cross sectional area was increased by a factor of 100,000 to make axial deformation negligible.

Key Results

Results were compared with two other independent analysis packages.

Result	Comparison 1	Comparison 2	Solver Value
LHS Vertical Reaction	6.293	6.293	6.293
LHS Moment Reaction	-906.250	-906.250	-906.250
RHS Vertical Reaction	-6.293	-6.293	-6.293

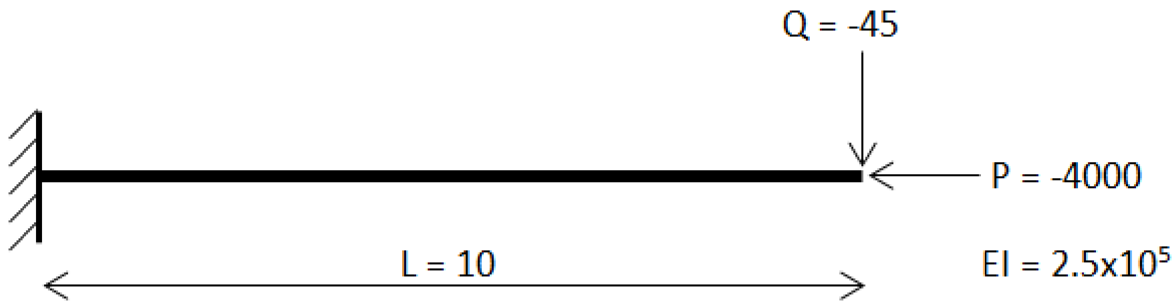
Conclusion

An exact match is shown between the solver and the two independent analysis packages.

2nd Order Linear - Simple Cantilever

Problem Definition

A 10 long cantilever is subjected to a lateral tip load of 45 and an axial tip load of 4000.



Assumptions

Shear deformations are ignored. Results are independent of cross section area; therefore any reasonable value can be used. Second order effects from stress stiffening are included, but those caused by update of geometry are not. The beam is modelled with only one finite element, (if more elements had been used the result would converge on a more exact value).

Key Results

Results were compared with an independent analysis package.

Result	Comparison	Solver Value
Tip Deflection	-0.1677	-0.1677
Base Moment Reaction	-1121	-1121

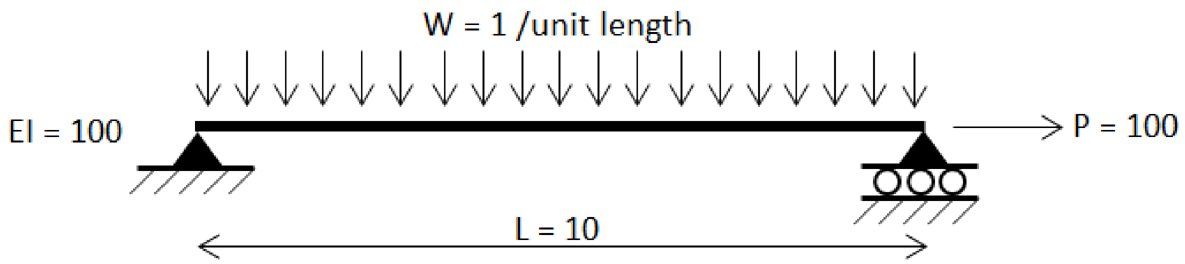
Conclusion

An exact match is observed between the values reported by the solver and the values reported in "*Comparison*".

2nd Order linear - Simply Supported Beam

Problem Definition

Determine the mid-span deflection and moment of the simply supported beam under transverse and tensile axial load.



Assumptions

Shear deformations are excluded. Results are independent of cross section area; therefore any reasonable value can be used. The number of internal nodes varies from 0-9.

Key Results

The theoretical value for deflection and moment are calculated as:

$$Y_{max} = -0.115 = \frac{5wL^4}{384EI} \times \frac{\frac{1}{\cosh U} - 1 + \frac{U^2}{2}}{\frac{5}{24}U^4}$$

$$M_{max} = -0.987 = \frac{wL^2}{8} \times \frac{2(\cosh U - 1)}{U^2 \cosh U}$$

Where *U* is a variable calculated:

No. internal nodes	Solver Deflection	Deflection Error %	Solver Moment	Moment Error %
1	-0.116	0.734%	-0.901	8.631%
3	-0.115	0.023%	-0.984	0.266%
5	-0.115	0.004%	-0.986	0.042%
7	-0.115	0.001%	-0.986	0.013%
9	-0.115	0.000%	-0.986	0.005%

Conclusion

As the element is subdivided the result converges to the correct theoretical value.

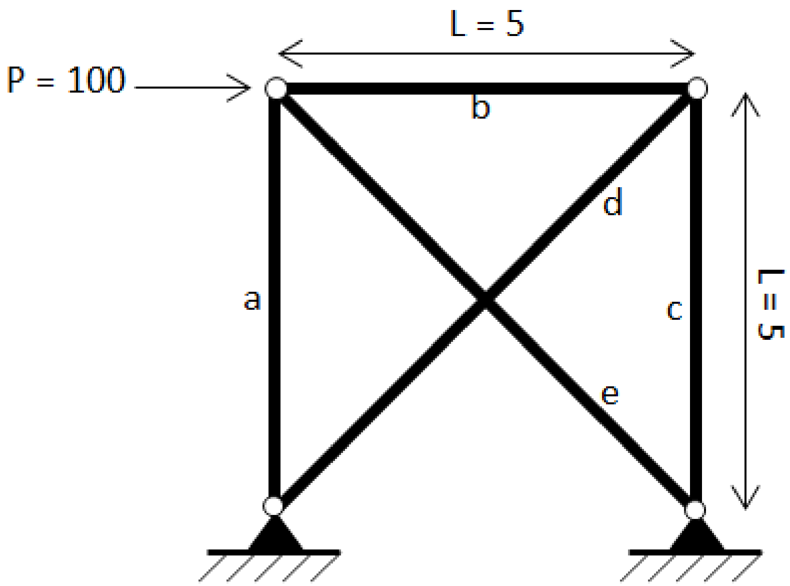
Reference

Timoshenko. S. 1956. *Strength of Materials, Part II, Advanced Theory and Problems*. 3rd Edition. D. Van Nostrand Co., Inc. New York, NY.

2nd Order Nonlinear - Tension Only Cross Brace

Problem Definition

Calculate the axial forces of the elements a-e shown in the 5x5 pin jointed plane frame shown below. Elements d and e can resist tensile forces only.



Assumptions

All elements are constant and equal EA. A smaller value of EA will increase the influence of second order effects, whereas a larger value will decrease the influence.

Key Results

Under the applied loading element e becomes inactive. The theoretical formulas presented below are obtained using basic statics. Note that a positive value indicates tension. These results assume no 2^{nd} order effects; this requires the value of EA to be sufficiently large to make the 2^{nd} order effect negligible.

Result	**Theoretical Formula**	**Theoretical Value**	**Solver Value**	**% Error**
a	0	0	0	0
b	-P	-100	-100	0
c	-P	-100	-100	0
d	$P\sqrt{2}$	141.42	141.42	0
e	0	0	0	0

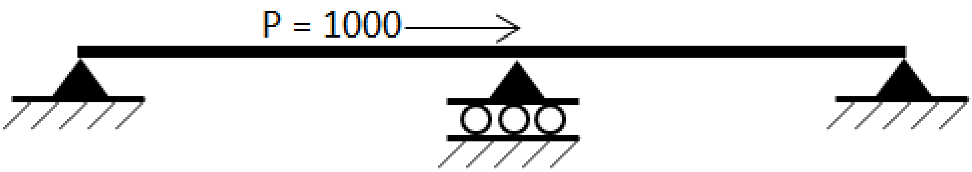
Conclusion

An exact match is observed between the values reported by the solver and the values predicted using statics. A 1^{st} order nonlinear analysis can be used, with any section sizes, to confirm this result without second order effects.

2nd Order Nonlinear - Compression Only Element

Problem Definition

Calculate the reaction forces for the compression only structure shown below.



Assumptions

All elements are constant and equal EA, and can resist only compressive forces

Key Results

Under the applied loading the element on the left becomes inactive, therefore all applied loading is resisted by the support on the right.

Result	Theoretical Formula	Theoretical Value	Solver Value
LHS Reaction	0	0	0
RHS Reaction	-P	-1000	-1000

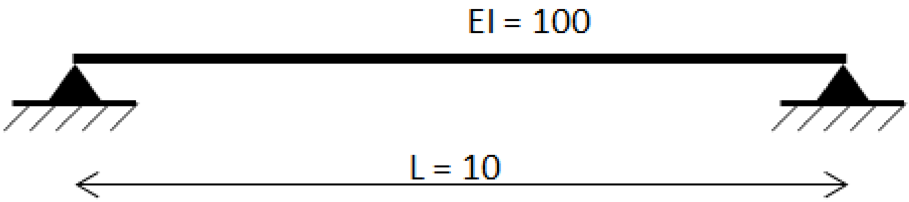
Conclusion

An exact match is observed between the values reported by the solver and the theoretical values.

1st Order Vibration - Simply Supported Beam

Problem Definition

Determine the fundamental frequency of a 10 long simply supported beam with uniform EI and mass per unit length equal to 1.0.



Assumptions

Shear deformations are excluded. The number of internal nodes varies from 0-5. Consistent mass is assumed.

Key Results

The theoretical value for the fundamental frequency is calculated as:

$$\omega = 0.9870 = \sqrt{\left(\frac{\pi}{10}\right)^4 \frac{100}{1}} = \sqrt{\left(\frac{\pi}{L}\right)^4 \frac{EI}{m/L}}$$

With *m* is the total mass of the beam.

No. internal nodes	Solver Value	% Error
0	1.0955	10.995%
1	0.9909	0.395%
2	0.9878	0.081%
3	0.9872	0.026%
4	0.9871	0.011%
5	0.9870	0.005%

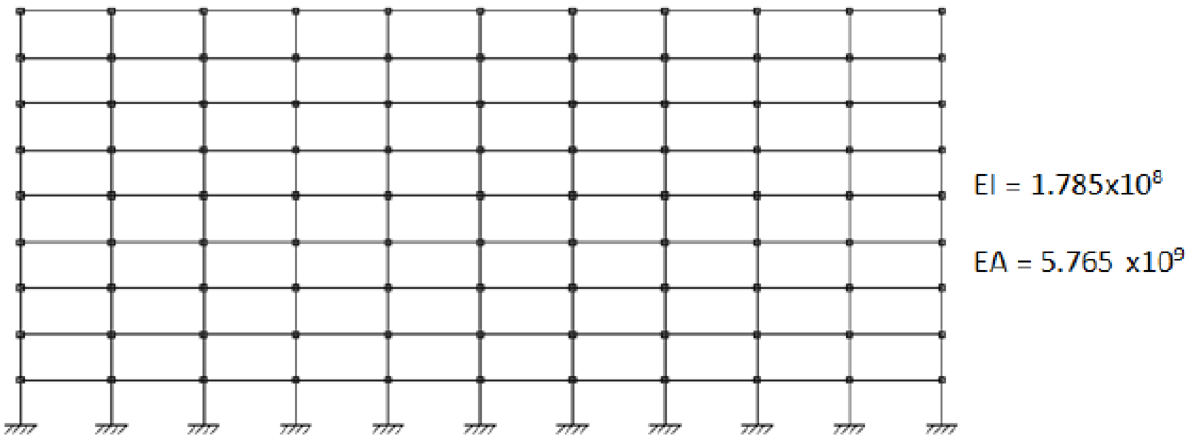
Conclusion

As the element is subdivided the result converges to the correct theoretical value.

1st Order Vibration - Bathe and Wilson Eigenvalue Problem

Problem Definition

A 2D plane frame structure has 10 equal bays each measuring 6.096m wide and 9 stories 3.048m tall. The column bases are fully fixed. All beams and columns are the same section, which have a constant mass/unit length equal to 1.438. Calculate the first three natural frequencies (in Hz) of the structure under self-weight.



Assumptions

Shear deformations are excluded. Each beam/column is represented by one finite element. Consistent mass is assumed.

Key Results

The results for this problem are compared with those published by Bathe and Wilson and against an independent analysis package.

Mode	Bathe and Wilson	Comparison	Solver Value
1	0.122	0.122	0.122
2	0.374	0.374	0.375
3	0.648	0.648	0.652

Conclusion

The results show a good comparison with the original published results and against the other analysis packages.

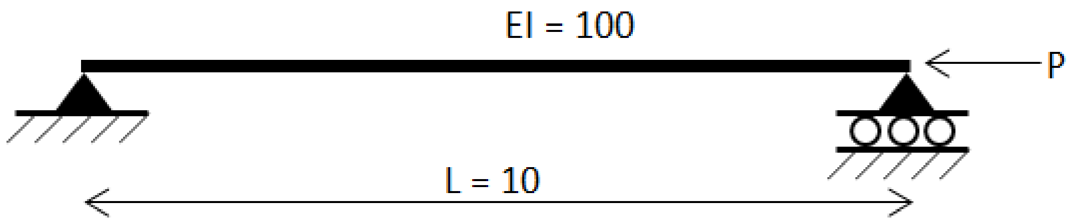
References

Bathe, K.J. and E.L. Wilson. 1972. *Large Eigen Values in Dynamic Analysis.* Journal of the Engineering Mechanics Division. ASCE Vol. 98, No. EM6. Proc. Paper 9433. December.

2nd Order Buckling - Euler Strut Buckling

Problem Definition

A 10 long simply supported beam is subjected to an axial tip load of P.



Assumptions

Shear deformations are excluded. The number of internal nodes varies from 0-5.

Key Results

The theoretical value for the first buckling mode is calculated using the Euler strut buckling formula:

$$\lambda = 9.869 = \frac{\pi^2 EI}{L^2}$$

With P = -1.0 the following buckling factors are obtained

No. internal nodes	Solver Value	% Error
0	12.000	21.59%
1	9.944	0.75%
2	9.885	0.16%
3	9.875	0.05%
4	9.872	0.02%
5	9.871	0.01%

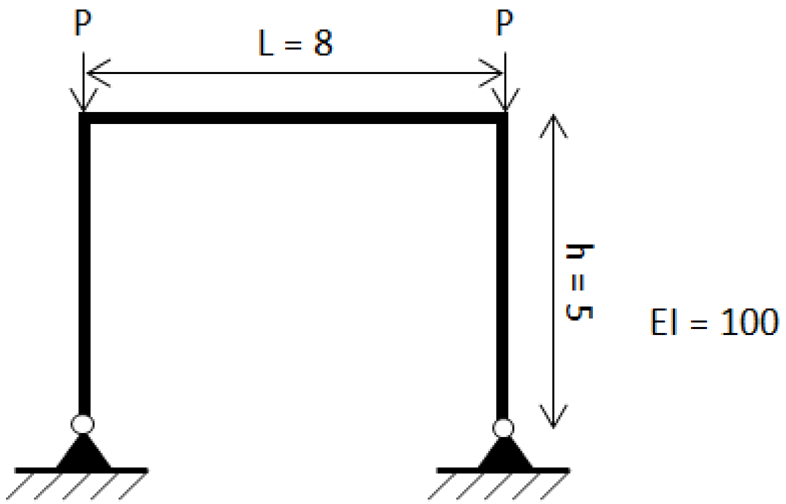
Conclusion

As the element is subdivided the result converges to the correct theoretical value.

2nd Order Buckling - Plane Frame

Problem Definition

Calculate the buckling factor of the moment frame shown below.



Assumptions

All elements are constant and equal EI. Axial deformations are ignored; to achieve this the cross section area is set to 1000. The number of elements per member is varied between 0 and 5.

Key Results

The theoretical buckling load is calculated by

$$P_{cr} = 6.242 = \frac{(kL)^2 EI}{h^2}$$

where

$$kL\tan(kL) = 1.249 = \frac{6h}{L}$$

Which can be solved using Newtons method and five iterations

No. internal nodes/member	Solver Value	% Error
0	6.253	0.17%
1	6.243	0.01%
2	6.242	0.00%
3	6.242	0.00%
4	6.242	0.00%
5	6.242	0.00%

Conclusion

A good match is shown between the solver and theory. The discrepancy decreases as the level of discretization is increased.

References

Timoshenko, S. and J. M. Gere. 1961. *Theory of Elastic Stability*. 2nd Edition. McGraw-Hill Book Company.

Loading -Eurocode 1

Eurocode Loading

This handbook provides a general overview of how loadcases and combinations are created in *Tekla Structural Designer* when the head code is set to the Base Eurocode, or Eurocode with a specific National Annex applied. The Eurocode Combination Generator is also described.

Nationally Determined Parameters (NDPs) (Eurocode)

The Eurocode has differing NDP's for the Eurocode (Base) and for each of Eurocode (UK), Eurocode (Irish) etc. These are defined in the relevant country's National Annex.

Gamma (γ) factors and psi (ψ) factors for each National Annex are listed below:

Combination gamma factors

Factor	EC Base value	UK Value	Irish Value	Singapore Value	Malaysia Value	Finland Value	Norway Value	Sweden Value
EQU combs								
$\gamma_{Gj,sup}$	1.10	1.10	1.10	1.10	1.10	$1.10k_{FI}$	1.2	$1.1\gamma_d$
$\gamma_{Gj,inf}$	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
γ_Q (fav)	1.5	1.5	1.5	1.5	1.5	$1.5k_{FI}$	1.5	$1.5\gamma_d$
STR combs								
$\gamma_{Gj,sup}$	1.35	1.35	1.35	1.35	1.35	$1.35k_{FI}$	1.35	$1.35\gamma_d$

$\gamma_{Gj,inf}$	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
γ_Q (fav)	1.5	1.5	1.5	1.5	1.5	$1.5k_{FI}$	1.5	$1.5\gamma_d$
ξ	0.85	0.925	0.85	0.925	0.925	0.85	0.89	0.89
GEO combs								
$\gamma_{Gj,sup}$	1.0	1.0	1.0	1.0	1.0	$1.0k_{FI}$	1.0	$1.1\gamma_d$
$\gamma_{Gj,inf}$	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
γ_Q	1.3	1.3	1.3	1.3	1.3	$1.3k_{FI}$	1.3	$1.4\gamma_d$

The k_{FI} factor used in the Finnish National Annex is set by specifying an appropriate Consequence Class in the Structure Properties (accessed via the Project Workspace).

- CC3 (high consequence for loss of human life or economic social or environmental consequences very great)
- CC2 (medium consequence for loss of human life or economic social or environmental consequences considerable)
- CC1 (Low consequence for loss of human life, economic, social or environmental consequences small or negligible)

The γ_d factor used in the Swedish National Annex is set by specifying an appropriate Reliability Class in the Structure Properties (accessed via the Project Workspace).

- RC3 (major risk of serious personal injury)
- RC2 (some risk of serious personal injury)
- RC1 (minor risk of serious personal injury)

psi factors

UK, Ireland, Singapore Malaysia

Factor	EC Base value			UK Value			Irish Value			Singapore Value			Malaysia Value		
	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2
Category A - imposed domestic/residential	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3
Category B - imposed office	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3

Category C - imposed congregation	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6
Category D- imposed shopping	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6
Category E- imposed storage	1.0	0.9	0.8	1.0	0.9	0.8	1.0	0.9	0.8	1.0	0.9	0.8	1.0	0.9	0.8
Category H- imposed roofs	0	0	0	0.7	0	0	0.6	0	0	0.7	0	0	0.7	0	0
Snow Loads $<$ 1000m	0.5	0.2	0	0.5	0.2	0	0.5	0.2	0	0.5	0.2	0	0.5	0.2	0
Snow Loads $>$ 1000m Snow $s_k \geq 2.75$ kN/m^2 Snow ≥ 2 kN/m^2 ie loads ($2 \leq s_k < 3$ kN/m^2)	0.7	0.5	0.2	0.5	0.2	0	0.5	0.2	0	0.5	0.2	0	0.5	0.2	0
Ice Loads	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Wind Loads	0.6	0.2	0	0.5	0.2	0	0.6	0.2	0	0.5	0.2	0	0.5	0.2	0
Thermal Loads	0.6	0.5	0	0.6	0.5	0	0.6	0.5	0	0.6	0.5	0	0.6	0.5	0

Nordic Countries

Factor	EC Base value			Finish Value			Norwegian Value			Swedish Value		
	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2
Category A - imposed domestic/residential	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3
Category B - imposed office	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3	0.7	0.5	0.3
Category C - imposed congregation	0.7	0.7	0.6	0.7	0.7	0.3	0.7	0.7	0.6	0.7	0.7	0.6
Category D- imposed shopping	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6	0.7	0.7	0.6
Category E- imposed storage	1.0	0.9	0.8	1.0	0.9	0.8	1.0	0.9	0.8	1.0	0.9	0.8
Category H- imposed	0	0	0	0	0	0	0	0	0	0	0	0

roofs												
Snow Loads (< 1000m) ($1 \leq s_k < 2$ kN/m^2)	0.5	0.2	0	0.7	0.4	0.2	0.7	0.5	0.2	0.6	0.3	0.1
Snow Loads > 1000m Snow $s_k \geq 2.75$ kN/m^2 Snow ≥ 2 kN/m^2 ie loads ($2 \leq s_k < 3$ kN/m^2)	0.7	0.5	0.2	0.7	0.5	0.2	0	0	0	0.7	0.4	0.2
Snow Loads ≥ 3kN/m^2	0	0	0	0	0	0	0	0	0	0.8	0.6	0.2
Ice Loads	0	0	0	0.7	0.3	0	0	0	0	0	0	0
Wind Loads	0.6	0.2	0	0.6	0.2	0	0.6	0.2	0	0.3	0.2	0
Thermal Loads	0.6	0.5	0	0.6	0.5	0	0.6	0.5	0	0.6	0.5	0

Seismic phi factors

Nordic Countries

An additional phi factor for Seismic design - from BS EN 1998-1 Table 4.2 is required for creation of the Seismic Inertia Combination. This is defined with each new imposed or roof imposed loadcase - alongside the psi factors for each of the load types.

Category	**Default for**		**Factor φ from BS EN 1998-1 Table 4.2**			
			EC	No Finnish NA so use EC	Norwegian	No Swedish NA so use EC
A - C	Roof Imposed	Roofs	1.0	1.0	1.0	1.0
A - C	Imposed = A, B, C	Stories with correlated occupancies	0.8	0.8	1.0	0.8
A - C		Stories independently occupied	0.5	0.5	1.0	0.5

D - F	Imposed = D, E	D-F and Archives	1.0	1.0	1.0	1.0

Load Cases (Eurocode)

Loadcase Types (Eurocode)

The following load case types can be created:

Loadcase Type	Calculated Automatically	Include in the Combination Generator	Imposed Load Reductions	Pattern Load
self weight (beams, columns and walls)	yes/no	yes/noI	N/A	N/A
slab wet	yes/no	N/A	N/A	N/A
slab dry	yes/no	yes/no	N/A	N/A
dead	N/A	yes/no	N/A	N/A
imposed	N/A	yes/no	yes/no	yes/no
roof imposed	N/A	yes/no	N/A	N/A
wind	N/A	yes/no	N/A	N/A
snow	N/A	yes/no	N/A	N/A
snow drift	N/A	yes/no	N/A	N/A
temperature	N/A	N/A	N/A	N/A
settlement	N/A	N/A	N/A	N/A
seismic	N/A	yes	N/A	N/A

As shown above, self weight loads can all be determined automatically. However other gravity load cases have to be applied manually as you build the structure.

Self Weight (Eurocode)

Self weight - excluding slabs loadcase

Tekla Structural Designer automatically calculates the self weight of the structural beams/columns for you. The **Self weight - excluding slabs** loadcase is pre-defined for this purpose. Its loadcase type is fixed as "Selfweight". It can not be edited and by default it is added to each new load combination.

Self weight of concrete slabs

Tekla Structural Designer expects the wet and dry weight of concrete slab to be defined in separate loadcases. This is required to ensure that members are designed for the correct loads at construction stage and post construction stage.

The **Slab self weight** loadcase is pre-defined for the dry weight of concrete post construction stage, its loadcase type is fixed as "Slab Dry".

There is no pre-defined loadcase for the wet weight of concrete slab at construction stage, but if you require it for the design of any composite beams in the model, the loadcase type should be set to "Slab Wet".

Tekla Structural Designer can automatically calculate the above weights for you taking into account the slab thickness, the shape of the deck profile and wet/dry concrete densities. It does not explicitly take account of the weight of any reinforcement but will include the weight of decking. Simply click the **Calc Automatically** check box when you create each loadcase. When calculated in this way you can't add extra loads of your own into the loadcase.

If you normally make an allowance for ponding in your slab weight calculations, *Tekla Structural Designer* can also do this for you. After selecting the composite slabs, you are able to review the slab item properties - you will find two ways to add an allowance for ponding (under the slab parameters heading). These are:

- as a value, by specifying the average increased thickness of slab
- or, as a percentage of total volume.

Using either of these methods the additional load is added as a uniform load over the whole area of slab.

Imposed and Roof Imposed Loads (Eurocode)

Definition of psi factors for imposed load cases

In the Loadcase dialog when an imposed loadcase is selected, you are able to select the Category of imposed load as follows - default Category B - office:

- Category A - domestic/residential
- Category B - office
- Category C - congregation
- Category D - shopping
- Category E - storage

The default values of ψ_0, ψ_1 and ψ_2 vary depending on the category selected and also with the National Annex being worked to. The values can be edited if required.

Definition of psi factors for roof imposed load cases

Roof imposed loads are not categorised so the default values of ψ_0, ψ_1 and ψ_2 only vary depending on the National Annex being worked to. Again, the values can be edited if required.

Imposed Load Reductions (Eurocode)

Reductions can be applied to imposed loads to take account of the unlikelihood of the whole building being loaded with its full design imposed load. Reductions can not however be applied to roof imposed loads.

If the imposed load is considered as an accompanying action (i.e. a ψ factor is applied to the imposed load case in a combination) then as stated in the Base Eurocode Cl3.3.2, the imposed load reduction should not be applied at the same time.

Imposed loads are only automatically reduced on:

- Vertical columns (both RC and steel)
- Vertical walls (RC)

The method used for determining the reductions is dependant on the National Annex:

- In the Base Eurocode a formula is given in Clause 6.3.1.2(11), this is also used if the Irish, Finish, Norwegian, Swedish or Singaporean National Annex is selected.
- In the UK, and Malaysia the NA permits an alternative method of reduction using NA 2.6.

Although the code allows for imposed load reductions to be applied to floors, *Tekla Structural Designer* does not implement this automatically. For concrete beams, slabs and mats it is however possible to define the level of imposed load reduction manually via the beam/slab item properties.

This is particularly relevant for the design of transfer beams/slabs:

- The imposed load reduction for beams, slabs and mats is intended to work with loads applied from columns acting on the beam or slab when the slab is acting in transfer or for a mat foundation supporting a column. (The theory being that if you want to design the columns for the reduced axial load, you should also design the supporting member for the reduced axial load applied by the column.)
- The engineer would need to work out the reduction of the axial load in the column and apply this as a the reduction percentage, i.e. if the raw axial load in the column is 100kN and the reduced load is 60kN, the reduction is 40%. You would than apply the 40% reduction to the transfer beam/slab or mat as well.
- The reduction is not applied to loads for analysis - it is a post-analysis process which does not affect the analysis results. It does not get applied solely to the imposed load applied directly to the beam or slab panel, but instead is applied to the design moment used in the beam/slab or mat design process.

Snow and Snow Drift Loads (Eurocode)

Definition of psi factors for snow and snow drift load cases

In the Loadcase dialog when a snow, snow drift, or ice loadcase is selected, the default values of ψ_0, ψ_1 and ψ_2 are displayed. These vary depending on the National Annex being worked to. The values can be edited if required.

Snow drift loads are considered to be accidental load cases and are combined in the Accidental combinations.

Wind Loads (Eurocode)

The EC1-4 Wind Wizard

The Wind Wizard used for automatic wind loadcase generation is fully described in the Wind Modelling Engineer's Handbook.

The **Wind Wizard** is run to create a series of static forces that are combined with other actions due to dead and imposed loads in accordance with BS EN 1990.

The following assumptions/limitations exist:-

- The shape of the building meets the limitations allowed for in the code.
- It must be a rigid structure.
- The structure must be either enclosed or partially enclosed.
- Parapets and roof overhangs are not explicitly dealt with.

Simple Wind Loading

If use of the Wind Wizard is not appropriate for your structure then wind loads can be applied via element or structure loads instead.

Definition of psi factors for wind load cases

In the Loadcase dialog when a wind loadcase is selected, the default values of ψ_0, ψ_1 and ψ_2 are displayed. These vary depending on the National Annex being worked to. The values can be edited if required.

Combinations (Eurocode)

Once your load cases have been generated as required, you then combine them into load combinations; these can either be created manually, by clicking **Add... -** or with the assistance of The Combinations Generator, by clicking **Generate...**

The Foreword to the Singapore National Annex to EN 1991-1-4 Wind Actions has a minimum horizontal load requirement (1.5% characteristic dead weight). Therefore if this National Annex has been applied, we are assuming that the wind load applied in manually defined combinations, or via the combination generator, satisfies this minimum horizontal load requirement. See: Minimum lateral load requirements of the Singapore National Annex

Manually Defined Combinations (Eurocode)

As you build up combinations manually, the combination factors are automatically adjusted as load cases are added and removed from the combination.

If you add/remove a load case type from a combination - the factors are defaulted as follows:

- 'Self weight' - default Strength factor = 1.35, default Service factor = 1.0
- 'Slab Dry' - default Strength factor = 1.35, default Service factor = 1.0
- 'Dead' - default Strength factor = 1.35, default Service factor = 1.0
- 'Imposed'- default Strength factor = 1.5, default Service factor = 1.0
- 'Roof Imposed'- default Strength factor = 1.05, default Service factor = 1.0
- With an Imposed load case
  - 'Wind' - default Strength factor = 0.75, default Service factor = 0.5
  - 'Snow' - default Strength factor = 0.75, default Service factor = 0.5
- With No Imposed load case
  - 'Wind' - default Strength factor = 1.5, default Service factor = 1.0
  - With Wind load case
    - 'Snow' - default Strength factor = 0.75, default Service factor = 0.5
  - With no Wind load case
    - 'Snow' - default Strength factor = 1.5, default Service factor = 1.0
- 'Temperature'- default Strength factor = 1.0, default Service factor = 1.0
- 'Settlement'- default Strength factor = 1.0, default Service factor = 1.0

Equivalent Horizontal Forces (EHF) (Eurocode)

EHFs are used to represent frame imperfections. The Eurocode requires they are applied to all combinations. (Lateral wind combinations therefore should also have EHFs applied).

EHFs are automatically derived from the factored load cases within the current combination. They are applied in the analysis as a horizontal force at each beam column intersection as a specified percentage of the vertical load in the column at the column/beam intersection.

Settings that control the EHF percentage can be adjusted from **Home > Model Settings > EHF**. (The default settings conservatively result in 0.5% EHF in both directions).

EHFs are applied to the structure in the building directions 1 and 2 as follows:

- EHF Dir1+
- EHF Dir1-
- EHF Dir2+
- EHF Dir2-

The Combinations Generator (Eurocode)

Accessed via the **Generate...** button, this automatically sets up combinations for both strength and serviceability.

Combination Generator - Initial Parameters

At the start of the generator, you need to define certain parameters so that the correct combinations are created - these are described below:

Combination for design of structural members (STR)

You can chose between:

- Table A1.2(B) - Eq 6.10, or
- Table A1.2(B) - Eq 6.10,a&b

Eq 6.10 is always equal to or more conservative than either 6.10a or 6.10b. The most economic combination of 6.10a or b will depend on if permanent actions are greater than 4.5 times the variable actions (except for storage loads).

Include GEO combinations - Table A1.2(C) - Eq 6.10

You should check this option in order to create the GEO combinations required for foundation design.

Include Accidental combinations - Table A2.5 Eq 6.11a&b

If you have defined an accidental load type such as Snow drift you should check this option for the correct load combinations to be generated.

The Combinations Generator refers to the relevant National Annex when determining the g factors to apply in the above combinations, as they may vary from the Base Eurocode values.

Include Seismic combinations - Table A2.5 Eq 6.12a&b

If you have defined seismic loads you should check this option for the correct load combinations to be generated.

Temperature and settlement load case types not included in the Generator at all - these have to be added manually.

Combination Generator - Combinations

The second page of the generator lists the combinations applicable (with appropriate factors) for the selections made on the first page. Any factors in bold will be multiplied by the relevant psi factors for that load case.

The type of structure chosen on the previous page affects which combinations default to being generated.

The combination names are automatically generated as per the table below:

No.	BS EN 1990 State and Eqn	Type	Load Combination
1	**Str – 6.10**	Gravity	$Str_1 - \gamma_{Gj,sup}D+\gamma_Q I+ \gamma_Q RI$
2	“	“	$Str_2 - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S$
3	“	Lateral (EHF)	$Str_{3.n} - \gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q RI+EHF$
4	“	“	$Str_{4.n} - \gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q\psi_0 S+EHF$
5	“	“	$Str_{5.n} - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S+EHF$
6	“	Lateral (Wind)	$Str_{6.n} - \gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q\psi_0 S+\gamma_Q\psi_0 W+EHF$
7	“	“	$Str_{7.n} - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S+\gamma_Q\psi_0 W+EHF$
8	“	“	$Str_{8.n} - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 S+\gamma_Q W+EHF$
9	“	Uplift	$Str_{9.n} - \gamma_{Gj,inf}D+\gamma_Q W+EHF$
1	**Str – 6.10a&b**	Gravity	$Str_1 - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+ \gamma_Q\psi_0 RI$
2	“	“	$Str_2 - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 S$
3	“	“	$Str_3 - \xi\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q RI$
4	“	“	$Str_4 - \xi\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S$
5	“	Lateral (EHF)	$Str_{5.n} - \gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 RI+EHF$

6	"	"	Str$_{6.n}$ - $\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 S+EHF$
7	"	"	Str$_{7.n}$ - $\xi\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q RI+EHF$
8	"	"	Str$_{8.n}$ - $\xi\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q\psi_0 S+EHF$
9	"	"	Str$_{9.n}$ - $\xi\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S+EHF$
10	"	Lateral (Wind)	Str$_{10.n}$ - $\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 S+\gamma_Q\psi_0 W+EHF$
11	"	"	Str$_{11.n}$ - $\xi\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q\psi_0 S+\gamma_Q\psi_0 W+EHF$
12	"	"	Str$_{12.n}$ - $\xi\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S+\gamma_Q\psi_0 W+EHF$
13	"	"	Str$_{13.n}$ - $\xi\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 S+\gamma_Q W+EHF$
14	"	Uplift	Str$_{14.n}$ - $\gamma_{Gj,inf}D+\gamma_Q W+EHF$
1	**Geo - 6.10**	Lateral (EHF)	Geo$_{1.n}$ - $\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q RI+EHF$
2	"	"	Geo$_{2.n}$ - $\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q\psi_0 S+EHF$
3	"	"	Geo$_{3.n}$ - $\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S+EHF$
4	"	Lateral (Wind)	Geo$_{4.n}$ - $\gamma_{Gj,sup}D+\gamma_Q I+\gamma_Q\psi_0 W+\gamma_Q\psi_0 S+EHF$
5	"	"	Geo$_{5.n}$ - $\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q S+\gamma_Q\psi_0 W+EHF$
6	"	"	Geo$_{6.n}$ - $\gamma_{Gj,sup}D+\gamma_Q\psi_0 I+\gamma_Q\psi_0 S+\gamma_Q W+EHF$
7	"	Uplift	Geo$_{7.n}$ - $\gamma_{Gj,inf}D+\gamma_{Q,1}W+EHF$
1	**Acc 6.11**	Lateral (EHF)	Acc$_{1.n}$ - $G+A_d+\psi_1 I+EHF$
2	"	Lateral (Wind)	Acc$_{2.n}$ - $G+A_d+\psi_2 I+\psi_1 W+EHF$
	Seis 6.12	Seismic	Seis$_{.n}$ - $G+A_{Ed}+\psi_2 RI+\psi_2 S+EHF$
			Seis$_{.n}$ - $G+A_{Ed}+EHF$

Combination Generator - Service Factors

This page indicates which combinations are to be checked for serviceability and the factors applied.

Combination Generator - Wind/EHF Directions

This page is used to select which EHF direction goes with each combination containing a specific wind load case.

All wind load cases are listed vertically, and the four EHF options (+Dir1, -Dir1, +Dir2, -Dir2) are each displayed with a factor (default 1.000).

By default (on first entry), none of the directions are set for any wind load case. You are required to set at least one for every wind load case and can set two, three or all four if you wish- these are then used when generating the combinations.

Combination Generator - EHF

The last page is used to set up the equivalent horizontal forces. You can specify EHF’s and factors in each of four directions. For each direction selected a separate EHF combination will be generated.

Any combination with wind in is automatically greyed as all the required information has already been set via the previous page.

Click **Finish** to see the list of generated combinations.

Combination Classes (Eurocode)

Having created your combinations you classify them as: Construction Stage, Gravity, Lateral, Seismic or Vibration Mass.

If generated via the Combinations Generator they are classified for you automatically.

Then (where applicable) you indicate whether they are to be checked for strength or service conditions, or both.

You also have the option to make any of the combinations inactive.

Construction Stage Combination (Eurocode)

A *Construction Stage* load combination is only required for the purpose of designing any composite beams within the model. It is distinguished from other combinations by setting its “Class” to **Construction Stage**.

Typically this combination would include a loadcase of type “Slab Wet”, (not “Slab Dry”), other loadcases being included in the combination as required.

If you add/remove a load case type from this combination - the factors are defaulted as follows:

- **Self weight** - default Strength factor = 1.35, default Service factor = 1.0
- **Slab Wet** - default Strength factor = 1.35, default Service factor = 1.0

- **Dead** - default Strength factor = 1.35, default Service factor = 1.0
- **Imposed** - default Strength factor = 1.5, default Service factor = 1.0

The Slab Wet loadcase type should not be included in any other combination.

Gravity Combinations (Eurocode)

These combinations are considered in both the Gravity Sizing and Full Design processes.

They are used in the Gravity Sizing processes as follows:

- **Design Concrete (Gravity)** - concrete members in the structure are automatically sized (or checked) for the gravity combinations.
- **Design Steel (Gravity)** - steel members in the structure are automatically sized (or checked) for the gravity combinations.
- **Design All (Gravity)** - all members in the structure are automatically sized (or checked) for the gravity combinations.

They are also used during the Full Design processes as follows:

- **Design Concrete (All)** - concrete members in the structure are automatically sized (or checked) for the gravity combinations.
- **Design Steel (All)** - steel members in the structure are automatically sized (or checked) for the gravity combinations.
- **Design All (All)** - all members in the structure are automatically sized (or checked) for the gravity combinations.

Quasi Permanent SLS Gravity Combination

In order to cater for the quasi-permanent SLS load combination, a gravity combination is permitted to have two SLS sets of factors.

The quasi permanent combination is only used for the spacing of reinforcement calculation for RC beams (and nothing else).

Lateral Combinations (Eurocode)

These combinations are **not** used in the Gravity Sizing processes.

They are used during the Full Design processes as follows:

- **Design Concrete (All)** - concrete members in the structure are automatically sized (or checked) for the lateral combinations.
- **Design Steel (All)** - steel members in the structure which have **not** been set as *Gravity Only* are automatically sized (or checked) for the lateral combinations.
- **Design All (All)** - all concrete members and all steel members which have **not** been set as *Gravity Only* are automatically sized (or checked) for the lateral combinations.

Seismic Combinations (Eurocode)

Although included in this documentation, these are only available for use in regions where seismic design is required.

These combinations are only considered during the Full Design process. They are **not** used in the Gravity Sizing process.

Vibration Mass Combinations (Eurocode)

For vibration analysis, you are required to set up specific “vibration mass” combinations. Provided these combinations are active they are always run through the vibration analysis.

It is always assumed that all loads in the load cases in the combination are converted to mass for vibration analysis.

You are permitted to add lumped mass directly to the model.

Minimum lateral load requirements of the Singapore National Annex (Eurocode)

The foreword to the “Singapore National Annex to EN 1991-1-4 Wind Actions” states:

> “*For continuation of an established design philosophy, all buildings should be capable of resisting, as a minimum, a design ultimate horizontal load applied at each floor or roof level simultaneously equal to 1.5% of the characteristic dead weight of the structure between mid-height of the storey below and either mid-height of the storey above or roof surface. The design ultimate wind load should not be taken as less than this value when considering load combinations*.”

In *Tekla Structural Designer* this requirement can be met by applying lateral loads at each floor level equal to 1.5% of the dead load at that level; these can then be designed for if they exceed the design ultimate wind load.

The checking procedure to be followed can be summarised as:

1. Establish the minimum lateral load to be resisted.
2. Compare the horizontal reaction this produces against that of the existing wind load.
3. If the minimum lateral load is greater than the wind load it must be designed for as necessary in up to four directions (+/- Dir 1, +/- Dir 2); if it is less, then no further action is required.

The checking procedure in detail

EN1994 requires all wind combinations to include EHFs - Settings that control the magnitude of EHFs can be adjusted from Home > Model Settings > EHF. (The default settings conservatively result in 0.5% EHF in both directions). It is recommended that these settings are reviewed prior to undertaking the procedure described below.

(Otherwise, if the EHF settings are subsequently changed, both the wind load combinations, and the factors applied to the minimum lateral load combinations are affected and consequently steps 2 and 3 of the checking procedure would have to be repeated.)

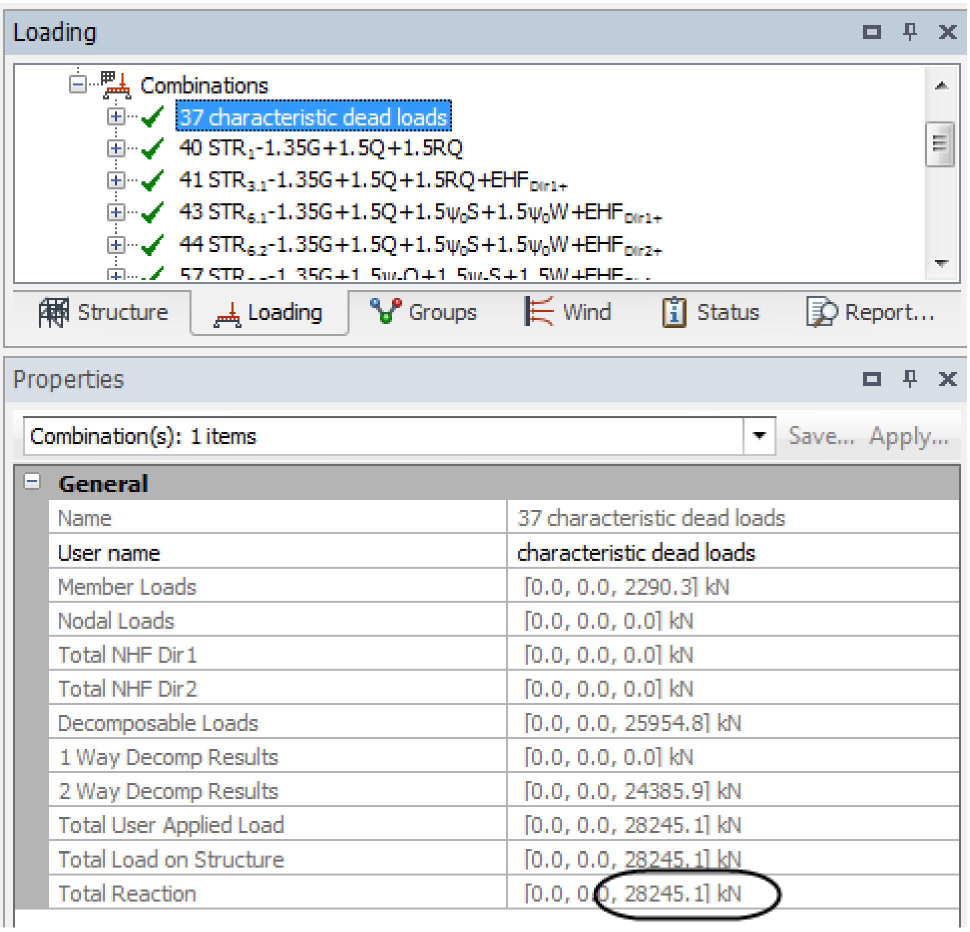
Step 1. Establish the minimum lateral load to be resisted

This can be determined as follows:

1. Create a special "characteristic dead loads" combination which only comprises the total dead weight of the structure and no EHFs. Make this combination "Active", but leave "Strength" and "Service" unchecked.

2. From the Analyse menu run **1st Order Linear** for all combinations.

3. From the Project Workspace Loading tab, select the "characteristic dead loads" combination and make a note of the total vertical reaction.



In the example shown above the vertical reaction = 28245 kN

4. Calculate 1.5% of this value – this is the minimum lateral load to be resisted.

   H_{min} = 0.015 * 28245 = 423.7kN

Step 2. Compare the minimum lateral load against the wind load

1. Still in the Project Workspace Loading tab, click on each of the wind load combinations and compare their lateral reactions against H_{min}

2. If H_{min} is greater than the maximum lateral reaction from all of the wind combinations, this indicates that the minimum lateral load governs and consequently you must ensure that the building is designed for this condition. (If it is not, then minimum lateral load does not govern and no further action is required.)

Step 3. Create (and design for) the minimum lateral load design combinations

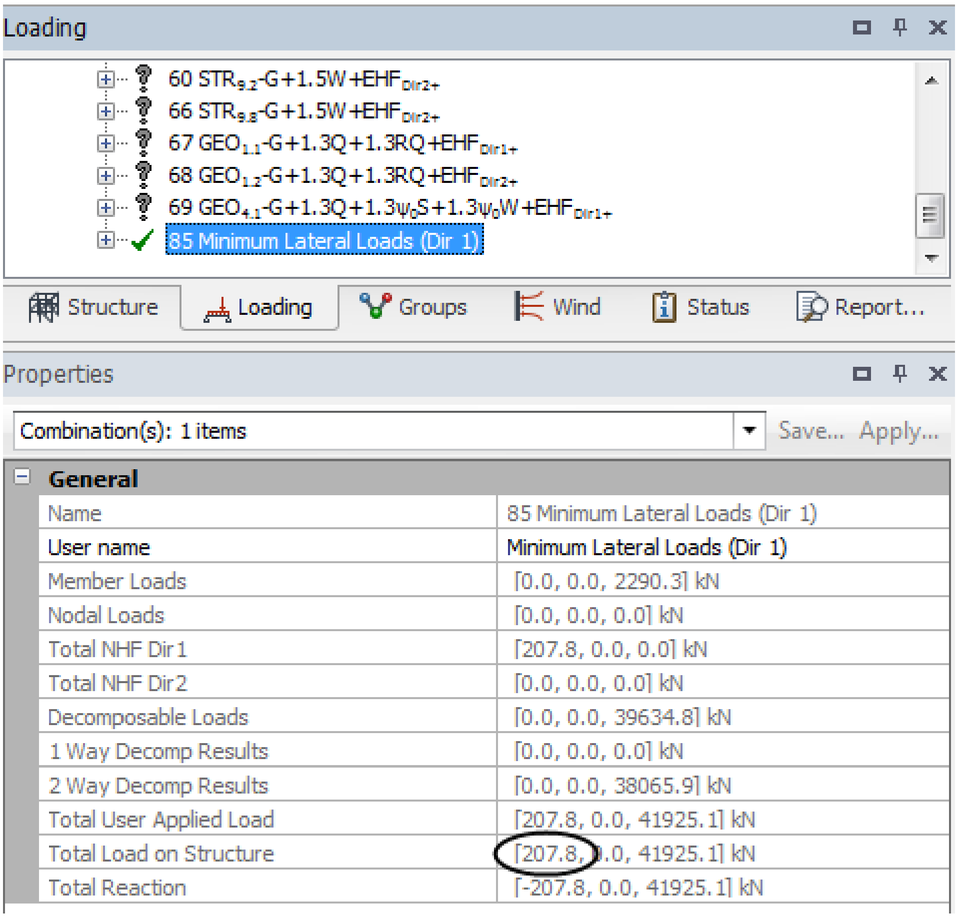
Assuming that the above comparison has established that the minimum lateral load governs, you will have to create minimum lateral load combinations in each of four directions (+/- Dir 1, +/- Dir 2) as follows:

1. Copy the existing dead and imposed only combination to create a new combination named "Minimum Lateral Loads (Dir 1+)"

2. Ensure the new combination includes EHF in (Dir 1+) only

3. The EHF strength factor has to be adjusted to generate H_{min} laterally - to do this:

- From the Analyse menu run **1st Order Linear** for this new combination.
- Record the total horizontal load on the structure, (H_1).



In the example shown above the total horizontal load, H_1 = 207.8kN

- Calculate the EHF strength factor required as the ratio: H_{min}/H_1

EHF strength factor = 423.7/ 207.8 = 2.04

4. Create a second minimum lateral load combination, “Minimum Lateral Loads (Dir 1-)”, this is similar to the first, using the same adjusted EHF strength factor, but with the EHF (Dir1+) loadcase replaced by EHF(Dir1-)

5. Repeat the above process to create similar minimum lateral load combinations in direction 2.

6. Run **Design All (Static)** to design the model for all combinations.

Concrete Design - Eurocode 2

Concrete Design to EC2

This handbook describes how BS EN 1992-1-1:2004 (Ref. 1) is applied to the design of concrete members in *Tekla Structural Designer*.

Unless explicitly noted otherwise, all clauses, figures and tables referred to are from BS EN 1992-1-1:2004

Within the remainder of this guide BS EN 1992-1-1:2004 is referred to as EC2.

Design Codes and Variants

Code Variants

The current release includes design in accordance with:

- Eurocode 2 **and** the following National Annexes
    - UK
    - Ireland
    - Malaysia
    - Singapore

Loading Codes

Loading codes appropriate to the design code variants will be used

General Parameters

Shrinkage and Creep

The following design parameters can be specified individually as part of each member's properties set.

Permanent Load Ratio

This is the ratio of quasi-permanent load to design ultimate load.

i.e. SLS/ULS = (1.0Gk + ψ_2Qk) / (factored Gk + factored Qk*IL reduction)

If Qk is taken as 0 then:

SLS/ULS = (1 / 1.25) = 0.8

Hence, setting the permanent load ratio to 0.8 should provide a conservative upper bound for all cases.

When determining this ratio more precisely, consideration should be given to the amount of IL reduction specified, for example (assuming Gk = Qk and ψ_2 = 0.3):

For 50%IL reduction,

SLS/ULS = (1 + 0.3) / (1.25 + 1.5*0.5) = 0.65

For no IL reduction,

SLS/ULS = (1 + 0.3) / (1.25 + 1.5) = 0.47

The program defaults to a permanent load ratio of 0.65 for all members - you are advised to consider if this is appropriate and adjust as necessary.

Relative Humidity

Typical input range 20 to 100%

Age of Loading

This is the age at which load is first applied to the member.

The Age of Loading should include adjustments necessary to allow for cement type and temperature as defined in EC2 Annex B.

The program defaults the Age of Loading to 14 days for all members - you are advised to consider if this is appropriate and adjust as necessary.

Reinforcement Anchorage Length Parameters

Max. Bond Quality Coefficient

Acceptable input range 0.5 to 1.0

In the bond stress calculation (Cl 8.4.2), the bond quality coefficient $\eta 1$ can be either 1.0 or 0.7 depending on section depth. Where 0.7 is used the bond strength is reduced and laps are extended.

Specifying a maximum of 1.0 for the Bond Quality Coefficient allows the coefficient to vary between 0.7 and 1.0 as required, hence lap lengths will vary accordingly.

Some users may prefer to specify a maximum of 0.7 (which actually fixes the coefficient at 0.7), the effect is to standardise on the use of extended lap lengths throughout. Further conservatism can be introduced in all lap lengths by using a value as low as 0.5.

Plain Bars Bond Quality Modifier

Acceptable input range 0.1 to 1.0

In the EC2 Cl 8.4.2 bond stress calculation, there is no factor relating to the rib type of reinforcement, and no guidance on what adjustments if any should be made for plain bars.

In *Tekla Structural Designer* a factor “T” has been introduced (as in BS8110) to allow for this adjustment. It is the users responsibility to enter a suitable value for plain bars. (Until further guidance becomes available, we would suggest that as per BS8110 a value of 0.5 would be reasonable.)

Type-1 Bars Bond Quality Modifier

Acceptable input range 0.1 to 1.0

In the EC2 Cl 8.4.2 bond stress calculation, there is no factor relating to the rib type of reinforcement, and no guidance on what adjustments if any should be made for Type 1 bars.

In *Tekla Structural Designer* a factor “T” has been introduced (as in BS8110) to allow for this adjustment. It is the users responsibility to enter a suitable value for Type 1 bars. (Until further guidance becomes available, we would suggest that as per BS8110 a value of 0.8 would be reasonable.)

Beam Design to EC2

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Limitations and Exclusions (Beams: EC2)

The following general exclusions apply:

- Seismic design,

- Consideration of fire resistance. [You are however given full control of the minimum cover dimension to the reinforcement and are therefore able to take due account of fire resistance requirements.]
- Openings in the beam web.
- Bundled bars.
- Lightweight concrete.
- Design for minor axis bending and shear.
- Design for axial forces.

In addition, for beams classified as “deep beams”:

- all beams with a ratio of 1.5 < span/overall depth ≤ 3.0 are designed but with an appropriate Warning
- beams with a ratio of span/overall depth ≤ 1.5 are *Beyond Scope*

Materials (Beams: EC2)

Concrete

Only normal weight is included in the current release. (Lightweight concrete is excluded).

Reinforcement

The reinforcement options are:

- Loose reinforcing bars,
- Loose reinforcing bars bent to form links.

Slender Beams (Beams: EC2)

Second order effects associated with lateral instability may be ignored if beams are within the geometric limits given by the following;

$L_{0t} \leq 50*b_{comp}/(h/b_{comp})^{1/3}$

and

$h/b_{comp} \leq 2.5$

where

L_{0t} = the distance between torsional restraints, which in *Tekla Structural Designer* is taken as the distance between the faces of the supports

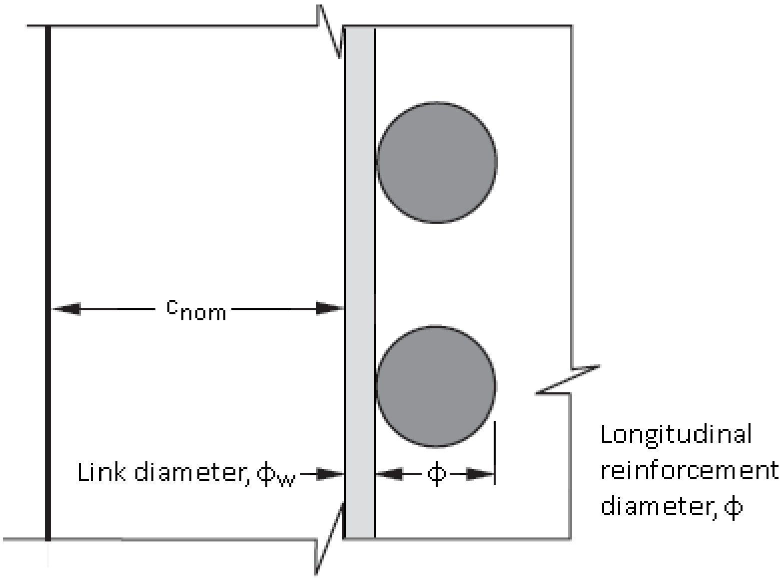
h = the total overall depth of the beam at the centre of L_{0t}

b_{comp} = the width of the compression flange of the beam (= b_w for rectangular sections and b_{eff} for flanged beams)

If either of the above checks fail then a Warning is displayed.

Cover to Reinforcement (Beams: EC2)

The nominal concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and surface reinforcement where relevant) and the nearest concrete surface.



You are required to set a minimum value for the nominal cover, $c_{nom,\,u}$, for the top, bottom, sides and ends of each beam in the beam properties.

These values are then checked against the nominal limiting cover, $c_{nom,lim}$ which depends on the diameter of the reinforcement plus an allowance for deviation, Δc_{dev} (specified in **Design Options > Beam > General Parameters**).

Generally, the allowance for deviation, Δc_{dev} is a NDP.[1] The recommended value is 10mm, but under strict controls it can be reduced to 5mm.

If $c_{nom,u} < c_{nom,lim}$ then a warning is displayed in the calculations.

1. BS EN 1992-1-1:2004 cl 4.4.1.3 (1)P

Design Parameters for Longitudinal Bars (Beams: EC2)

For each of these parameters, the user defined limits (specified in **Design Options > Beam > Reinforcement Settings**) are considered in addition to the EC2 or NA recommendations.

Minimum and Maximum Diameter of Reinforcement

At Section 8.8 of BS EN 1992-1-8:2004, additional rules are specified when “large diameter bars” are used in the design. A large diameter bar is defined as being a bar with a diameter larger than φ_{large} where φ_{large} is an NDP value.

For design in accordance with **EC2 Recommendations**;

φ_{large} = 32 mm

For design in accordance with **UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

φ_{large} = 40 mm

In the current release the provisions of Section 8.8 **are not** implemented. If the design results in a bar size with $\varphi > \varphi_{large}$ then a warning is displayed.

Clause 7.3.3 (2) indicates that cracking can be controlled either by restricting the bar diameter or the max spacing. Tekla Structural Designer adopts the latter approach using Table 7.3N- therefore the maximum bar diameters specified in Table 7.2N are not checked.

Minimum Distance between Bars

The minimum **clear** horizontal distance between individual parallel bars, $s_{cl,min}$, is given by;[1]

$s_{cl,min} \geq$ MAX [$k_1 * \varphi$, $d_g + k_2$, $s_{cl,u,min}$, 20 mm]

where

k_1 = the appropriate NDP

k_2 = the appropriate NDP

d_g = the maximum size of aggregate

φ = the maximum diameter of adjacent bars, φ_i and φ_j

$s_{cl,u,min}$ = user specified minimum clear distance between bars

To allow you to make decisions regarding access for concrete compaction or size of aggregate, a value for the minimum clear distance between bars is specified in **Design Options > Beam > Reinforcement Settings** - separate values being set for bars in the top of the beam and for those in the bottom of the beam.

The minimum **clear** vertical distance between horizontal layers of parallel bars, $s_{cl,min}$, is given by;

$s_{cl,min} \geq$ MAX[$k_1 * \varphi$, $d_g + k_2$, 20 mm]

For design in accordance with **UK NA,EC2 Recommendations, Irish NA**, **Malaysian NA** and **Singapore NA**;

k_1 = 1.0

k_2 = 5.0 mm

Maximum Spacing of Tension Bars

The maximum centre to centre bar spacing for crack control, $s_{cr,max}$, is dependent on the maximum allowable crack width, w_{max}, specified in the beam properties from a menu of values which are: 0.20mm, 0.30mm or 0.40mm with a default value of 0.30mm.

The service stress in the reinforcement, σ_s, is given by;

σ_s = $(A_{s,reqd}/A_{s,prov})*(f_{yk}/\gamma_s)*R_{PL}$

where

$A_{s,reqd}$ = the area of reinforcement required for the maximum design Ultimate Limit State bending moment, M_{Ed}

$A_{s,prov}$ = the area of reinforcement provided

R_{PL} = the permanent load ratio

In the beam properties you are required to supply a value for the permanent load ratio, R_{PL}. A default of 0.65 has been assumed, but you are advised to consider if this is appropriate and adjust as necessary.

The maximum allowable centre to centre bar spacing, $s_{cr,max}$, is then obtained from table 7.3N (shown below) by looking up the calculated value of the service stress in the reinforcement, σ_s, using interpolation between values of σ_s

Steel Service Stress, σ_s (N/mm^2)	**Max Allowable bar Spacing, $s_{cr,max}$**		
	w_{max} = 0.40 mm	**w_{max} = 0.30 mm**	**w_{max} = 0.20 mm**
≤ 160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	Warning
360	100	50	Warning
>360	Warning	Warning	Warning

Minimum Area of Reinforcement

The minimum area of longitudinal tension reinforcement, $A_{s,min}$, is given by;[2]

$$A_{s,min} \geq MAX[k_{min1}*b_w*d*(f_{ctm}/f_{yk}),\ k_{min2}*b_w*d]$$

where

k_{min1} = the appropriate NDP value

k_{min2} = the appropriate NDP value

f_{ctm} = mean value of the axial tensile strength of the concrete

f_{yk} = characteristic yield strength of the reinforcement

For design in accordance with **UK NA,EC2 Recommendations, Irish NA**, **Malaysian NA** and **Singapore NA**;

k_{min1} = 0.26

k_{min2} = 0.0013

Note that there is no requirement to have a minimum area of compression reinforcement.

The minimum area of longitudinal tension reinforcement for crack control, $A_{s,min,cr}$ is given by;[3]

$A_{s,min,cr} \geq 0.4*k*f_{ctm}*A_{ct}/\sigma_s$

where

k = 1.0 when h ≤ 300

= 0.65 when h ≥ 800

f_{ctm} = mean value of axial tensile strength of concrete

= $0.30*f_{ck}^{(2/3)}$ for concrete grades ≤ C50/60

= $2.12*\ln(1+((f_{ck}+8)/10))$ for concrete grades > C50/60

σ_s = the interpolated reinforcement service stress from appropriate for the bar spacing of the reinforcement provided

A_{ct} = area of concrete in tension just before formation of first crack

= b*y

where

y = the distance of the Elastic NA from bottom of beam

The minimum area of longitudinal tension reinforcement required, $A_{s,min,reqd}$, is then given by;

$A_{s,min,reqd} \geq \text{MAX}(A_{s,min}, A_{s,min,cr})$

Maximum Area of Reinforcement

The maximum area of longitudinal tension reinforcement, $A_{st,max}$, is given by;[4]

$A_{st,max} \leq k_{max}*A_c$

The maximum area of longitudinal compression reinforcement, $A_{sc,max}$, is given by;

$A_{sc,max} \leq k_{max}*A_c$

where

k_{max} = the appropriate NDP value

A_c = the cross sectional area of the beam

= $h*b_w$

For design in accordance with **UK NA,EC2 Recommendations, Irish NA**, **Malaysian NA** and **Singapore NA**;

k_{max} = 0.04

1. BS EN 1992-1-1:2004 Section 8.2(2)
2. BS EN 1992-1-1:2004 Section 9.2.1.1(1)
3. BS EN 1992-1-1:2004 Section 7.3.2(2)
4. BS EN 1992-1-1:2004 Section 9.2.1.1(3)

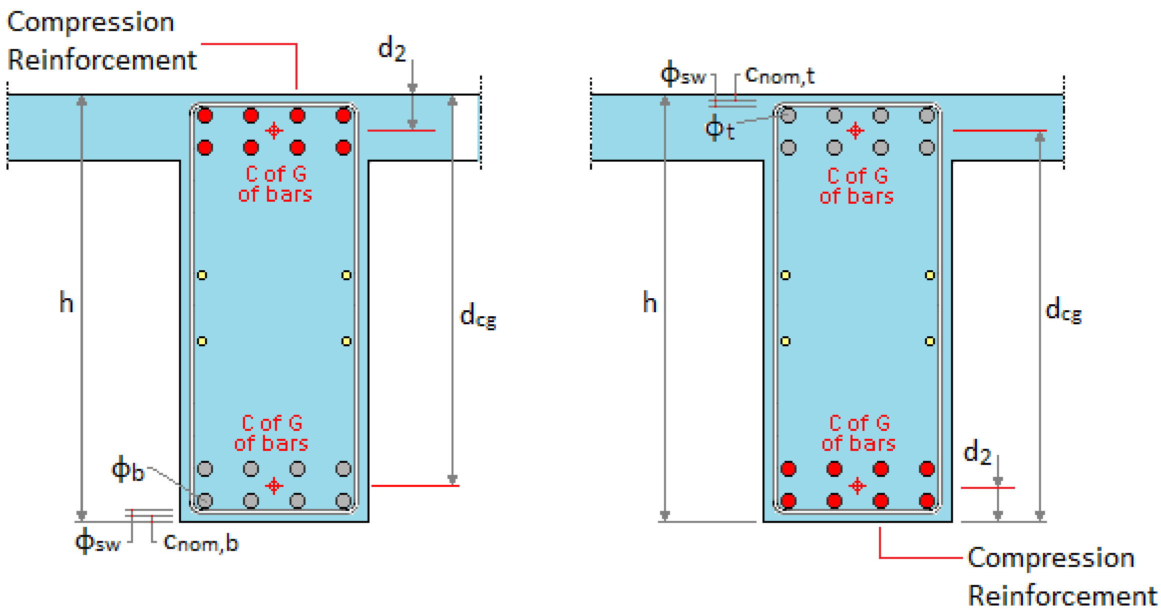
Side Reinforcement in Beams (EC2)

To control cracking in beams with a total depth ≥ 1000 mm, side bars are provided in the side faces of the beam as per BS EN 1992-1-1:2004 Section 7.3.3(3).

Effective Depth of Section (EC2)

For the design of the longitudinal tension reinforcement, the effective depth of a section, d is defined as the distance from the extreme concrete fibre in compression to the centre of gravity of the longitudinal tension reinforcement.

For the design of the longitudinal compression reinforcement, the effective depth in compression, d_2 is defined as the distance from the extreme fibre in compression to the centre of gravity of the longitudinal compression reinforcement.



Tension Reinforcement in Bottom of Beam   Tension Reinforcement in Top of Beam

Design for Bending (Beams: EC2)

Although BS EN 1992-1-1:2004 permits the limited redistribution of the elastic global analysis bending moments at ultimate limit state (ULS) in beams this is beyond scope in the current release of Tekla Structural Designer.

Design for Bending for Rectangular Sections (Beams: EC2)

Calculate the value of *K* from;

$K = M_{Ed}/(f_{ck}*b_w*d^2)$

Then calculate the limiting value of K, known as *K'* from;

$K' = (2*\eta*\alpha_{cc}/\gamma_C)*(1 - \lambda*(\delta - k_1)/(2*k_2))*(\lambda*(\delta - k_1)/(2*k_2))$ for $f_{ck} \leq 50$ N/mm^2

$K' = (2*\eta*\alpha_{cc}/\gamma_C)*(1 - \lambda*(\delta - k_3)/(2*k_4))*(\lambda*(\delta - k_3)/(2*k_4))$ for $f_{ck} > 50$ N/mm^2

where

k_i = moment redistribution factors

δ = moment redistribution ratio (= 1.0 in the current release)

γ_C = the NDP partial safety factor for concrete

α_{cc} = coefficient to take account of long term effects on compressive strength of concrete

λ = 0.8 for $f_{ck} \leq 50$ N/mm^2

= $0.8-(f_{ck}-50)/400$ for $50 < f_{ck} \leq 90$ N/mm^2

η = 1.0 for $f_{ck} \leq 50$ N/mm

= $1.0-(f_{ck}-50)/200$ for $50 < f_{ck} \leq 90$ N/mm^2

For design in accordance with **UK NA**, **Irish NA**, **Malaysian NA** and **Singapore NA**;

$\gamma_C = 1.5$

$\alpha_{cc} = 0.85$

For design in accordance with **EC2 Recommendations**;

$\gamma_C = 1.5$

$\alpha_{cc} = 1.0$

-

IF K ≤ K' THEN compression reinforcement is not required.

Calculate the lever arm, *z* from;

$z = MIN(0.5*d*[1 + (1 - 2*K/(\eta*\alpha_{cc}/\gamma_C))^{0.5}], 0.95*d)$

The area of tension reinforcement required is then given by;

$A_{st,reqd} = M_{Ed}/(f_{yd}*z)$

where

$f_{yd} = f_{yk}/\gamma_S$

γ_S = the NDP partial safety factor for reinforcement

The depth to the neutral axis, x_u is given by;

$x_u = 2*(d-z)/\lambda$

For design in accordance with **UK NA, EC2 Recommendations**, **Irish NA**, **Malaysian NA** and **Singapore NA**;

$\gamma_S = 1.15$

IF K > K' THEN compression reinforcement is required.

Calculate the depth to the neutral axis from;

$x_u = d*(\delta-k_1)/k_2$ for $f_{ck} \leq 50$ N/mm^2

$x_u = d*(\delta-k_3)/k_4$ for $f_{ck} > 50$ N/mm^2

Calculate the stress in the reinforcement from;

$f_{sc} = MIN(E_s*\varepsilon_{cu3}*(1-(d_2/x_u), f_{yd})$

where

d_2= the distance from the extreme fibre in compression to the c of g of the compression reinforcement

Calculate the limiting bending strength, *M'* from;

$M' = K'*f_{ck}*b_w*d^2$

Calculate the lever arm from;

$z = 0.5*d*[1 + (1 - 2*K'/(\eta*\alpha_{cc}/\gamma_C))^{0.5}]$

The area of compression reinforcement required, $A_{s2,reqd}$ is given by;

$A_{s2,reqd} = (M_{Ed}-M')/(f_{sc}*(d-d_2))$

The area of tension reinforcement required, $A_{st,reqd}$ is given by;

$$A_{st,reqd} = M'/(f_{yd}*z) + A_{s2,reqd}*f_{sc}/f_{yd}$$

Design for Bending for Flanged Sections (Beams: EC2)

IF $h_f < 0.1*d$ THEN treat the beam as rectangular.

$$h_f = MIN(h_{f,side1}, h_{f,side2})$$

where

$h_{f,sidei}$ = the depth of the slab on side "*i*" of the beam

Calculate the value of *K* from;

$$K = M_{Ed}/(f_{ck}*b_{eff}*d^2)$$

Calculate the lever arm, *z* from;

$$z = MIN(0.5*d*[1 + (1 - 2*K/(\eta*\alpha_{cc}/\gamma_C))^{0.5}], 0.95*d)$$

Calculate the depth of the rectangular stress block, *λ*x* from;

$$\lambda*x = 2*(d-z)$$

IF $\lambda*x \le h_f$ THEN the rectangular compression block is wholly in the depth of the flange and the section can be designed as a rectangular section by setting $b_w = b_{eff}$.

IF $\lambda*x > h_f$ THEN the rectangular compression block extends into the rib of the flanged section and the following design method is to be used.

The design bending strength of the flange, M_f is given by;

$$M_f = f_{cd}*h_f*(b_{eff}-b_w)*(d-0.5*h_f)$$

The area of reinforcement required to provide this bending strength, $A_{sf,reqd}$ is given by;

$$A_{sf,reqd} = M_f/(f_{yd}*(d-0.5*h_f))$$

The remaining design moment, $(M_{Ed}-M_f)$ is then taken by the rectangular beam section.

Calculate the value of *K* from;

$$K = (M_{Ed}-M_f)/(f_{ck}*b_w*d^2)$$

Then calculate the limiting value of K, known as *K'* from;

$$K' = (2*\eta*\alpha_{cc}/\gamma_C)*(1 - \lambda*(\delta - k_1)/(2*k_2))*(\lambda*(\delta - k_1)/(2*k_2)) \text{ for } f_{ck} \le 50 \text{ N/mm}^2$$

$K' = (2*\eta*\alpha_{cc}/\gamma_C)*(1 - \lambda*(\delta - k_3)/(2*k_4))*(\lambda*(\delta - k_3)/(2*k_4))$ for $f_{ck} > 50$ N/mm^2

IF K ≤ K' THEN compression reinforcement is not required.

Calculate the lever arm, *z* from;

$z = MIN(0.5*d*[1 + (1 - 2*K/(\eta*\alpha_{cc}/\gamma_C))^{0.5}], 0.95*d)$

The area of tension reinforcement required is then given by;

$A_{sr,reqd} = (M_{Ed}-M_f)/(f_{yd}*z)$

The total area of tension reinforcement required, $A_{st,reqd}$ is then given by;

$A_{st,reqd} = A_{sf,reqd}+A_{sr,reqd}$

The depth to the neutral axis, x_u is given by;

$x_u = 2*(d-z)/\lambda$

IF K > K' THEN compression reinforcement is required.

Calculate the depth to the neutral axis from;

$x_u = d*(\delta-k_1)/k_2$ for $f_{ck} \leq 50$ N/mm^2

$x_u = d*(\delta-k_3)/k_4$ for $f_{ck} > 50$ N/mm^2

Calculate the stress in the reinforcement from;

$f_{sc} = MAX(E_s*\varepsilon_{cu3}*(1-(d_2/x_u), f_{yd})$

where

d_2 = the distance from the extreme fibre in compression to the c of g of the compression reinforcement

Calculate the limiting bending strength, *M'* from;

$M' = K'*f_{ck}*b_w*d^2$

Calculate the lever arm from;

$z = 0.5*d*[1 + (1 - 2*K'/(\eta*\alpha_{cc}/\gamma_C))^{0.5}]$

The area of compression reinforcement required, $A_{s2,reqd}$ is given by;

$A_{s2,reqd} = (M_{Ed}-M_f-M')/(f_{sc}*(d-d_2))$

The area of tension reinforcement required, $A_{sr,reqd}$ is given by;

$A_{sr,reqd} = M'/(f_{yd}*z) + A_{s2,reqd}*f_{sc}/f_{yd}$

The total area of tension reinforcement required, $A_{st,reqd}$ is then given by;

$A_{st,reqd} = A_{sf,reqd}+A_{sr,reqd}$

Design for Shear (EC2)

Design Shear Resistance (Beams: EC2)

The design value of the shear resistance of a concrete section with vertical shear reinforcement, $V_{Rd,max}$ is given by;

$V_{Rd,max} = 0.9*\alpha_{cw}*b_w*d*\nu_1*f_{cwd}/(\cot\theta + \tan\theta)$

where

$\theta = MIN \{\theta_{max}, MAX[0.5*\sin^{-1}[2*V_{Ed,max}/(\alpha_{cw}*b_w*0.9*d*\nu_1*f_{cwd})], \theta_{min}]\}$

f_{cwd} [1] $= \alpha_{ccw}*f_{ck}/\gamma_C$

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA;**

$\alpha_{cw} = 1.0$ (assuming no axial load in the beam)

$\alpha_{ccw} = 1.0$

$\gamma_C = 1.5$

$\nu_1 = 0.6*(1 – (f_{ck}/250))$ f_{ck} in N/mm^2

The limits of θ are given by 1 ≤ cotθ ≤ 2.5 which gives;

$\theta_{max} = \tan^{-1}1$

$\theta_{min} = \tan^{-1}(0.4)$

IF $V_{Ed,max} > V_{Rd,max}$

where

$V_{Ed,max}$ = the maximum design shear force acting anywhere on the beam

THEN the shear design process FAILS since the section size is inadequate for shear (the compression strut has failed at the maximum allowable angle).

The design shear capacity of the minimum area of shear links actually provided, V_{nom} is given by[2];

$V_{nom} = (A_{sw,min,prov} / s_l) * 0.9 * d * f_{ywd} * \cot\theta$

where

$A_{sw,min, prov}$ is the area of shear reinforcement provided to meet the minimum requirements.

$f_{ywd} = f_{ywk}/\gamma_S$

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA** the limiting values of θ are given by;

$1 \leq \cot\theta \leq 2.5$

and: $\gamma_S = 1.15$

The maximum possible value for the shear resistance provided by this area of shear reinforcement will be when the angle of the compression strut is the minimum value i.e. cotθ = 2.5 and therefore V_{nom} can be simplified to;

$V_{nom} = (A_{sw,min,prov}/s_l) * 2.25 * d * f_{ywd}$

In any region, *i*;

IF

$V_{Ed,i} > V_{nom}$

where

$V_{Ed,i}$ = the maximum shear in region *i*

THEN shear links are required in the region.

For designed shear links in shear region *Si*, first calculate the angle of the compression strut from;

$\theta_{Si} = \text{MIN}\{\theta_{max}, \text{MAX}[0.5*\sin^{-1}[2*V_{Ed,Si} /(\alpha_{cw}*b_w*0.9*d*\nu_1*f_{cd})], \theta_{min}]\}$

The area of links required in shear region *Si* is then given by;

$(A_{sw,reqd}/s)_{Si} = V_{Ed,Si}/(0.9*d*f_{ywd}*\cot\theta_{Si})$

where

$V_{Ed,Si}$ = the maximum shear force in shear region *Si*

1. Eqn (3.15) BS EN 1992-1-1:2004 Section 3.1.6(1)P

2. BS EN 1992-1-1:2004 Section 6.2.3(3) Eqn (6.8)

Minimum Area of Shear Reinforcement (Beams: EC2)

The minimum area of shear reinforcement required, $A_{sw,min}$ is given by[1];

$A_{sw,min}$ = MAX[s_l*$\rho_{w,min}$*b_w, $A_{sw,min,u}$]

where

s_l = the spacing of the shear reinforcement along the longitudinal axis of the beam

$A_{sw,min,u}$ = the total minimum area of the shear reinforcement calculated from data supplied i.e. maximum spacing across the beam, minimum link diameter and number of legs

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA**;

$\rho_{w,min}$ = $(0.08*\sqrt{(f_{ck})})/f_{yk}$

1. BS EN 1992-1-1:2004 Section 9.2.2(5) Eqn (9.4)

Spacing of Shear Reinforcement (Beams: EC2)

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA** the longitudinal spacing, s_l between the legs of shear reinforcement is given by;

$s_{l,min,u} \leq s_l \leq$ MIN[0.75*d, $s_{l,max,u}$]

where

$s_{l,max,u}$ = the maximum longitudinal spacing specified

$s_{l,min,u}$ = the minimum longitudinal spacing specified

If compression reinforcement is required for bending, for design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA** the longitudinal spacing, s_l between the legs of shear reinforcement is given by;

$s_{l,min,u} \leq s_l \leq$ MIN{MIN[0.75*d, 15*Φ_{comp}] , $s_{l,max,u}$}

where

Φ_{comp} = the minimum diameter of the compression reinforcement[1]

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA** the transverse spacing, s_t between the legs of shear reinforcement is given by;

$s_t \leq$ MIN[0.75*d, 600, $s_{t,max,u}$]

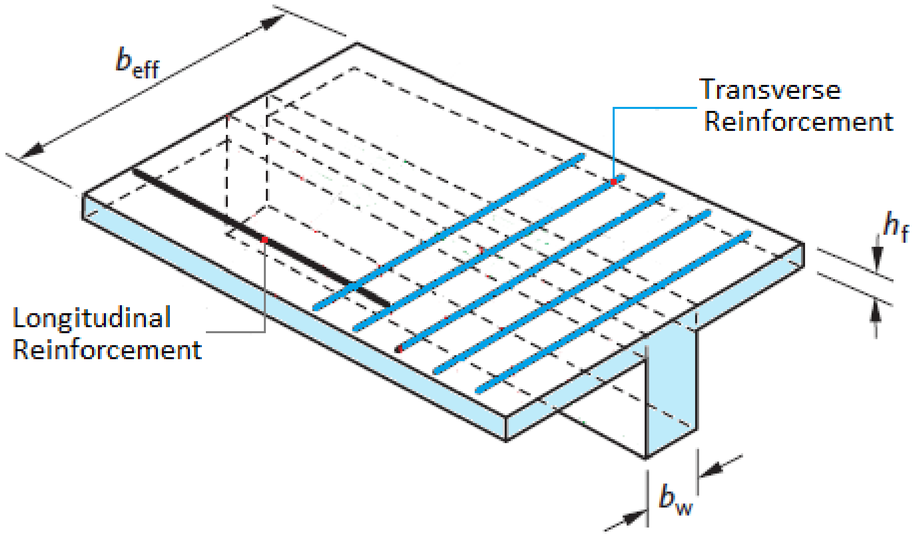
where

$s_{t,max,u}$ = the maximum link leg spacing across the beam

1. Looking for the smallest value therefore use minimum diameter

Shear between Flanges and Web of Flanged Beams (EC2)

The shear strength of the interface between the flanges and the web of a flanged beam is checked and, if necessary, transverse reinforcement is provided as shown in the diagram below.[1]



1. BS EN 1992-1-1:2001 Section 6.2.4

Additional Tension Reinforcement (Beams: EC2)

In BS EN 1992-1-1:2004, the method of designing for vertical shear is based on a truss analogy with a diagonal strut acting at an angle θ. This strut action means that there must be a tension force developed in the longitudinal reinforcement which is additional to that arising from bending action.

To resist this tension force, an area of reinforcement additional to that required to resist bending is required.

The total area of longitudinal tension reinforcement in each of the regions then becomes;

$A_{stt,reqd,i} = A_{st,reqd,i} + A_{swa,reqd,i}$

where

$A_{st,reqd,i}$ = the area of longitudinal reinforcement required to resist bending as appropriate in region "*i*"

$A_{swa,reqd,i}$ = the area of longitudinal reinforcement required to resist the additional tension force from vertical shear in region "*i*"

Design for Torsion (Beams: EC2)

Design values of the shear resistance and torsional resistance moment (Beams: EC2)

The design value of the shear resistance of a concrete section with vertical shear reinforcement, $V_{Rd,max}$ is given by;

$V_{Rd,max} = 0.9*\alpha_{cw}*b_w*d*\nu_1*f_{cwd}/(\cot\theta + \tan\theta)$

where

$\theta = MIN\{\theta_{max}, MAX[0.5*\sin^{-1}[2*V_{Ed,max}/(\alpha_{cw}*b_w*0.9*d*\nu_1*f_{cwd})], \theta_{min}]\}$

f_{cwd}[1] $= \alpha_{ccw}*f_{ck}/\gamma_C$

The maximum design value of the torsional resistance moment, $T_{Rd,max}$ is given by;

$T_{Rd,max} = 2*\nu_1*\alpha_{ccw}*f_{cwd}*A_k*t_{ef}*\sin\theta*\cos\theta$

where

$A_k = (h-t_{ef})*(b_w-t_{ef})$

and

$t_{ef} = MAX(A/u, 2*(h-d_o))$[2]

where

$A = h*b_w$

u = the outer circumference of the cross-section

$= 2*(h+b_w)$

d_o = the effective depth of the outer layer of longitudinal reinforcement

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA;**

$\alpha_{cw} = 1.0$

$\alpha_{ccw} = 1.0$

$\gamma_C = 1.5$

$\nu_1 = 0.6*(1 - f_{ck}/250)$ f_{ck} in N/mm^2

The limits of θ are given by $1 \leq \cot\theta \leq 2.5$ which gives;

$\theta_{max} = \tan^{-1}1$

$\theta_{min} = \tan^{-1}(0.4)$

The design value of the torsional resistance moment of a concrete section with no shear reinforcement, $T_{Rd,c}$ is given by[3];

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

where

f_{ctd} = the design tensile strength of the concrete

$= \alpha_{ct}*f_{ctk,0.05}/\gamma_C$

If the maximum torsional moment acting on the beam, $T_{Ed,max}$ is less than the ignorable torque limit then no further calculations are necessary.

Otherwise:

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

then the torsion design process can proceed.

ELSE the torsion design FAILS since the section size is inadequate for torsion.

1. Eqn (3.15) BS EN 1992-1-1:2004 Section 3.1.6(1)P

2. BS EN 1992-1-1:2004 Section 6.3.2(1).

3. BS EN 1992-1-1:2004 Eqn (6.26) with $\tau_{t,i} = f_{ctd}$

Additional reinforcement for torsion (Beams: EC2)

The design value of the shear resistance of a concrete section with no shear reinforcement, $V_{Rd,c}$ is given by;[1]

$V_{Rd,c} = \nu_{min} * b_w * d$

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA;**

$C_{Rd,c} = 0.18/\gamma_C$

$\gamma_C = 1.5$

$\nu_{min} = 0.035 * k^{1.5} * f_{ck}^{0.5}$

where

$k = MIN(1 + \sqrt{(200/d)}, 2.0)$ *d* in mm

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA;**

$\alpha_{ct} = 1.0$

$\gamma_C = 1.5$

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

THEN no additional longitudinal reinforcement for torsion is required.

IF $(T_{Ed,max}/T_{Rd,c}) + (V_{Ed,max}/V_{Rd,c}) > 1.0$

THEN additional longitudinal reinforcement for torsion, $A_{slT,reqd}$ is required in some or all regions.

The additional longitudinal reinforcement is given by;

$A_{slT,reqd} = (T_{Ed} * u_k * \cot\theta)/(2 * A_k * f_{yd})$

where

$u_k = 2*((h-t_{ef})+(b_w-t_{ef}))$

This reinforcement is **in addition** to that required for bending and tension arising from vertical shear and it is distributed in each of the four faces of the beam in proportion to the length of the face of the cross-section.

The area of the additional link reinforcement that is required to resist torsion is given by;

$A_{swt}/s = (T_{Ed})/(2 * A_k * 0.9 * f_{ywd} * \cot\theta)$ per leg

1. The design value of the shear resistance is calculated ignoring the longitudinal reinforcement as it is not known if this reinforcement is adequately anchored beyond the point under consideration. This is a conservative approach.

Deflection Check (Beams: EC2)

The deflection of reinforced concrete beams is not directly calculated and the serviceability of the beam is measured by comparing the calculated limiting span/effective depth ratio *L/d* to the maximum allowable values as given by;[1]

IF $\rho \leq \rho_0$

$$(L/d)_{max} = K_{ss}*f_1*f_2*(11+1.5*(f_{ck})^{1/2}*(\rho_0/\rho)+3.2*(f_{ck})^{1/2}*((\rho_0/\rho)-1)^{3/2})*(500*A_{st,prov}/(f_{yk}*A_{st,reqd}))$$

IF $\rho > \rho_0$

$$(L/d)_{max} = K_{ss}*f_1*f_2*(11+1.5*(f_{ck})^{1/2}*(\rho_0/(\rho-\rho'))+(1/12)*(f_{ck})^{1/2}*(\rho'/\rho_0)^{1/2})*(500*A_{st,prov}/(f_{yk}*A_{st,reqd}))$$

where

ρ = the designed tension reinforcement ratio at mid-span (or at support for cantilevers) required to resist bending

= $A_{st.reqd}/(b_w*d)$ for rectangular beams

= $A_{st,reqd}/(b_{eff}*d)$ for flanged beams

ρ' = the designed compression reinforcement ratio at mid-span (or at support for cantilevers) required to resist bending

= $A_{s2,reqd}/(b_w*d)$ for rectangular beams

= $A_{s2,reqd}/(b_{eff}*d)$ for flanged beams

$A_{st,reqd}$ = the designed area of tension reinforcement at mid-span (or at support for cantilevers) required to resist bending

$A_{st,prov}$ = MIN(the area of tension reinforcement provided at mid-span (or at support for cantilevers), $f_3*A_{st,reqd}$)

$A_{s2,reqd}$ = the designed area of compression reinforcement at mid-span (or at support for cantilevers) required to resist bending

f_1 = 1.0 for rectangular beams

= 1.0 for flanged beams with $b_{eff}/b_w \leq 3.0$

= 0.8 for flanged beams with $b_{eff}/b_w > 3.0$

f_2 = 1.0 IF $L_{eff} \leq 7$ m

= $7/L_{eff}$ IF $L_{eff} > 7$ m with L_{eff} in metre units

L_{eff} = the length of the beam between the centre of its supports[A]

f_3 = an NDP factor as given below

K_{ss} = the structural system factor which is an NDP and is given below

[A] This definition of effective length will return conservative results when the width of the support is greater than the depth of the beam – see BS EN 1992-1-1:2004 Section 5.3.2.2(1)

For design in accordance with **EC2 Recommendations** the NDP value of f_3 is given by [2];

$f_3 = 1.5$

For design in accordance with **UK NA, Irish NA, Malaysian NA and Singapore NA** the NDP value of f_3 is given by [3];

$f_3 = 1.5$

For design in accordance with **UK NA, EC2 Recommendations, Irish NA, Malaysian NA and Singapore NA** the NDP value of K_{ss} is given by the following table:

Span Detail	LH End Fixity	RH End Fixity	K_{ss}
LH End Span	Fixed	Fixed	1.3
	Fixed	Pinned	1.0
	Pinned	Fixed	1.3
	Pinned	Pinned	1.0
Internal Span	Fixed	Fixed	1.5
	Fixed	Pinned	1.3
	Pinned	Fixed	1.3
	Pinned	Pinned	1.0
RH End Span	Fixed	Fixed	1.3
	Fixed	Pinned	1.3
	Pinned	Fixed	1.0
	Pinned	Pinned	1.0
Cantilever			0.4

1. BS EN 1992-1-1:2004 Section 7.4.2

2. BS EN 1992-1-1:2004 is silent on the recommended value to use therefore adopt 1.5 since if f_3 is greater than 1.5 no benefit arises.

3. For Irish NA refer to Table NA.3 and for other others refer to Table NA.5

Column Design to EC2

Limitations and Exclusions (Columns: EC2)

The longitudinal and transverse reinforcement requirements of clause 9.5 are applied to all columns, including columns where the larger dimension is greater than 4 times the smaller dimension - this is conservative.

The following general exclusions also apply:

- Seismic design,
- Consideration of fire resistance. [You are however given full control of the minimum cover dimension to the reinforcement and are therefore able to take due account of fire resistance requirements.],
- Lightweight concrete,
- Chamfers,
- Multi-stack reinforcement lifts.

Materials (Columns: EC2)

Concrete

Only normal weight is included in the current release. (Lightweight concrete is excluded).

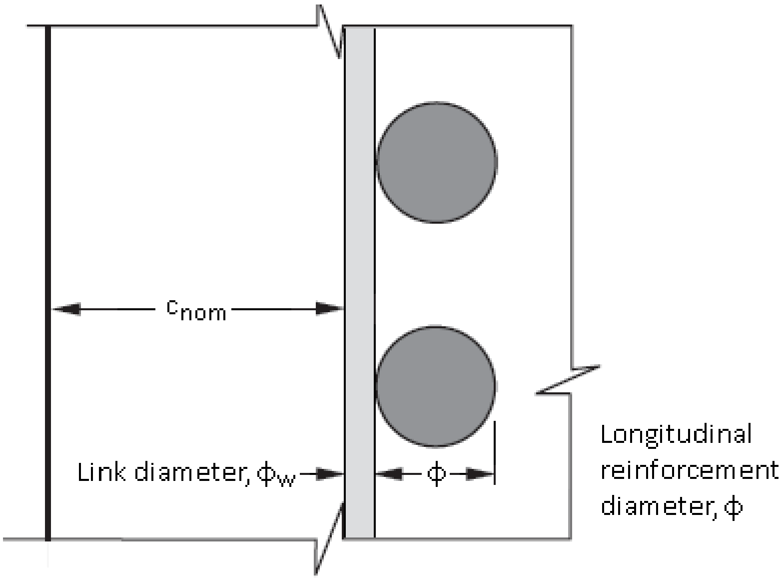
Reinforcement

The reinforcement options are:

- Loose reinforcing bars,
- Loose reinforcing bars bent to form links.

Cover to Reinforcement (Columns: EC2)

The nominal concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and surface reinforcement where relevant) and the nearest concrete surface.



You are required to set a minimum value for the nominal cover, $c_{nom,u}$, for each column in the column properties.

These values are then checked against the nominal limiting cover, $c_{nom,lim}$ which depends on the diameter of the reinforcement plus an allowance for deviation, Δc_{dev} (specified in **Design Options > Column > General Parameters**).

Generally, the allowance for deviation, Δc_{dev} is a NDP.[1] The recommended value is 10mm, but under strict controls it can be reduced to 5mm.

If $c_{nom,u} < c_{nom,lim}$ then a warning is displayed in the calculations.

1. BS EN 1992-1-1:2004 cl 4.4.1.3 (1)P

Design Parameters for Longitudinal Bars (Columns: EC2)

For some of the longitudinal reinforcement design parameters, additional user defined limits can be applied - where this is the case minimum and maximum values are specified in **Design Options > Column > Reinforcement Layout**.

Minimum Longitudinal Bar Diameter

For design in accordance with **EC2 Recommendations**;

$\varphi_{long,min}$ = 8mm

For design in accordance with **Malaysian NA**;

$\varphi_{long,min}$ = 10m

For design in accordance with **UK NA, Irish NA** and **Singapore NA**;

$\varphi_{long,min}$ = 12mm

Minimum Longitudinal Bar Spacing[1]

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$s_{cl,min}$ ≥ MAX[maximum longitudinal bar diameter, 20mm, d_g + 5mm]

Where d_g is the maximum aggregate size.

Maximum Longitudinal Bar Spacing

You are given control over this value by specifying an upper limit in **Design Options > Column > Reinforcement Layout**.

Minimum Longitudinal Total Steel Area2

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

If N_{Ed} >= 0 (i.e. compression)

$A_{sl,min}$ = MAX[(0.1 * N_{Ed}) / f_{yd}, 0.2% * column area]

Else

$A_{sl,min}$ = 0.45% * column area

It has been decided that in the tension case, in the absence of clear guidance by EC2, it is responsible and conservative to adopt the 0.45% used by BS8110.

Maximum Longitudinal Total Steel Area3

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$A_{sl,max}$ = 4% * column area (8% in lap regions)

Long Term Compressive Strength Factor4

For design in accordance with **UK NA, Irish NA, Malaysian NA** and **Singapore NA**;

α_{cc} = 0.85

For design in accordance with **EC2 Recommendations**;

α_{cc} = 1.0

Design Concrete Compressive Strength for Shear5

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$f_{cd} = \alpha_{cc} * f_{ck} / \gamma_C$

1. BS EN 1992-1-1:2004 Section 8.2
2. BS EN 1992-1-1:2004 Section 9.5.2(2)
3. BS EN 1992-1-1:2004 Section 9.5.2(3)
4. BS EN 1992-1-1:2004 Section 3.1.6(1)
5. BS EN 1992-1-1:2004 Section 3.1.6(1)

Ultimate Axial Load Limit (Columns: EC2)

This limit is when the section is under pure compression (i.e. no moment is applied). It is observed that for non-symmetric arrangements, applying a small moment in one direction may increase the maximum axial load that can be applied to a section because the peak of the N-M interaction diagram is shifted away from the N-axis (i.e. the zero moment line). Checking that the axial load does not exceed the ultimate axial load limit of the section ensures that there is always a positive moment limit and a negative moment limit for the applied axial load for the section.

The ultimate axial load limit of the section, assuming a rectangular stress distribution, is calculated from:

$$N_{max} = (RF * A_c * f_{cd} * \eta) + \sum(A_{s,i} * f_{s,i})$$

Given that,

$$A_c = A - \sum A_{s,i}$$

$$f_{s,i} = \varepsilon_c * E_{s,i}$$

Where

RF is the concrete design reduction factor, (this is a fixed value of 0.9 which cannot be changed)

A is the overall area of the section,

A_c is the area of concrete in the section,

$A_{s.i}$ is the area of bar *i*,

f_{cd} is the design compressive strength of the concrete,

η is a reduction factor for the design compressive strength for high strength concrete for the rectangular stress distribution,

ε_c is the strain in the concrete at reaching the maximum strength,

$f_{s,i}$ is the stress in bar *i* when the concrete reaches the maximum strength,

$E_{s,i}$ is the modulus of elasticity of the steel used in bar *i*.

The concrete design reduction factor RF originates from EC2 section 3.1.7(3): "Note: If the width of the compression zone decreases in the direction of the extreme compression fibre, the value ηf_{cd} should be reduced by 10%"

In Tekla Structural Designer the RF factor is applied in both the axial-moment interaction check and the ultimate axial resistance check (even though there is no extreme compression fibre in this latter calculation) so that the ultimate axial resistance matches the peak position of the interaction diagram - its inclusion creates a conservative result.

Effective Length Calculations (Columns: EC2)

Clear Height

The clear height is the clear dimension between the restraining beams at the bottom of the stack and the restraining beams at the top of the stack. The clear height may be different in each direction.

If, at an end of the stack, no effective beams or flat slab to include are found, then the clear height includes the stack beyond this restraint, and the same rules apply for finding the end of the clear height at the end of the next stack (and so on).

Effective Length

The effective length, l_0 is calculated automatically - you also have the ability to override the calculated value.

From EC2, cl. 5.8.3.2, the equations for calculating the effective length are as follows.

For stacks designated as "braced", the effective length is given by[1]:

$$l_0 = 0.5 * l * \sqrt{(1 + (k_1 / (0.45 + k_1)))} * \sqrt{(1 + (k_2 / (0.45 + k_2)))}$$

In addition *Tekla Structural Designer* imposes the following limits for stacks that are designated as braced:

$$0.5 \le l_0 / l \le 1$$

For stacks designated as "bracing", the effective length is the larger of[2]:

$$l_0 = l * \sqrt{(1 + (10 * k_1 * k_2 / (k_1 + k_2)))}$$

Or

$$l_0 = l * (1 + (k_1 / (1 + k_1))) * (1 + (k_2 / (1 + k_2)))$$

Where

k_1 and k_2 are the relative flexibilities of rotational restraints at ends 1 and 2 respectively, in the direction under consideration. Which way the ends are numbered is irrelevant to the result. The program uses the bottom end of the stack as end 1 and the top end as end 2.

The value of k, which may refer to either k_1 or k_2 depending on which end of the stack is being examined, is defined by[3]:

$$k = (\theta / M) * (E * I / l)$$

Where

M is the moment applied to the restraining members by the buckling member or members,

θ is the rotation of the joint at the end of the stack considered for the bending moment M,

$(E * I / l)$ is the bending stiffness of the compression member or members considered to be buckling.

It is recommended to take "θ / M" for the beams as "$l / (2 * E * I)$".

The standard approximation[4] for "θ / M" is between "l / (4 * E * I)" and "l / (3 * E * I)", so to allow for cracking the value is increased. Also, "E * I / l" is the sum of the stiffness of column stacks joining at the connection.

The above equation then becomes:

$$k = \sum(E * I / l)_{cols} / \sum(2 * E * I / l)_{beams}$$

If there are any adjacent stacks beyond the joint at the end of the restrained length under consideration, then it must be considered whether these adjacent stacks are likely to contribute to the deflection or restrain it. If the stiffness are similar then the stiffness of the adjacent stacks can be ignored, and the guidance in PD6687 suggests that this range of similarity of stiffness can be taken as 15% above or below the stiffness of the stack being designed. Therefore:

If

$$0.85 \leq \sum((E * I / l)_{\text{stacks beyond this joint}}) / (E * I / l)_{\text{stack under consideration}} \leq 1.15$$

Then

$$\sum(E * I / l)_{cols} = (E * I / l)_{\text{stack under consideration}}$$

Else

$$\sum(E * I / l)_{cols} = (E * I / l)_{\text{stack under consideration}} + \sum(E * I / l)_{\text{stacks beyond this joint}}$$

These stacks can be part of the same column length or another column length.

Note that as the restrained length may be multiple stacks, "E * I" for this stack are the values for the stack being designed, and l is the restrained length. For the stacks beyond the restraint, "E * I" are the values for the stack attached to the restraint, and l is the restrained length that the stack exists within.

Any beams framing into the end of the stack within 45 degrees of the axis being considered are said to be restraining beams for the stack in that direction.

There is a lower limit[5] for the value of k:

$$k \geq 0.1$$

Additionally, *Tekla Structural Designer* imposes an upper limit:

$$k \leq 20$$

For bracing stacks, a warning is displayed when the calculated value of k exceeds this limit.

Fixed Column Base

k = 0.1 for fixed bases in *Tekla Structural Designer*. There is no clear guidance in EC2, but the Concrete Centre guidance suggests that this is suitable.

If you have set the bottom of the column to be "fixed" but the support as "pinned". The program will always assume that the support is fixed and therefore only ever consider the fixity applied to the column.

Pinned Column End

In any situation where the end of a column anywhere in the structure is pinned, k = 20.

No Effective Beams Found

If no effective beams are found to restrain the end of the stack in the direction in question, then the program will consider whether there is a flat slab restraining the stack at this end. If a flat slab is found it will either be considered as a restraint, or not, in each direction at each end of the stack - this is controlled by checking the option **Use slab for stiffness calculation...** located as a Stiffness setting in the column properties. If there are no effective beams and there is no flat slab (or any flat slab is not to be considered), then the program looks for the far end of the stack on the other side of the joint, and look at the restraints there, and so on until a restraint with an effective beam or flat slab to be considered is found.

If the stack is restrained by a flat slab, then the slab will be considered to act as a beam in this direction – note that it is one beam in the direction and NOT a beam on each side of the column.

If the stack is an end stack and there are no supports, beams or flat slabs considered to restrain the stack at this end in the direction, the end is therefore free in this direction and k = 20.

1. BS EN 1992-1-1:2004 Section 5.8.3.2(3)

2. BS EN 1992-1-1:2004 Section 5.8.3.2(3)

3. BS EN 1992-1-1:2004 Section 5.8.3.2(3)

4. PD 6687-1:2010 Section 2.11.2

5. BS EN 1992-1-1:2004 Section 5.8.3.2(3)

Column Stack Classification (Columns: EC2)

Slenderness ratio

The slenderness ratio, λ, of the restrained length about each axis is calculated as follows:

$$\lambda = l_0 / i = l_0 / \sqrt{(I / A)}$$

Where

l_0 is the effective height of the stack,

i is the radius of gyration of the stack section about the axis under consideration,

I is the second moment of area of the stack section about the axis,

A is the cross-sectional area of the stack section.

The slenderness ratio λ is then checked against the limiting slenderness ratio λ_{lim} in each direction. If the slenderness is less than this limit, then the member is short and slenderness effects are ignored,.otherwise it is slender.

limiting slenderness ratio

$\lambda_{lim} = 20 * A * B * C / \sqrt{n}$

Where

$A = 1 / (1 + (0.2 * \varphi_{ef})) \geq 0.7$

$B = \sqrt{(1 + (2 * \omega))} \geq 1.1$

$C = 1.7 - r_m$

Where

φ_{ef} is the effective creep ratio,

$\omega = A_s * f_{yd} / (A_c * f_{cd})$,

f_{yd} is the design yield strength of the reinforcement,

f_{cd} is the design compressive strength of the concrete,

A_s is the total area of longitudinal reinforcement,

$n = N_{Ed} / (A_c * f_{cd})$,

N_{Ed} is the design axial force between restrained floor levels in this direction,

$r_m = M_{1.1} / M_{2.1}$,

$M_{1.1}$ and $M_{2.1}$ are the first order moments at the ends of the stack about the axis being considered, with $|M_{2.1}| \geq |M_{1.1}|$.

If $M_{1.1}$ and $M_{2.1}$ cause tension in the same side of the stack then r_m is positive and $C \leq 1.7$. If the converse is true then the stack is in double curvature, and it follows that r_m is negative and $C > 1.7$.

For braced stacks in which the first order moments arise only from transverse loads (lateral loading is significant) or imperfections ($M_{imp.1} > |M_{2.1}|$), C must be taken as 0.7,

For

bracing stacks, C must be taken as 0.7,

For restrained lengths encompassing more than one stack, C is taken as 0.7.

The effective creep ratio, φ_{ef}, is derived as follows:

$f_{cm} = f_{ck} + 8$ (N/mm^2)

$h_0 = 2 * A_g / u$

Where

u is the section perimeter in contact with the atmosphere (assumed to be the full section perimeter),

A_g is the gross section area.

$\alpha_1 = (35 / f_{cm})^{0.7}$

$\alpha_2 = (35 / f_{cm})^{0.2}$

$\alpha_3 = (35 / f_{cm})^{0.5}$

If $f_{cm} \leq 35$ N/mm^2,

$\beta_H = (1.5 * (1 + (1.2 * RH))^{18} * h_0) + 250 \leq 1500$

Else,

$\beta_H = (1.5 * (1 + (1.2 * RH))^{18} * h_0) + (250 * \alpha_3) \leq 1500 * \alpha_3$

Where

RH is the relative humidity, which is set under Design parameters in the column properties.

$\beta_c(t , t_0) = ((t - t_0) / (\beta_H + t - t_0))^{0.3}$

$\beta_{t0} = 1 / (0.1 + t_0^{0.2})$

$\beta_{fcm} = 16.8 / \sqrt{f_{cm}}$

Where

t_0 is the age of column loading and defaults to 14 days, if required it can be changed under Design parameters in the column properties.

If $f_{cm} \leq 35$ N/mm^2,

$\varphi_{RH} = 1 + ((1 - (RH / 100)) / (0.1 * h_0^{1/3}))$

Else,

$\varphi_{RH} = (1 + (((1 - (RH / 100)) / (0.1 * h_0^{1/3})) * \alpha_1)) * \alpha_2$

Then,

$\varphi_0 = \varphi_{RH} * \beta_{fcm} * \beta_{t0}$

$\varphi(\infty , t_0) = \varphi_0 * \beta_c(\infty , t_0)$

If $\varphi(\infty , t_0) \leq 2$ and $\lambda < 75$ and $M_{max.1} / N_{Ed} \geq h$ and $\omega \geq 0.25$,

$\varphi_{ef} = 0$

Else

$\varphi_{ef} = \varphi(\infty, t_0) * R_{PL}$

Where

$M_{max.1}$ is the largest first order moment in the restrained length in this direction,

N_{Ed} is the design axial force in the restrained length in this direction,

R_{PL} is the permanent load ratio.

You are required to supply a value for the permanent load ratio which is located under Design parameters in the column properties. A default of 0.65 has been assumed, but you are advised to consider if this is appropriate and adjust as necessary.

Tekla Structural Designer assumes that t_∞ (t-infinity) is equal to 70 years (25550 days).

Overview of Second Order Effects (Columns: EC2)

For 'isolated' columns and walls, EN1992-1-1 (EC2) allows for second order effects and member imperfections in a number of ways,

- It specifies a minimum level of member imperfection along with a conservative value - see Clause 5.2 (7).

- It provides for the additional moment due to slenderness (member buckling) using one of two methods. One method (the (Nominal) Stiffness Method) increases the first-order moments in the column using an amplifier based on the elastic critical buckling load of the member - see Clause 5.8.7.3. The second method (the (Nominal) Curvature Method) calculates the 'second-order' moment directly based on an adjustment to the maximum predicted curvature that the column section can achieve at failure in bending - see Clause 5.8.8.
- The impact of the slenderness is increased or decreased depending upon the effective length factor for the member. For braced members this will be ≤ 1.0 and for unbraced (bracing) members it will be ≥ 1.0 see Clause 5.8.3.2.

Finally, EC2 also requires consideration of a minimum moment based on the likelihood that the axial load cannot be fully concentric see Clause 6.1 (4).

Minimum moment (Clause 6.1 (4))

The minimum moment about each axis, M_{min} is calculated. When using the Curvature Method, M_2 is added to the minimum moment. When using the Stiffness Method M_2 is calculated from $M_{min} \times \pi^2/(8(\alpha_{cr} - 1))$ and added to M_{min}.

If for any design combination and design position the minimum moment including second-order moment is greater than the overall design moment then the former is used when comparing the values on the locus of moment of resistance. Note that the minimum might be governing about neither axis, one axis or both axes.

Member imperfections (Clause 5.2 (7))

The imperfection moment is calculated using the eccentricity, $e_i = l_0/400$, and it is conservatively assumed that it increases the first-order moments irrespective of sign. In the case of the Stiffness Method the imperfection moment is added before the moment magnifier is applied. It is applied to both braced and bracing columns/walls.

Curvature Method (Clause 5.8.8)

This method is only applied to symmetrical, rectangular and circular sections and is equally applicable to columns and walls. The second-order moment, M_2 (= N_{Ed} e_2), is calculated but the resulting design moment is only used if it is **less** than that calculated from the Stiffness Method. It is applied in the same manner as that for the Stiffness Method to both braced and bracing columns.

Stiffness Method (Clause 5.8.7)

This method is applied to all columns and walls.

For braced columns the second-order moment M_2 is calculated from:

$M_2 \quad = \quad M_{e.1} \times \pi^2/(8 \times (N_B/N_{Ed} - 1))$

Where,

$M_{e.1} \quad = \quad$ the maximum first-order moment in the mid-fifth

N_B = the buckling load of the column based on nominal stiffness and the effective length

= $\pi^2 EI/l_0^2$

N_{Ed} = the maximum axial force in the design length

When a point of zero shear occurs inside the mid-fifth or does not exist in the member length, the value of M_2 is added algebraically to the first-order moments at the ends but only if this increases the first-order moment. At the mid-fifth position M_2 is always "added" in such a way as to increase the first-order mid-fifth moment.

When a point of zero shear occurs within the member length and is outside the mid-fifth, the second-order moments is taken as the greater of that calculated as above and that calculated as per Clause 5.8.7.3 (4) by multiplying all first-order moments by the amplifier,

$1/(1 – N_{Ed}/N_B)$

For bracing columns the second-order moments are calculated in the same way as braced columns except that in the determination of the amplifier, the buckling load is based on bracing effective lengths. These are greater than 1.0L and hence produce more severe amplifiers.

Second-order analysis

When second-order analysis is selected then both braced and bracing columns are treated the same as if first-order analysis were selected. If the second-order analysis is either the amplified forces method or the rigorous method then this approach will double count some of the global P-Δ effects in columns that are determined as having significant lateral loads. Also, when it is a rigorous second-order analysis there is some double counting of member P-δ effects in both braced and bracing columns.

Design Moment Calculations (Columns: EC2)

For each combination and for each analysis model the end moments in the two local member directions, "1" and "2" are established. From these and the local load profile, the moment at any position and the maximum axial force in the member can be established.

Step 1 - the amplifier

Calculate the "amplifier" due to buckling in each of Direction 1 and Direction 2[1] from Equ. 5.28 and Equ. 5.30 of EC2 as,

$k_{5.28}$ = $1 + \pi^2/(8*(N_B/N_{Ed} – 1))$

$k_{5.30}$ = $1 + 1/(N_B/N_{Ed} – 1)$

Where

N_B = the (Euler) buckling load in the appropriate direction

= $\pi^2 EI/l_o^2$

l_o = the effective length in the appropriate direction which for braced columns will be ≤ 1.0L and for unbraced columns ≥ 1.0L.

N_{Ed} = the maximum axial compressive force in the column length under consideration (stack)

When N_{Ed} ≤ zero i.e. tension, $k_{5.28}$ and $k_{5.30}$ are 1.0.

Step 2 - minimum moment

Calculate the minimum moment due to non-concentric axial force in each of the two directions from,

$M_{min.1}$ = $|N_{Ed}|$ * MAX(h/30, 20)

Where

h = the major dimension of the column in the appropriate direction

N_{Ed} = the maximum axial force (compression or tension) in the column length under consideration (stack)

Step 3 - imperfection moment

Calculate the “first-order” and “second-order” imperfection moment in Direction 1 and Direction 2 as,

$M_{imp.1}$ = N_{Ed} * e_i

$M_{imp.2}$ = $M_{imp.1}$ * $k_{5.28}$

Where

$M_{imp.1}$ = the “first-order” imperfection moment in a given direction

$M_{imp.2}$ = the “second-order” imperfection moment in a given direction

e_i = the effective length in the appropriate direction divided by 400

= $l_o/400$

N_{Ed} = the maximum axial compressive force in the column length under consideration (stack)

When $N_{Ed} \leq$ zero i.e. tension, $M_{imp.1}$ and $M_{imp.2}$ are zero.

Step 4 - second-order moment, curvature method

For rectangular and circular sections[2], the second-order moment, $M_{2.curv}$, using the Curvature Method is calculated for each direction.

$M_{2.curv}$ = $N_{Ed} * e_2$

Where

e_2 = the deflection due to the maximum curvature achievable with the given axial force

= $(1/r)\ l_o^2/c$

N_{Ed} = the maximum axial compressive force in the column length under consideration (stack)

When $N_{Ed} \leq$ zero i.e. tension, $M_{2.curv}$ is zero.

Step 5 - second-order moment, stiffness method

For all section shapes, the second-order moment, $M_{2.stiff}$, using the Stiffness Method is calculated in each direction based on the maximum first-order moment in the mid-fifth of the column, $M_{e.1}$, in the appropriate direction.

$M_{2.stiff}$ = $M_{e.1} * (\pi^2/(8*(N_B/N_{Ed} - 1))$

Where

$M_{e.1}$ = the maximum absolute moment in the mid-fifth of the column length under consideration (stack) in the appropriate direction

N_{Ed} = the maximum axial compressive force in the column length under consideration (stack)

When $N_{Ed} \leq$ zero i.e. tension, $M_{2.stiff}$ is zero.

Step 6 - lateral loading classification

For the current design combination, for each direction using the member analysis routines, check for point(s) of zero shear within the column length. If none exist or are within the mid-fifth of the column length then this design case is designated as having lateral loads that are "not significant". Else the lateral loads are considered as "significant".

Step 7 - design moment at top

Calculate the design moment at the top of the column in each direction (for both braced and unbraced columns) taking into account if lateral loads are "significant", or "not significant".

Step 8 - design moment at bottom

Calculate the design moment at the bottom of the column in each direction (for both braced and unbraced columns) taking into account if lateral loads that are "significant", or "not significant".

Step 9 - design moment in mid-fifth

Calculate the design moment in the mid-fifth of the column in each direction (for both braced and unbraced columns) taking into account if lateral loads that are "significant", or "not significant".

1. Direction 1 and Direction 2 are referring here to the member local axes as defined by the user.

2. This could be extended to include any bisymmetric section e.g. circular and polygonal.

Design for Combined Axial and Bending (Columns: EC2)

Tekla Structural Designer designs the column for an applied axial force and applied bending about one or both axes of the section. In the case of bi-axial bending, a resultant moment is created for the combination of the applied moments.

*A*n iterative approach is applied determined from first principles. This involves the calculation of the neutral axis position (rotation and depth) at which the ratio of the moment limits in each direction is equal to the ratio of the applied moments and the resultant axial resistance of the section is equal to the applied axial force.

When the final neutral axis angle has been found, the program then compares the resultant applied moment with the resultant moment resistance to find the moment utilization ratio:

$$\sqrt{(M^2 major) + (M^2 minor)} \;/\; \sqrt{(M^2 major,res) + (M^2 minor,res)} \leq 1.0$$

where

M_{major} = Moment about the major axis

M_{minor} = Moment about the minor axis

$M_{major,res}$ = Moment of resistance about the major axis

$M_{minor,res}$ = Moment of resistance about the minor axis

Tekla Structural Designer adopts the above approach in preference to the simplified method specified in equation 5.39 of EC2 as it has a wider range of application.

Design for Shear (Columns: EC2)

Design Parameters for Shear Design

For some of the shear design parameters, additional user defined limits can be applied - where this is the case minimum and maximum values are specified in **Design Options > Column > Reinforcement Layout**.

Minimum Shear Link Diameter

$\varphi_{w,min}$ = MAX[6mm, 0.25 * largest longitudinal bar diameter]

Maximum Span Region Shear Link Spacing[1]

For design to **UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$\varphi_{w,max}$ = MIN[20 * smallest longitudinal bar diameter, lesser column dimension, 400mm]

For UK NA when concrete class > C50/60 there are separate calculations in clause 9.5.3(3). These are not implemented but a warning is displayed in this situation.

For design to **EC2 Recommendations:**

$\varphi_{w,max}$ = MIN[20 * smallest longitudinal bar diameter, lesser column dimension, 400mm]

Maximum Support Region Shear Link Spacing[2]

For support regions, the maximum link spacing is reduced by 40%.

Long Term Compressive Strength Factor for Shear, αccw[3]

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

α_{ccw} = 1.0

Design Concrete Compressive Strength for Shear, fcwd

For design to **UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$f_{cwd} = \alpha_{ccw} * \text{MIN} (f_{ck} , 50) /\gamma_C$

For design to **EC2 Recommendations:**

$f_{cwd} = \alpha_{ccw} * f_{ck} /\gamma_C$

Factor[4] CRd,c

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$C_{Rd,c} = 0.18 / \gamma_C$

Factor5 k1

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$k_1 = 0.15$

Cracked Concrete Reduction Factor6, v

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$v = 0.6 * (1 – (f_{ck} / 250))$

Cracked Concrete Reduction Factor7, v1

For design to **UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$v_1 = v* (1 – (0.5 * \cos(\alpha))$

α is the inclination of links.

Note that links in columns are always assumed to be at 90° to column direction.

Therefore $v_1 = v$

For design to **EC2 Recommendations**:

$v_1 = v$

Minimum Shear Reinforcement Ratio8, ρw,min

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$0.08 * \sqrt{(f_{ck})} / f_{yk}$

Maximum Angle of Compression Strut9, θmax

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$\cot(\theta) = 1$

$\theta = 45°$

Minimum Angle of Compression Strut10, θmin

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$\cot(\theta) = 2.5$

$\theta = 21.8°$

Angle of Compression Strut11, θ

For design to UK NA, Irish NA, Malaysian NA and Singapore NA:

If $N_{Ed} \geq 0$ i.e. compression

$\theta = 0.5 * \arcsin(2 * v_{Ed} / (0.9 * \alpha_{cw} * v_1 * f_{cwd}))$

else

$\cot(\theta) = 1.25$

$\theta = 38.7°$

For design to **EC2 Recommendations**:

$\theta = 0.5 * \arcsin(2 * v_{Ed} / (0.9 * \alpha_{cw} * v_1 * f_{cwd}))$

Stress State Factor, αcw

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

If $\sigma_{cp} \leq 0$

$\alpha_{cw} = 1.0$

else if $0 < \sigma_{cp} \leq 0.25 * f_{cd}$

$\alpha_{cw} = 1.0 + (\sigma_{cp} / f_{cd})$

else if $0.25 * f_{cd} < \sigma_{cp} \leq 0.5 * f_{cd}$

$\alpha_{cw} = 1.25$

else if $0.5 * f_{cd} < \sigma_{cp} \leq f_{cd}$

$\alpha_{cw} = 2.5 * (1.0 – (\sigma_{cp} / f_{cd}))$

Minimum Shear Strength[12], vmin

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**:

$0.035 * k^{1.5} * f_{ck}^{0.5}$

1. BS EN 1992-1-1:2004 Section 9.5.3(3)
2. BS EN 1992-1-1:2004 Section 9.5.3(4)
3. BS EN 1992-1-1:2004 Section 3.1.6(1)
4. BS EN 1992-1-1:2004 Section 6.2.2(1)
5. BS EN 1992-1-1:2004 Section 6.2.2(1)
6. BS EN 1992-1-1:2004 Section 6.2.2(6)
7. BS EN 1992-1-1:2004 Section 6.2.3(3)
8. BS EN 1992-1-1:2004 Section 9.2.2(5)
9. BS EN 1992-1-1:2004 Section 6.2.3(2)
10. clause 6.2.3(2)
11. clause 6.2.3(2)

12. clause 6.2.2(1)

Wall Design to EC2

Tekla Structural Designer will design wall panels to resist axial load combined with shear and bending in each of the two planes of the wall.

Limitations and Exclusions (Walls: EC2)

The requirements of clause 9.6 are applied to all walls, irrespective of their length to thickness ratio. (Isolated compression members with a length to thickness ratio less than 4 should be defined as columns rather than walls.)

The following general exclusions also apply:

- Seismic design,
- Consideration of fire resistance. [You are however given full control of the minimum cover dimension to the reinforcement and are therefore able to take due account of fire resistance requirements],
- Lightweight concrete,
- Multi-stack reinforcement lifts.

Materials (Walls: EC2)

Concrete

Only normal weight is included in the current release. (Lightweight concrete is excluded).

Reinforcement

The reinforcement options are:

- Loose reinforcing bars,
- Mesh (Standard Meshes)
- Loose reinforcing bars bent to form links.

Cover to Reinforcement (Walls: EC2)

For 1 layer of reinforcement, the vertical bar is on the centre-line of the wall thickness, the face of the horizontal bar is closest to the critical concrete face.

For 2 layers of reinforcement, the horizontal bars are placed outside the vertical bars at each face.

The nominal concrete cover is measured to the face of the horizontal bar or any link/confinement transverse reinforcement that may be present.

You are required to set a minimum value for the nominal cover, $c_{nom,u}$, for each wall in the wall properties.

This value is then checked against the nominal limiting cover, $c_{nom,lim}$ which depends on the diameter of the reinforcement plus an allowance for deviation, Δc_{dev} (specified in **Design Options > Wall > General Parameters**).

Generally, the allowance for deviation, Δc_{dev} is a NDP.[1] The recommended value is 10mm, but under strict controls it can be reduced to 5mm.

If $c_{nom,u} < c_{nom,lim}$ then a warning is displayed in the calculations.

1. BS EN 1992-1-1:2004 cl 4.4.1.3 (1)P

Design Parameters for Vertical Bars (Walls: EC2)

For some of the vertical bar parameters, additional user defined limits can be applied - where this is the case minimum and maximum values are specified in **Design Options > Wall > Reinforcement Layout**.

> *In the following, the concrete area is the gross area of the general wall, or the gross area of the mid zone if one exists.*
> *For the end zone the design criteria for a reinforced concrete column element applies.*

Minimum Vertical Bar Diameter

For design in accordance with **EC2 Recommendations**;

$\varphi_{v,min}$ = 8mm

For design in accordance with **UK NA, Irish NA, Malaysian NA** and **Singapore NA**;

$\varphi_{v,min}$ = 12mm

Minimum Vertical Bar Spacing[1]

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$s_{cl,min} \geq$ MAX[largest bar diameter, 20mm, d_g + 5mm]

Where d_g is the maximum aggregate size.

Maximum Longitudinal Bar Spacing

You are given control over this value by specifying an upper limit in **Design Options > Wall > Reinforcement Layout**.

Minimum Reinforcement Area[2]

Total minimum area of vertical reinforcement, $A_{s,\,min} = \rho_{v,min} * A_{cg}$

Where

A_{cg} = gross area of the concrete wall

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$\rho_{v, min}$ = 0.002

Where 2 layers are specified distributed equally to each face, this is a minimum of $0.001*A_{cg}$ placed at each face.

You are given control over the minimum reinforcement ratio value via a user limit in **Design Options > Wall > Reinforcement Layout** (default 0.004).

For walls subjected to "predominantly out-of-plane bending", the minimum area rules for "slabs" apply if they are more critical than the above, [cl 9.3 and reference to cl 9.2.1.1 (1) (2) and (3)], so an additional check for any value of minor axis bending is applied.

$A_{s, min}$ = max $[(2*0.26*f_{ctm}*l_{wp}*d/f_{yk})$, $(2*0.0013*l_{wp}*d)$, $(\rho_{v, min}*A_{cg})]$

This applies for the general wall length, or the mid zone if it exists.

For a general wall panel length, $l_{wp} = l_w$

Gross area of the wall, $A_{cg} = l_w*h_w$

For a mid zone panel length, $l_{wp} = l_{mz}$

Gross area of the mid zone, $A_{cg, mz} = l_{mz}*h_w$

Effective depth of the cross section, d, is the dimension of the extreme concrete compression fibre to the centroid of reinforcement layer on the tension side, which for a wall is the line of the vertical reinforcement.

It does not apply for the end zones, since these are subject to the minimum reinforcement requirements as a column section.

Gross area of each end zone, $A_{cg, ez} = l_{ez}*h_w$
Length of each end zone, l_{ez}

Maximum Reinforcement Area[3]

Total maximum area of vertical reinforcement, $A_{s, max} = \rho_{v,max}*A_{cg}$

Where

A_{cg} = gross area of the concrete wall

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$\rho_{v, max}$ = 0.04

You are given control over the maximum reinforcement ratio value via a user limit in **Design Options > Wall > Reinforcement Layout** (default 0.02).

1. BS EN 1992-1-1:2004 Section 8.2

2. BS EN 1992-1-1:2004 Section 9.5.2(2)

3. BS EN 1992-1-1:2004 Section 9.5.2(2)

Design Parameters for Horizontal Bars (Walls: EC2)

For some of the horizontal bar parameters, additional user defined limits can be applied - where this is the case minimum and maximum values are specified in **Design Options > Wall > Reinforcement Layout**.

Minimum Horizontal Bar Diameter

The suggested minimum for design in accordance with **EC2 Recommendations, UK NA, Irish NA, Malaysian NA** and **Singapore NA**;

$\varphi_{h,min}$ = 8mm

Minimum Horizontal Bar Spacing1

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

$s_{cl,min}$ ≥ MAX [largest bar diameter, 20mm, d_g + 5mm]

Where d_g is the maximum aggregate size.

Maximum Longitudinal Bar Spacing

You are given control over this value by specifying an upper limit in **Design Options > Wall > Reinforcement Layout**.

Maximum Horizontal Bar Spacing2

To satisfy the slab condition if “predominantly out-of-plane bending”;

Limiting maximum horizontal spacing, $s_{cr,\,max}$ = min (3*hw , 400 mm)

You are given control over this value by specifying a user limit in **Design Options > Wall > Reinforcement Layout**.

1. BS EN 1992-1-1:2004 Section 8.2

2. BS EN 1992-1-1:2004 Section 8.2

Ultimate Axial Load Limit (Walls: EC2)

The axial resistance calculations for walls are the same as for columns - see: *(Columns: EC2)*

Effective Length and Slenderness Calculations (Walls: EC2)

The slenderness calculations for walls are generally the same as for columns - see: Effective Length Calculations *(Columns: EC2)* and Column Stack Classification *(Columns: EC2)*, except that for walls:

Where the criteria for each axis is:

If $\lambda < \lambda_{lim}$, section is “non-slender”

Elseif $\lambda \geq \lambda_{lim}$, section is “slender”

Since the wall panel has a rectangular plan shape, the calculation can be simplified:

In-plane,

Slenderness, $\lambda_y = l_{0,y} / i_y$

Radius of gyration, $i_y = l_w/(12)^{0.5}$

Effective length, $l_{0,y}$
Length of wall panel, l_w

Out-of-plane,

Slenderness, $\lambda_z = l_{0,z} / i_z$

Radius of gyration, $i_z = h_w/(12)^{0.5}$

Effective length, $l_{0,z}$
Thickness of wall panel, h_w

Pre-selection of Bracing Contribution:

The significant parameter within the slenderness criteria is a choice of how a wall (or column) is contributing to the stability of the structure.

In-plane direction, a wall is usually considered to be a bracing member.
Out-of-plane direction, a wall is usually considered to be braced by other stabilizing members.
These are the default settings but can be edited.

Design Moment Calculations (Walls: EC2)

For each combination, member forces are returned from one or more sets of analyses, in the same way as for columns. For details, see: *Column Design to EC2>* Design Moment Calculations.

Design for Combined Axial and Bending (Walls: EC2)

These calculations are the same whether the design element is a column or a wall -see: Design for Combined Axial and Bending *(Columns: EC2)*

Design for Shear (Walls: EC2)

The shear design calculations are the same whether the design element is a column or a wall - see: Design for Shear *(Columns: EC2)*

Slab Design to EC2

Limitations and Exclusions (Slabs: EC2)

The following general exclusions apply:

- Seismic design
- Consideration of fire resistance. [You are however given full control of the minimum cover dimension to the reinforcement and are therefore able to take due account of fire resistance requirements]
- Lightweight concrete

Materials (Slabs: EC2)

Concrete

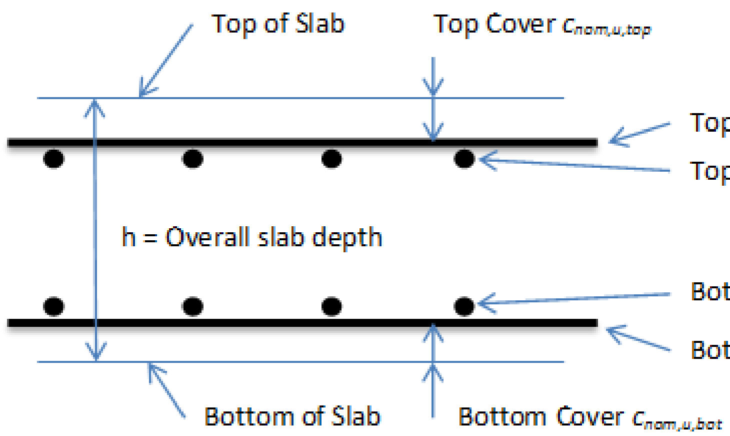
Only normal weight is included in the current release. (Lightweight concrete is excluded).

Reinforcement

The reinforcement options are:

- Loose reinforcing bars
- Mesh (Standard Meshes)
- Loose reinforcing bars bent to form links

Reinforcement Parameters (Slabs: EC2)



Note that when panel and patch reinforcement is considered in combination it is possible that there will be more than one bar size used in a layer, so for the purposes of the calculations in the sections below:

φ_{top1} = the diameter of the **largest** longitudinal reinforcing bar in top layer 1 (the bars nearest to the top surface of the slab)

φ_{top2} = the diameter of the **largest** longitudinal reinforcing bar in top layer 2

φ_{bot1} = the diameter of the **largest** longitudinal reinforcing bar in bottom layer 1 (the bars nearest to the bottom surface of the slab)

φ_{bot2} = the diameter of the **largest** longitudinal reinforcing bar in bottom layer 2

Slab design will always consider a rectangular section of unit width:

h = overall slab depth

b = unit width

for EC2 design the unit width of slab is 1m, and so the design cross section will always be a rectangular section where b = 1000mm

Cover to Reinforcement (Slabs: EC2)

The nominal concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and surface reinforcement where relevant) and the nearest concrete surface.

You are required to set a minimum value for the nominal cover, $c_{nom,u}$, for each slab panel. These values for top and bottom cover are specified in the Reinforcement properties section of the slab panel properties.

This value is then checked against the nominal limiting cover, $c_{nom,lim}$ which depends on the diameter of the reinforcement plus an allowance for deviation, Δc_{dev} (specified in **Design Options > Slab > General Parameters**).

Generally, the allowance for deviation, Δc_{dev} is a NDP.[1] The recommended value is 10mm, but under strict controls it can be reduced to 5mm.

If $c_{nom,u} < c_{nom,lim}$ then a warning is displayed in the calculations.

1. BS EN 1992-1-1:2004 cl 4.4.1.3 (1)P

Limiting Reinforcement Parameters (Slabs: EC2)

Limiting reinforcement parameters are specified in **Design Options > Slab > Reinforcement Layout**.

The parameters applied to “flat slab” design are held separately to those for “beam and slab” design.

Minimum and Maximum Loose Bar Diameter (Slabs: EC2)

Bar diameters are checked against the user defined minimum and maximum sizes specified in **Design Options > Slab > Reinforcement Layout**.

Minimum Loose Bar Diameter

For “flat slab”:

φ_{min} = 8mm (default)

For “beam and slab”:

φ_{min} = 10mm (default)

Maximum Loose Bar Diameter

For “flat slab”:

φ_{min} = 16mm (default)

For “beam and slab”:

φ_{min} = 25mm (default)

Clause 7.3.3 (2) indicates that cracking can be controlled either by restricting the bar diameter or the max spacing. Tekla Structural Designer adopts the latter approach using Table 7.3N- therefore the maximum bar diameters specified in Table 7.2N are not checked.

Minimum Clear Spacing (Slabs: EC2)

The minimum clear horizontal distance between individual parallel bars, $s_{cl,min}$, is given by;[1]

$s_{cl,min} \geq$ MAX[$k_1*\varphi$, d_g+k_2, $s_{cl,u,min}$, 20 mm]

where

k_1 = the appropriate NDP

k_2 = the appropriate NDP

d_g = the maximum size of aggregate (default 20mm)

φ = the maximum diameter of bars in the layer

$s_{cl,u,min}$ = user specified min clear distance between bars (default 100mm in “beam and slab” and 50mm in “flat slab”)

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

k_1 = 1.0

k_2 = 5.0mm

1. BS EN 1992-1-1:2004 Section 8.2(2)

Maximum Spacing of Tension Bars (all slabs) (Slabs: EC2)

In accordance with clause 7.3.3(1) of EC2 for slabs not exceeding 200mm in overall depth and not subjected to significant axial tension the maximum limit on CENTRE to CENTRE bar spacing is governed by clause 9.3 only and there is no need to perform specific checks on the bar spacings to control cracking. These limits are applied to all slabs and then the additional limit in the next section are applied to slabs greater than 200mm thick.

From clause 9.3 the maximum limit on bar spacings can be somewhat subjective so these limits will be user definable with conservative defaults as follows :-

Principal bars (NDP) (cl. 9.3.1.1(3))

s_{max} = 2h but <= 250mm

Secondary bars (NDP) (cl. 9.3.1.1(3))

s_{max} = 3h but <= 400mm

Bars are classed as "secondary" if both the following are true:

1. The design moment for bars in this direction is lower than the design moment for bars in the other direction.
2. The calculated reinforcement requirement based on the design moment is less than the minimum reinforcement requirement.

Maximum Spacing of Tension Bars (h> 200mm) (Slabs: EC2)

The maximum centre to centre bar spacing for crack control, $s_{cr,max}$, is dependent on the maximum allowable crack width, w_{max}, specified in the slab properties from a menu of values which are: 0.20mm, 0.30mm or 0.40mm with a default value of 0.30mm.

The service stress in the reinforcement, σ_s, is given by;

$$\sigma_s = (A_{s,reqd}/A_{s,prov})*(f_{yk}/\gamma_s)*R_{PL}$$

where

$A_{s,reqd}$ = the area of reinforcement required for the maximum design Ultimate Limit State bending moment, M_{Ed}

$A_{s,prov}$ = the area of reinforcement provided

R_{PL} = the permanent load ratio

In the slab properties you are required to supply a value for the permanent load ratio, R_{PL}. A default of 0.65 has been assumed, but you are advised to consider if this is appropriate and adjust as necessary.

The maximum allowable centre to centre bar spacing, $s_{cr,max}$ is then obtained from table 7.3N (shown below) by looking up the calculated value of the service stress in the reinforcement, σ_s, using interpolation between values of σ_s

Steel Service Stress, σ_s (N/mm^2)	**Max Allowable bar Spacing, $s_{cr,max}$**		
	w_{max} = 0.40 mm	**w_{max} = 0.30 mm**	**w_{max} = 0.20 mm**
≤ 160	300	300	200
200	300	250	150
240	250	200	100

280	200	150	50
320	150	100	Warning
360	100	50	Warning
>360	Warning	Warning	Warning

Minimum Area of Reinforcement (Slabs: EC2)

The minimum area of longitudinal tension reinforcement, $A_{s,min}$, is given by;[1]

$A_{s,min} \geq MAX[k_{min1} * b * d * (f_{ctm}/f_{yk}), k_{min2} * b * d]$

where

k_{min1} = the appropriate NDP value

k_{min2} = the appropriate NDP value

f_{ctm} = mean value of the axial tensile strength of the concrete

f_{yk} = characteristic yield strength of the reinforcement

For design in accordance with **UK NA,EC2 Recommendations, Irish NA**, **Malaysian NA** and **Singapore NA**;

k_{min1} = 0.26

k_{min2} = 0.0013

Note that there is no requirement to have a minimum area of compression reinforcement.

1. BS EN 1992-1-1:2004 Section 9.2.1.1(1)

Maximum Area of Reinforcement (Slabs: EC2)

The maximum area of longitudinal tension reinforcement, $A_{st, max}$, is given by[1]

$A_{st, max} \leq k_{max} * A_c$

The maximum area of longitudinal compression reinforcement, $A_{sc, max}$, is given by

$A_{sc, max} \leq k_{max} * A_c$

where

k_{max} = the appropriate NDP value

A_c = the cross sectional area of the slab design section

= h * b

For design to **EC2 Recommendations, UK NA, Irish NA**, **Malaysian NA** and **Singapore NA**;

k_{max} = 0.04

1. BS EN 1992-1-1:2004 Section 9.2.1.1(3)

Basic Cross Section Design (Slabs: EC2)

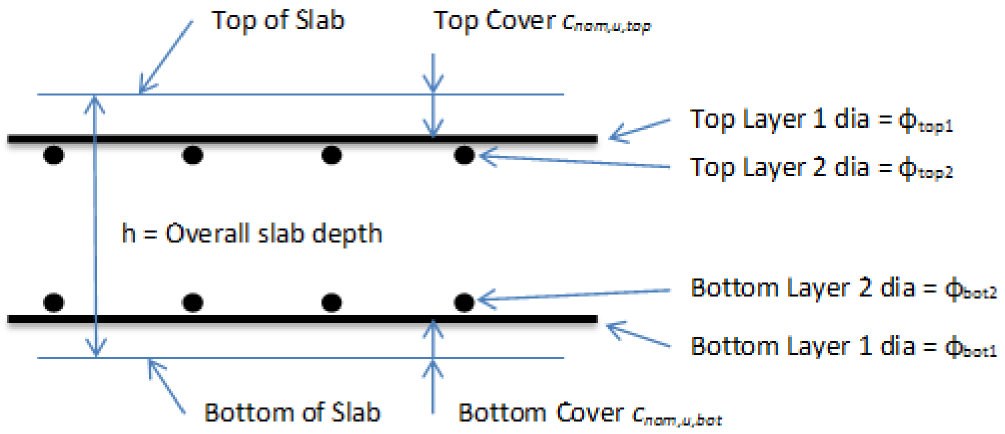
Regardless of whether design is being carried out for a slab panel or a patch, a unit width of slab is always designed for a known design force.

h = overall slab depth

b = unit width

for EC2 design the unit width of slab is 1m, and so the design cross section is a rectangular section where b = 1000mm

Matching Design Moments to Reinforcement Layers



In any panel or patch potentially up to 4 sets of Design Bending Moments are established:

- Mdx-top - used to determine the reinforcement requirement of the x-direction bars in the top of the slab.
- Mdy-top - used to determine the reinforcement requirement of the y-direction bars in the top of the slab
- Mdx-bot - is used to determine the reinforcement requirement of the x-direction bars in the bottom of the slab.
- Mdy-bot - is used to determine the reinforcement requirement of the y-direction bars in the bottom of the slab.

For each set of design bending moments, the effective depths d and d_2 are established - taking account of the direction of the outer bar layer (as specified in the Reinforcement properties section of the slab panel properties).

Design for Bending (Slabs: EC2)

For the design moment under consideration the appropriate d and d_2 are calculated as described above. The design is then basically the same as employed for the design of rectangular beams

Calculate the value of *K* from;

$K = M_{Ed}/(f_{ck}*b*d^2)$

$b =)1000mm$

Then calculate the limiting value of K, known as *K'* from;

$K' = (2*\eta*\alpha_{cc}/\gamma_C)*(1 - \lambda*(\delta - k_1)/(2*k_2))*(\lambda*(\delta - k_1)/(2*k_2))$ for $f_{ck} \leq 50$ N/mm^2

$K' = (2*\eta*\alpha_{cc}/\gamma_C)*(1 - \lambda*(\delta - k_3)/(2*k_4))*(\lambda*(\delta - k_3)/(2*k_4))$ for $f_{ck} > 50$ N/mm^2

where

k_i = moment redistribution factors

δ = moment redistribution ratio (= 1.0 in the current release)

γ_C = the NDP partial safety factor for concrete

α_{cc} = coefficient to take account of long term effects on compressive strength of concrete

λ = 0.8 for $f_{ck} \leq 50$ N/mm^2

= 0.8-(f_{ck}-50)/400 for 50 < $f_{ck} \leq 90$ N/mm^2

η = 1.0 for $f_{ck} \leq 50$ N/mm

= 1.0-(f_{ck}-50)/200 for 50 < $f_{ck} \leq 90$ N/mm^2

For design in accordance with **UK NA**, **Irish NA**, **Malaysian NA** and **Singapore NA**;

$k_1 = 0.40$

$k_2 = 1.0*(0.6+0.0014/\varepsilon_{cu2})$

$k_3 = 0.40$

$k_4 = 1.0*(0.6+0.0014/\varepsilon_{cu2})$

$\gamma_C = 1.5$

$\alpha_{cc} = 0.85$

For design in accordance with **EC2 Recommendations**;

$k_1 = 0.44$

$k_2 = 1.25*(0.6+0.0014/\varepsilon_{cu2})$

$k_3 = 0.54$

$k_4 = 1.25*(0.6+0.0014/\varepsilon_{cu2})$

$\gamma_C = 1.5$

$\alpha_{cc} = 1.0$

IF K ≤ K' THEN compression reinforcement is not required.

Calculate the lever arm, *z* from;

$z = MIN(0.5*d*[1 + (1 – 2*K/(\eta*\alpha_{cc}/\gamma_C))^{0.5}], 0.95*d)$

The area of tension reinforcement required is then given by;

$A_{st,reqd} = M_{Ed}/(f_{yd}*z)$

where

$f_{yd} = f_{yk}/\gamma_S$

γ_S = the NDP partial safety factor for reinforcement

The depth to the neutral axis, x_u is given by;

$x_u = 2*(d-z)/\lambda$

For design in accordance with **UK NA, EC2 Recommendations**, **Irish NA**, **Malaysian NA** and **Singapore NA**;

$\gamma_S = 1.15$

IF K > K' THEN compression reinforcement is required.

Calculate the depth to the neutral axis from;

$x_u = d*(\delta-k_1)/k_2$ for $f_{ck} \leq 50$ N/mm^2

$x_u = d*(\delta-k_3)/k_4$ for $f_{ck} > 50$ N/mm^2

Calculate the stress in the reinforcement from;

$f_{sc} = MIN(E_s*\varepsilon_{cu3}*(1-(d_2/x_u), f_{yd})$

where

d_2 = the distance from the extreme fibre in compression to the c of g of the compression reinforcement

Calculate the limiting bending strength, *M'* from;

$M' = K'*f_{ck}*b*d^2$

Calculate the lever arm from;

$z = 0.5*d*[1 + (1 - 2*K'/(\eta*\alpha_{cc}/\gamma_C))^{0.5}]$

The area of compression reinforcement required, $A_{s2,reqd}$ is given by;

$A_{s2,reqd} = (M_{Ed}-M')/(f_{sc}*(d-d_2))$

The area of tension reinforcement required, $A_{st,reqd}$ is given by;

$A_{st,reqd} = M'/(f_{yd}*z) + A_{s2,reqd}*f_{sc}/f_{yd}$

Deflection Check (Slabs: EC2)

The span-effective depth check only applies to "Beam and Slab" panels. The basic principle is the same as used for beams.

see: Deflection Check *(Beams: EC2)*

Punching Shear Checks (Slabs: EC2)

-

Punching shear limitations and assumptions (Slabs: EC2)

Punching shear limitations and assumptions (Slabs: EC2)

Applicability of wall punching checks (Slabs: EC2)

Checks on walls are made but should be viewed with particular caution.

In particular there is some debate regarding the applicability of a punching check to a long wall - the check doesn't consider the potential for stress concentrations at the ends of the wall.

Columns and Walls not perpendicular to slabs (Slabs: EC2)

EC2 only provides specific design guidance for rectangular columns which are perpendicular to slabs. The program treats all columns and walls that are not perpendicular to slabs as if they are for the punching areas developed.

This is conservative as the punching area/perimeter will be smaller than that for the angled column or wall.

Loaded perimeter near slab edges (Slabs: EC2)

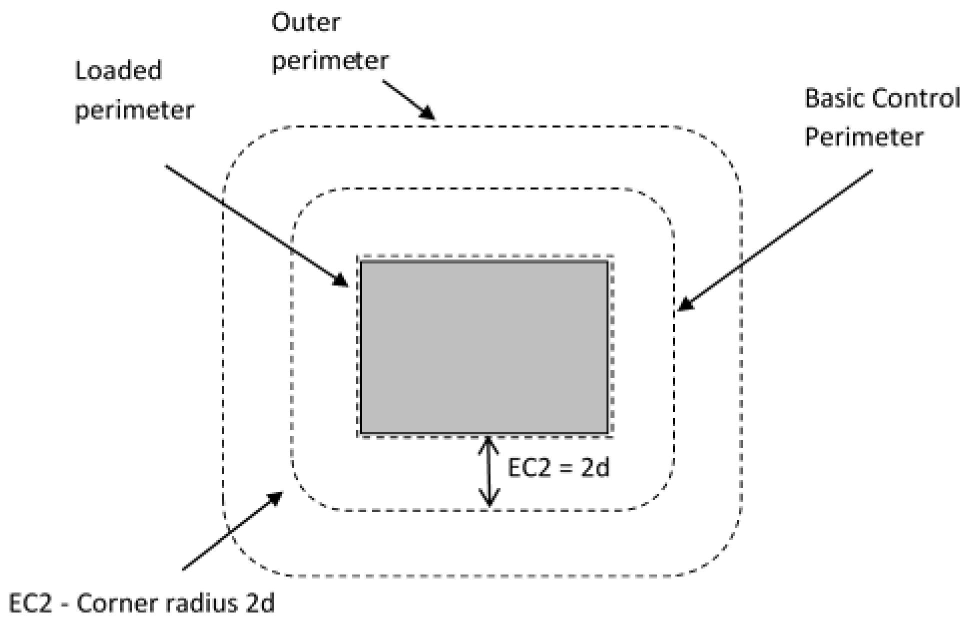
EC2 only provides specific design guidance for rectangular columns with this guidance being further limited, for the case of edge and corner columns, to those cases where the edge(s) of the slab coincide with the edge(s) of the column. In the program the equations for other column shapes and scenarios are therefore obtained by modifying those equations presented in EC2. It is considered that the modified equations result in either the correct perimeter length being obtained or a conservative value i.e. an underestimate of the perimeter length.

Control Perimeters (Slabs: EC2)

The calculations are beyond scope in the following situations:

- If two control areas touch then both areas are set to Beyond Scope.
- If an edge or corner area contains another column or wall then both areas are set to Beyond Scope.

Punching shear perimeters (Slabs: EC2)



There are a number of perimeters associated with Punching Shear

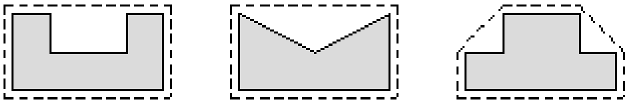
- Loaded perimeter - perimeter around the loaded area - eg face of the wall or column
- Basic control perimeter - is the check punching shear perimeter 2d from the loaded perimeter
- Outer perimeter - the first perimeter at which the punching check passes with no need for shear reinforcement - equal to or outside the basic control perimeter (equal to if the check passes at the basic control perimeter).

Length of the loaded perimeter u0 (Slabs: EC2)

Loaded perimeter for Columns

The length of the loaded perimeter at the column face is calculated in accordance with clause 6.4.5(3) of EC2 as determined below.

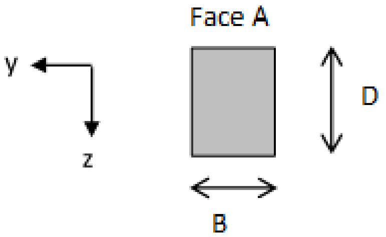
Note - for columns which have a re-entrant corner, ie where an internal angle is greater than 180 degrees, the length of a side and the slab/column interface is adjusted as indicated in the sketches below with the perimeter taken as the shortest distance around the column.



The following are the loaded perimeters for the possible column shapes. Each has a bounding rectangle or circle to aid in the design calculations.

Note all columns shown at 0 deg orientation looking down on column - face A to the top of each depiction.

Rectangular (D and B)



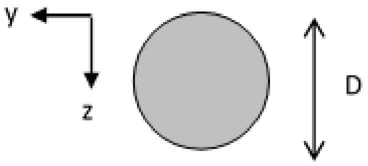
u_0 = 2 x (D + B)

Bounding rectangle D_{bound} = D

Bounding rectangle B_{bound} = B

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

Circular (D)

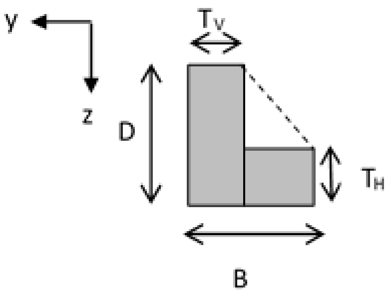


$u_0 = \pi$ x D

Bounding circle D_{bound} = D

Bounding circle perimeter - $u_{0bound} = \pi$ x D_{bound}

L section (D, B, T_H and T_V)



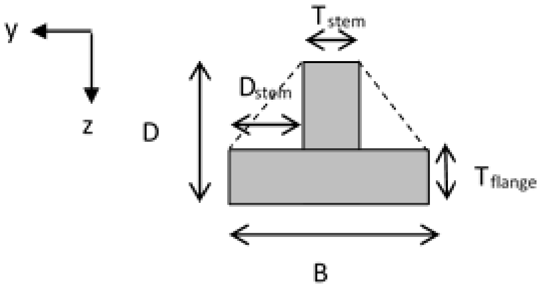
$u_0 = D + B + T_V + T_H + \mathrm{Sqrt}((B-T_V)^2 + (D-T_H)^2)$

Bounding rectangle D_{bound} = D

Bounding rectangle B_{bound} = B

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

T section (D, B, T_{stem}, T_{flange} and D_{stem})



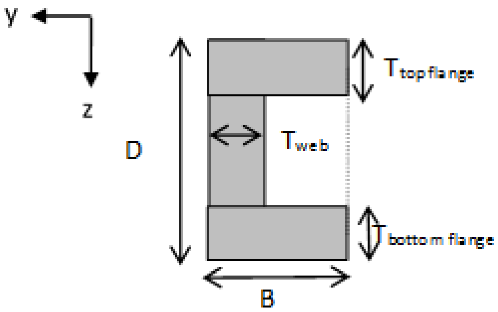
$u_0 = B + 2xT_{flange} + T_{stem}$ + Sqrt $((D_{stem}{}^2+(D-T_{stem})^2)$ + Sqrt$((B-T_{stem}-D_{stem})^2+(D-T_{stem})^2)$

Bounding rectangle D_{bound} = D

Bounding rectangle B_{bound} = B

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

C section (D, B, T_{web}, $T_{top\ flange}$ and $T_{bottom\ flange}$)



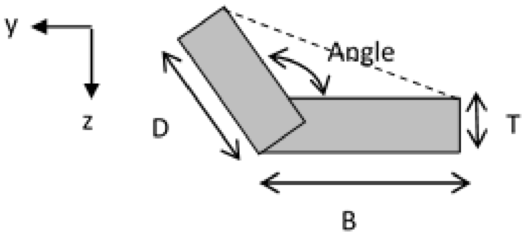
$u_0 = 2 \times (B + D)$

Bounding rectangle D_{bound} = D

Bounding rectangle B_{bound} = B

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Elbow (D, B, T, angle)



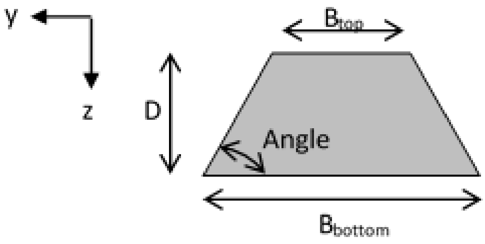
u_0 = B + D + 2 x T + Sqrt((B + D x Sin(angle-90)-T x Cos(angle-90))^2 + (D x Cos(90-angle) - T)^2)

Bounding rectangle D_{bound} = D Sin(180-Angle) x T Cos(180-Angle)

Bounding rectangle B_{bound} = B + D Cos(180-Angle)

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Trapezium (D, B_{bottom}, B_{top}, angle)



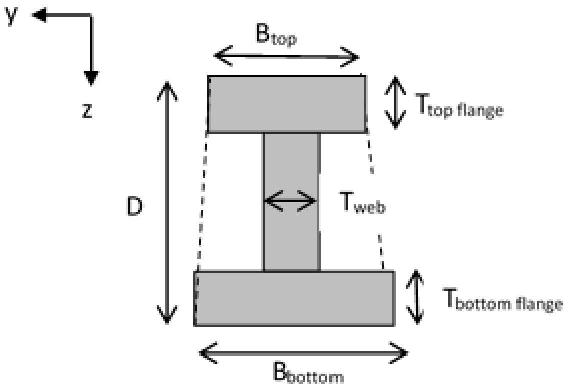
$u_0 = B_{top} + B_{bottom} + 2 \times D / \sin(Angle)$

Bounding rectangle $D_{bound} = D$

Bounding rectangle $B_{bound} = \max(B_{bottom}, B_{top})$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

I section (D, B_{top}, B_{bottom}, T_{web}, $T_{top\ flange}$, $T_{bottom\ flange}$)



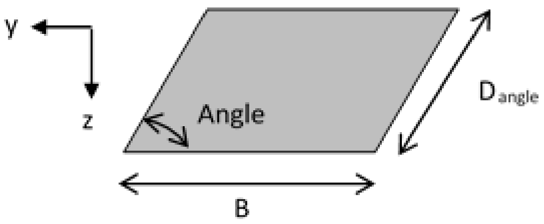
$u_0 = B_{bottom} + B_{top} + 2 \times Sqrt(((B_{bottom}-B_{top})/2)^2 + D^2)$ (approx)

Bounding rectangle $D_{bound} = D$

Bounding rectangle $B_{bound} = \max(B_{bottom}, B_{top})$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Parallelogram (D_{angle}, B, angle)



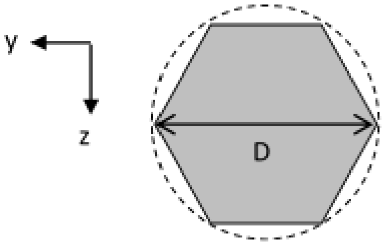
$u_0 = 2 \times (B + D_{angle})$

Bounding rectangle $D_{bound} = D \times \sin(Angle)$

Bounding rectangle $B_{bound} = B + D_{angle} \times \cos(Angle)$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Polygon (D, n) - n >4

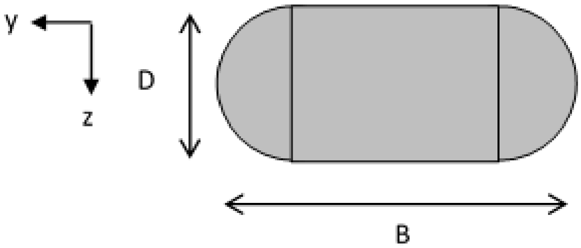


$u_0 = 2 \times n \times D/2 \times \sin(180/n)$

Bounding circle $D_{bound} = n \times D/\pi \times \sin(180/n)$ (equivalent perimeter)

Bounding circle perimeter - $u_{0bound} = \pi \times D_{bound}$

Lozenge (D, B)



$u_0 = 2 \times (B - D) + \pi D$

Bounding rectangle $D_{bound} = D$

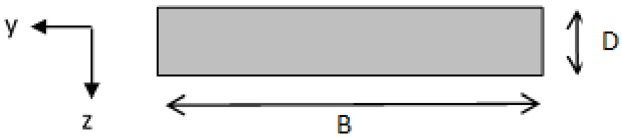
Bounding rectangle $B_{bound} = B$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Loaded perimeter for Walls

The length of the loaded perimeter at the wall face may be calculated in accordance with clause 6.4.5(3) of EC2 as determined below.

Rectangular (D and B)



$u_0 = 2 \times (D + B)$

Bounding rectangle $D_{bound} = D$

Bounding rectangle $B_{bound} = B$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Loaded perimeter for Point Loads

The length of the loaded perimeter at the point load may be calculated in accordance with clause 6.4.5(3) of EC2 as determined below.

$u_0 = 2 \times (D_{load} + B_{load})$

Bounding rectangle $D_{bound} = D_{load}$

Bounding rectangle $B_{bound} = B_{load}$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Additional Loaded perimeter drops

The additional loaded perimeter for a column/wall with a drop is defined by the perimeter of the rectangular drop

$u_{0drop} = 2 \times B_{drop} \times D_{drop}$

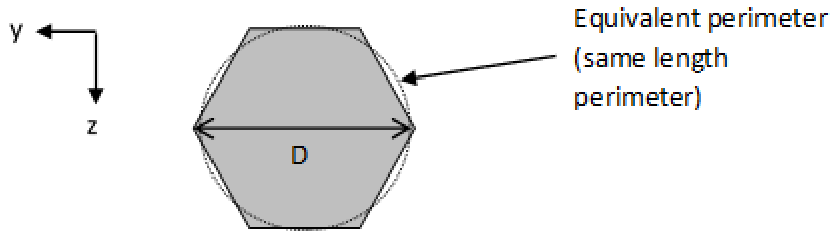
The equivalent perimeter

Note that we use an equivalent rectangle or circle in the calculations for non rectangular and non circular columns to allow us to handle openings and moments in a slightly simplified manner.

The equivalent rectangle or circle has the same u_0 and u_1 value as the "true" punching shape. So for axial loads the identical answer is given. For openings and moments, then there is a small deviation from the "true" result - but in the context of punching, the net effect will be small.

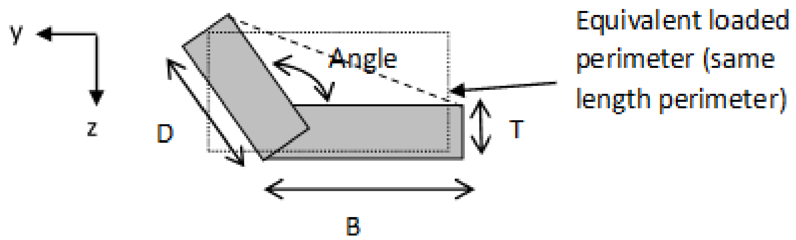
For "circular" shapes of column (circle and polygon with n sides), the equivalent perimeter -

- $D_{equiv} = D_{bound} \times u_0 / u_{0bound}$



For "rectangular" shapes of column (all except circle and polygon of n sides) and walls, the equivalent loaded perimeter -

- $D_{equiv} = D_{bound} \times u_0 / u_{0bound}$
- $B_{equiv} = B_{bound} \times u_0 / u_{0bound}$



The equivalent perimeter is used in three situations

- adjustment of the loaded perimeter length/shape u_0 for edge and corner columns/walls
- in calculation of beta for edge and corner columns/walls.

- Reduction in V_{Ed}

Length of the basic control perimeter u1 (Slabs: EC2)

Basic control perimeter without drops

The length of the column/wall basic control perimeter is the length as determined below.

- For all internal column/wall shapes and point loads

  $u_1 = u_0 + 4 \pi d$

- For all corner column/wall shapes and point loads

  $u_1 = A + B + C$

where

for a rectangle

A = dist from centroid to edge along local y

$B = c_1/2 + \pi \times d + c_2/2$

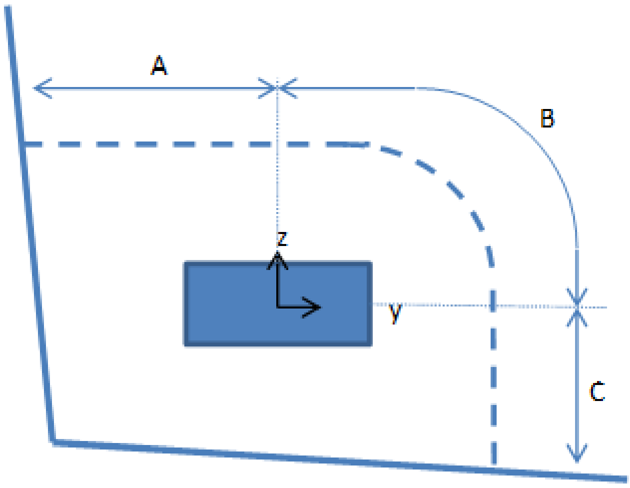
C = dist from centroid to edge along local z

For a circle

A = dist from centroid to edge along local y

$B = \pi \times (4 \times d + c)/4$

C = dist from centroid to edge along local



- For all edge column/wall shapes and point loads

$u_1 = A + B + C$

where

For a rectangle

A = dist from centroid to edge along local y or local z

$B = c_1 + 2 \times \pi \times d + c_2$

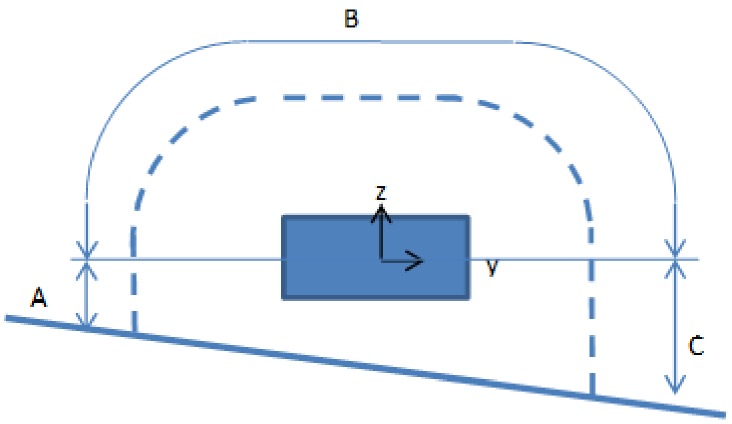
C = dist from centroid to edge along local y or local z

For a circle

A = dist from centroid to edge along local y or local z

B = π x (4 x d + c) / 2

C = dist from centroid to edge along local y or local z



If a basic control perimeter passes across a slab edge then only the perimeter length in the slab is counted in u_1.

Note if a slab around a column/wall/point load changes depth, the thinnest slab and its "d" values above is used.

Modification of control perimeters to take account of slab openings (Slabs: EC2)

If any openings have been defined in the slab and if the nearest opening edge is not greater than 6d from the face of the column then the length of the loaded perimeter at the column face, u_0, and the length of the basic control perimeter, u_1, are both reduced to take account of the presence of the opening(s) in accordance with fig. 6.14 of EC2.

When a perimeter length has been reduced to cater for openings - as the exact position of the opening in relation to the reinforcement strips is not known, the calculations conservatively ignore any patch reinforcement in the punching checks - only the slab reinforcement is used.

User Modification of control perimeters

If you want to consider the effect of openings, but do not want to place them in the slab, this can be done by defining the following properties:

- u_0 - user reduction
- u_1 - user reduction

When applied, the length of the respective shear perimeters (except that at the column/wall face) are reduced by the specified amount.

Basic design procedure (Slabs: EC2)

The basic design procedure applied to an internal column is described below:

Required area of punching shear reinforcement

A_{sw}/s_r = (u_1 /1.5 $f_{ywd,ef}$) x ($V_{Ed,1}$ – (0.75 $V_{Rd,c}$))

where

$f_{ywd,ef}$ = the design strength of the reinforcement

A_v = is the cross sectional area of all legs of reinforcement on one peripheral line

s_r = the spacing of link legs away from the column faces

Calculate the provided area of punching shear reinforcement

$A_{sw,provided}/s_r$= N_s × d_{sr} /s_r

where

N_s = is the number of the rebars/studs per perimeter

d_{sr} = is the cross sectional area of single reinforcement element (stud/stirrup)

s_r = is the spacing between the reinforcement perimeters

Check area of punching shear reinforcement

IF $A_{sw,provided}/ s_r \geq A_{sw}/ s_r$ – the choice of bars (number & diameter) and their spacing are adequate.

Otherwise, if auto-design is active a new arrangement is tried; the number of bars, and diameter are increased and the spacing decreased until the check passes.

Length of the outer control perimeter where shear reinforcement not required

u_{out}= β $V_{Ed,red}$/($v_{Rd,c}$ × d)

Determine the required length of the outer perimeter of reinforcement

Lr_{req}= R_{out} - s_2

Where

R_{out} = the distance of the outer control perimeter from the column face and

s_2 = the spacing between the outer control perimeter and the outer perimeter of reinforcement

= k × d

(k = 1.5 for Eurocode, 1.5 for UK NA, (Irish, Singapore, Malaysia)

Determine the provided distance to the outer perimeter of reinforcement

Lr_{prov} = s_0 + (N - 1) × s_r

Check length of punching shear reinforcement

IF $Lr_{prov} \geq Lr_{req}$ – check passes

Otherwise, if auto-design is active the number of studs/stirrups are increased until the check passes.

Magnification factor, beta (Slabs: EC2)

The magnification factor β is used to increase the basic transfer shear force V_{Ed} to take account of the increase in shear stress across part of the control perimeter due to the moment transferred into the column. It is calculated differently depending on whether the column is internal, at an edge or at a corner.

User limit on beta for internal columns

If you want to set a lower limit on the magnification factor β, this can be done by selecting **Beta - user limit** in the punching check properties.

When checked, the calculated value of β will be a minimum of 1.15 or greater.

Pad and Strip Base Design to EC2 and EC7

Pad and strip bases are designed to resist the applied forces and moments for the three phenomena of bending, beam shear and punching shear.

Base calculations are performed in accordance with Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings EN1992-1-1:2004 and Eurocode 7: Geotechnical design - Part 1: General rules EN1997-1:2004.

Checks Performed (Pad and Strip Base: EC2)

The checks performed for both directions are:

- EC7 - Max soil bearing pressure must not exceed allowable bearing pressure.
- EC2 - Provided steel must be greater than $A_{s,min}$ for both vertical directions.
- EC2 - Provided bar spacing must be inside the limiting spacing
- EC2 - Provided bar size must be inside the limiting sizes
- EC2 - Check for bending moment capacity

- EC2 - Check for shear capacity
- EC2 - Punching check at column face
- EC2 - Punching check at critical perimeter
- EC7 - Check for overturning forces - **not in the current release**
- EC7 - Check for sliding
- EC7 - Check for uplift

Foundation Bearing Capacity (Pad and Strip Base: EC2)

Foundation Bearing Capacity (Pad and Strip Base: EC2)

Annex A of EC7 allows bearing capacity to be checked using two sets of partial factors: A1 and A2.

In *Tekla Structural Designer* the bearing capacity check is performed on STR load combinations using set A1 and on GEO load combinations using set A2.

Alternatively, an option is also provided to check a "Presumed Bearing Resistance" in accordance with EN1997-1cl.6.5.2.4).

Check for Pad Base Bearing Capacity

Total vertical force:

$F_{dz} = \gamma_G*(F_{swt} + F_{soil} + F_{sur,G}) + \gamma_Q* F_{sur,Q} - F_{z,sup}$

Moment about X axis:

$M_{x,c} = M_{x,sup} + F_{z,sup}* y_1 + h*F_{y,sup}$

Moment about Y axis:

$M_{y,c} = M_{y,sup} + F_{z,sup}* x_1 + h*F_{x,sup}$

Where:

L_x = Length of foundation in X-direction

L_y = Length of foundation in Y-direction

A = $L_x * L_y$ = Foundation area

h = Depth of foundation

h_{soil} = Depth of soil above the foundation

l_x = Length of column/wall in X-direction

l_y = Length of column/wall in Y-direction

x_1 = Offset in X-axis. (Distance between centre of the pad to the centre of the support in X-direction)

y_1 = Offset in Y-axis. (Distance between centre of the pad to the centre of the support in Y-direction)

γ_G = 1.35 = Permanent partial factor - unfavourable action when Set A1 used

= 1.0 = Permanent partial factor - unfavourable action when Set A2 used

γ_Q = 1.5 = Variable partial factor - unfavourable action when Set A1 used

= 1.3 = Variable partial factor - unfavourable action when Set A2 used

F_{swt} = $A * h * \gamma_{conc}$ = foundation self-weight

F_{soil} = $(A - A_c)*h_{soil}*\gamma_{soil}$ = Unfactored load from soil

γ_{soil} = Density of soil - user input

$F_{sur,G}$ = $(A - A_c)*sc_G$ = Unfactored load from surcharge for permanent load case

$F_{sur,Q}$ = $(A - A_c)*sc_Q$ = Unfactored load from surcharge for variable load case

sc_G = Surcharge in permanent load case - user input

sc_Q = Surcharge in variable load case - user input

A_c = cross section of the column/wall

$F_{z,sup}$ = Vertical load acting on support in STR/GEO limit states– (from analysis)

$M_{x,sup}$ = Moment acting on support around X-axis in STR/GEO limit states– from analysis

$M_{y,sup}$ = Moment acting on support around Y-axis in STR/GEO limit states – from analysis

$F_{x,sup}$ = Horizontal force acting on support X-direction in STR/GEO limit states – from analysis

$F_{y,sup}$ = Horizontal force acting on support Y-direction in STR/GEO limit states – from analysis

Eccentricity in X-direction:

$e_x \quad = \quad M_{y,c} / F_{dz}$

Eccentricity in Y-direction:

$e_y \quad = \quad M_{x,c} / F_{dz}$

If

$abs(ex) / Lx + abs(ey) / Ly \quad \leq \quad 0.167$

Then Base reaction acts within middle third - no loss of contact and:

Effective length in X-direction:

$L'_x \quad = \quad L_x - 2e_x$ when $e_x > 0$

$L'_x \quad = \quad L_x + 2e_x$ when $e_x < 0$

Effective length in Y-direction:

$L'_y \quad = \quad L_y - 2e_y$ when $e_y > 0$

$L'_y \quad = \quad L_y + 2e_y$ when $e_y < 0$

Design bearing pressure:

$f_{dz} \quad = \quad F_{dz} / (L'_x * L'_y)$

Else base reaction acts outside middle third - loss of contact.

In this case the pressure calculations are more complex - in *Tekla Structural Designer* these are done using sets of equations presented in an article by Kenneth E. Wilson published in the Journal of Bridge Engineering in 1997.

Check for Strip Base Bearing Capacity

The principles used in the strip base bearing capacity calculations are similar to those for pad foundations. Only the direction X is checked (around Y-axis) using segment widths.

Design bearing pressure:

$f_{dz} \quad = \quad F_{dz} / (L'_x * L_y)$

Design for Bending (Pad and Strip Base: EC2)

Bending design calculations are performed for the STR load combinations.

For tension on the bottom face of the foundation, the design bending moment may be taken as that at the face of the column or wall and may therefore be less than the peak bending moment.

The bending capacity check follows the same basic principle as used for beams.
See: Beam Design - Design for Bending for Rectangular Sections .

Checks for Limiting Parameters (Pad and Strip Base: EC2)

Checks for Limiting Parameters (Pad and Strip Base: EC2)

Limiting reinforcement parameters are specified in **Design Options > Foundations > Isolated Foundations > Reinforcement Layout**

Limits on bar size and reinforcement quantities

The minimum bar diameter:

ϕ_{min} = 8mm   EC2 recommendations

is an NDP (cl.9.8.2.1(1)):

ϕ_{min} = 8mm   for UK, Irish and Malaysian NA

ϕ_{min} = 10mm   for Singapore NA

The maximum bar diameter:

ϕ_{large} = 32mm   EC2 recommendations

ϕ_{large} = 40mm   for UK, Irish and Malaysian and Singapore NA

If the design results in a bar size with $\varphi > \varphi_{large}$ then a 'Warning' is displayed.

The minimum area of tension reinforcement (NDP) ((cl. 9.3.1.1(1)) for EC2 recom., UK, Irish, Malaysian and Singapore NA

$A_{s,min} \geq \max(0.26 * f_{ctm} / f_{yk}, 0.0013) * b * d$ for X-direction

$A_{s,min} \geq \max(0.26 * f_{ctm} / f_{yk}, 0.0013) * b * d$ for Y-direction

where b = unit width

The maximum area of tension reinforcement (NDP) ((cl. 9.3.1.1(1)) for EC2 recom., UK, Irish, Malaysian and Singapore NA

$A_{s,max} = 0.04 * b * d$

Limits on bar spacing

The minimum clear horizontal distance between individual parallel bars, $s_{cl,min}$, is given by;[1]

$s_{cl,min} \geq \mathrm{MAX}[k_1*\varphi, d_g+k_2, s_{cl,u,min}, 20\text{ mm}]$

where

k_1 = the appropriate NDP

k_2 = the appropriate NDP

d_g = the maximum size of aggregate

φ = the maximum diameter of bar

$s_{cl,u,min}$ = user specified min clear distance between bars

For design to EC2 Recommendations, UK NA, Irish NA, Malaysian NA and Singapore NA;

k_1 = 1.0

k_2 = 5.0mm

The maximum centre to centre bar spacing for crack control, $s_{cr,max}$, is dependent on the maximum allowable crack width, w_{max}, specified in the base properties from a menu of values which are: 0.20mm, 0.30mm or 0.40mm with a default value of 0.30mm.

The service stress in the reinforcement, σ_s, is given by;

$$\sigma_s = (A_{s,reqd}/A_{s,prov})*(f_{yk}/\gamma_s)*R_{PL}$$

where

$A_{s,reqd}$ = the area of reinforcement required for the maximum design Ultimate Limit State bending moment, M_{Ed}

$A_{s,prov}$ = the area of reinforcement provided

R_{PL} = the permanent load ratio

In the base properties you are required to supply a value for the permanent load ratio, R_{PL}. A default of 0.65 has been assumed, but you are advised to consider if this is appropriate and adjust as necessary.

The maximum allowable centre to centre bar spacing, $s_{cr,max}$ is then obtained from table 7.3N (shown below) by looking up the calculated value of the service stress in the reinforcement, σ_s, using interpolation between values of σ_s

Steel Service Stress, σ_s (N/mm^2)	**Max Allowable bar Spacing, $s_{cr,max}$**		
	w_{max} = 0.40 mm	**w_{max} = 0.30 mm**	**w_{max} = 0.20 mm**
≤ 160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50

320	150	100	Warning
360	100	50	Warning
>360	Warning	Warning	Warning

1. BS EN 1992-1-1:2004 Section 8.2(2)

Shear Design (Pad and Strip Base: EC2)

Shear Design (Pad and Strip Base: EC2)

Topics in this section

Pad base shear design check

Calculate tension reinforcement ratio (cl 6.2.2(1)):

ρ_l = $\min(A_{sl} / (L*d), 0.02)$

Where:

A_{sl} = area of tension reinforcement

L = unit width of foundation in which A_{sl} is provided

d = effective depth of reinforcement in direction considered

$F_{y,sup}$ = Horizontal force acting on support Y-direction in STR/GEO limit states – from analysis

Calculate k (cl. 6.2.2(1)):

k = $\min(1 + (200mm/d)^{1/2}, 2.0)$

Calc min shear strength (NDP) (cl. 6.2.2(1)):

v_{min} = $0.035k^{3/2}f_{ck}^{1/2}$ EC2 recommendations

v_{min} = $0.035k^{3/2}f_{ck}^{1/2}$ for UK, Irish, Malaysian and Singapore NA

Calculate resistance without shear reinforcement for X and Y directions (cl. 6.2.2(1)):

$v_{Rd.c,x}$ = $\max(C_{Rd.c} * k * (100\ N^2/mm^4 * \rho_l * f_{ck})^{1/3}, v_{min})$

$v_{Rd.c,y}$ = $\max(C_{Rd.c} * k * (100\ N^2/mm^4 * \rho_l * f_{ck})^{1/3}, v_{min})$

Maximum allowable shear resistance (cl. 6.2.2(6));

$v_{Rd,max}$ = $0.5 * v * f_{cd}$

where

f_{cd} = concrete design compressive strength

v = cracked concrete reduction factor

If applied design shear force is less than or equal to the shear resistance i.e $v_{Ed} \leq v_{Rd}$ = min ($v_{Rd,c}$, $v_{Rd,max}$) the foundation thickness is adequate for beam shear.

Strip base shear design check

The principle of the strip base shear design check is similar to that for the pad base. Only the direction X is checked (around Y-axis) using segment widths.

Punching Shear Design (Pad and Strip Base: EC2)

Punching shear checks are carried out for pad foundations only, using STR load combinations.

Punching shear should be checked at the face of the column and clause 6.4.4(2) of EC2 states that punching shear should also be checked at perimeters within 2d from the column face where d is the average effective depth of the tension reinforcement in the two orthogonal directions.

In *Tekla Structural Designer* punching shear is checked at 9 locations i.e. at the column face and at the control perimeter located at 0.25d, 0.5d, 0.75d, d, 1.25d, 1.5d, 1.75d and 2d from the face of the column. The design check from the column face and the most critical from the rest of the locations being reported.

The punching shear checks for pad bases follow the same basic principle as used for mats. see: Punching Shear Checks *(Slabs: EC2)*

The main differences between mat and pad base punching shear checks are:

- Checks at multiple perimeters up to 2d are required in pad base punching checks.
- Column Local axes are always parallel with the pad base edges in the pad base punching checks.
- Loads from the column are always above the pad base (one direction).
- No openings can be placed in pad bases.
- No shear reinforcement is used in pad bases.

Check for Overturning Forces (Pad and Strip Base: EC2)

Checks for overturning forces are beyond scope in the current release of Tekla Structural Designer.

Check for Sliding (Pad and Strip Base: EC2)

The check for sliding is carried out for pad foundations only.

If there is no horizontal force acting on foundation check for sliding is not required.

Sliding resistance (EC7 Section 6.5.3) – forces are defined from STR combinations.

Horizontal Forces on foundation for each direction:

F_{dx} = $F_{x,sup}$

F_{dy} = $F_{y,sup}$

Where:

$F_{x,sup}$ = factored horizontal force acting on support in X-dir. (from analysis)

$F_{y,sup}$ = factored horizontal force acting on support in Y-dir. (from analysis)

Horizontal force on foundation:

H_d = $[abs(F_{dx})^{2} + abs(F_{dy})^{2}]^{0.5}$

Sliding resistance verification (Section 6.5.3)

Sliding resistance (exp.6.3b and table A.5):

$R_{H.d}$ = $[F_{zG,d} + \gamma_{Gf} * F_{swt}] * \tan(\delta_k) / \gamma_{R.h}$

Where:

δ_k = factored horizontal force acting on support in X-dir.

$\gamma_{R.h}$ = 1.1 (set R2) EC7 recomm. and for Irish and Malaysian NA

$\gamma_{R.h}$ = 1.0 (set R1) for UK, Singapore NA

γ_{Gf} = 1.0 (permanent favourable action)

$F_{zG,d}$ = Vertical load acting on support in STR/GEO limit states where favourable actions considered.

Check for Uplift (Pad and Strip Base: EC2)

For combinations producing tension at the support the tension value is compared to the stabilizing loads. Auto-design can automatically increment the base size to achieve a passing status.

Pile Cap Design to EC2

The forces acting on a pile cap are applied to the foundation at the foundation level. The foundation can take axial load and bi-axial shear and moment.

Pile cap design is divided between pile design (pile capacity check) and structural design of the pile cap which includes bending, shear and punching shear design checks.

Pile cap calculations are performed in accordance with Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings EN1992-1-1:2004.

Bottom reinforcement is designed to the Eurocode - (EC base, UK NA, Irish NA, Singapore NA or Malaysia NA).

Pile Capacity (Pile Cap: EC2)

Annex A of EC7 allows bearing capacity to be checked using two sets of partial factors: A1 and A2.

In *Tekla Structural Designer* the pile capacity check is performed on STR load combinations using set A1 and on GEO load combinations using set A2.

Combination	Generate	Dead	Imposed
STR_1-1.35G+1.5Q+1.5RQ	☑	1.350	1.500
STR_2-1.35G+1.5Q+1.5RQ+EHF	☑	1.350	1.500
GEO_1-G+1.3Q+1.3RQ+EHF	☑	1.000	1.300

Pile capacity passes if:

$R_{c,d} \geq P_n \geq -R_{t,d}$

Where:

$R_{c,d}$ = Pile design compression resistance

$R_{t,d}$ = Pile design tension resistance

P_n = Pile load

Design for Bending (Pile Cap: EC2)

The pile cap is treated as a beam in bending, where the critical bending moments for the design for the bottom reinforcement are taken at the face of the column.

Bending design calculations are performed for the STR load combinations.

The bending capacity check follows the same basic principle as used for beams - see: Design for Bending for Rectangular Sections *(Beams: EC2)*

Shear Design (Pile Cap: EC2)

Shear design calculations are performed for the STR load combinations.

Determination of Design Shear Stress

Shear stress acting on side 1 in direction X $v_{Ed,x1} = \Sigma P_{n,1} / (d_x*L_y)$

Shear stress acting on side 2 in direction X $v_{Ed,x2} = \Sigma P_{n,2} / (d_x*L_y)$

Shear stress acting on side 1 in direction Y $v_{Ed,y1} = \Sigma P_{n,1} / (d_y*L_x)$

Shear stress acting on side 2 in direction Y $v_{Ed,y2} = \Sigma P_{n,2} / (d_y*L_x)$

Maximum allowable shear resistance[1]

$v_{Rd,max} = 0.5 * v * f_{cd}$

where:

$f_{cd} = \alpha_{cc} * f_{ck} / \gamma_c$

$\alpha_{cc} = 1.0$ (EC2); $\alpha_{cc} = 0.85$ (supported NAs)

$\gamma_c = 1.5$

$v = 0.6 * [1-(f_{ck}/250)]$

Check for Shear

The shear capacity check procedure is identical to that for pad bases - see: Pad base shear design check *(Pad and Strip Base: EC2)*

1. BS EN 1992-1-1:2004 Section 6.2.2(6)

Punching Shear Design (Pile Cap: EC2)

Punching shear checks are performed for the column and the individual piles.

Columns

The punching shear check is similar to that for pad bases, but with the following difference:

- the shear force at a perimeter uses the value from the column reduced by pile loads within the perimeter

See: Punching Shear Design *(Pad and Strip Base: EC2)*

Piles

The punching shear check is similar to that for pad bases, but with the following differences:

- variable d is replaced with d_{red} where d_{red} =min (h – “pile penetration depth”, average reinforcement effective depth)
- no moments act on top of the pile, only axial load considered
- shear stress at the column face is checked only for the pile with the largest pile load:
  - $v_{Ed,0} = \beta * P_{n,max} / (u_0*d)$

See: Punching Shear Design *(Pad and Strip Base: EC2)*

Checks for Limiting Parameters (Pile Cap: EC2)

Limiting reinforcement parameters are specified in **Design Options > Foundations > Isolated Foundations > Reinforcement Layout**

Check for distance of pile cap overhang

Check pile edge distance "e" for pile "i" in a pile group for both directions:

The check passes if:

If min $e_i > e_{min,user}$

Check for minimum pile spacing

Check centre to centre spacing "s" between piles "i" and "j" in a pile group:

The check passes if:

If $s_{ij} > s_{min,user}$

where

$s_{min,user}$ = user input

Check for maximum pile spacing

Check centre to centre maximum spacing "s" between piles "i" and "j" in a pile group:

The check passes if:

If $s_{ij} < s_{max,user}$

$s_{max,user}$ = user input

Other checks

The remaining checks are identical to those for pad bases - see: Checks for Limiting Parameters *(Pad and Strip Base: EC2)*

References (EC2)

1. **British Standards Institution**. *BS EN 1992-1-1:2004. Eurocode 2: Design of concrete structures. General rules and rules for buildings.* **BSI 2004**.

2. **British Standards Institution**. *NA to BS EN 1992-1-1:2004. Eurocode 2: Design of concrete structures. General rules and rules for buildings.* **BSI 2005**.

Steel Design - EC3 and EC4

Steel Design to EC3 and EC4

Tekla Structural Designer designs steel members and composite members to a range of international codes. This reference guide specifically describes the design methods applied in the software when the BS EN 1993-1-1:2005 (Ref. 1) and BS EN 1994-1-1:2004 (Ref. 4) codes are selected.

Within the remainder of this handbook BS EN 1993-1-1:2005 and BS EN 1994-1-1:2004 are referred to as EC3 and EC4 respectively.

Unless explicitly noted otherwise, all clauses, figures and tables referred to are from EC3; apart from the Composite Beam section, within which references are to EC4 unless stated.

Basic Principles (EC3)

Definitions (EC3)

The following terms are relevant when using *Tekla Structural Designer* to design to the Eurocodes.

National Annex (NA)

Safety factors in the Eurocodes are recommended values and may be altered by the national annex of each member state.

Tekla Structural Designer currently has the following EC3 national annex options are available:

- EC3 Europe
- EC3 UK NA

- EC3 Ireland NA
- EC3 Ireland NA
- EC3 Malaysia NA
- EC3 Singapore NA

You can select the desired National Annex as appropriate, in which case the nationally determined parameters are automatically applied (see next section), or if you choose EC3 Europe, the Eurocode recommended values are applied.

Nationally Determined Parameters (NDP's)

NDP's are choices of values, classes or alternative methods contained in a National Annex that can be applied in place of the base Eurocode, EC3 Europe.

Partial Factors for Buildings

The partial factors γ_M for buildings as described in Clause 6.1(1) Note 2B should be applied to the various characteristic values of resistance as follows:

- resistance of cross-sections irrespective of class: γ_{M0}
- resistance of members to instability assessed by member checks: γ_{M1}
- resistance of cross-sections in tension to fracture: γ_{M2}

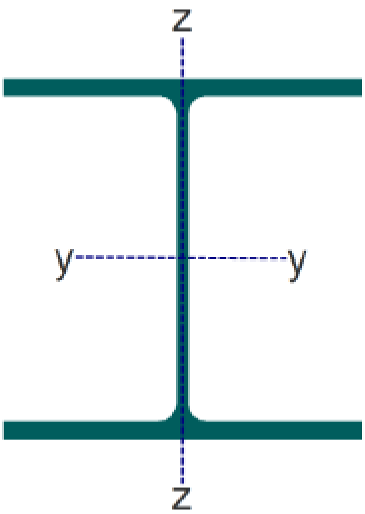
Depending on your choice of National Annex the above partial factors for buildings are set as follows:

Factor	EC3 Base value	UK	Ireland	Malaysia	Singapore
γ_{M0}	1.00	1.00	1.00	1.00	1.00
γ_{M1}	1.00	1.00	1.00	1.00	1.00
γ_{M2}	1.25	1.10*	1.25	1.20	1.10

NOTE - for connection design BS EN1991-1-8 - γ_{M2} = 1.25

Convention for member axes (EC3)

The sign convention for member axes when designing to Eurocodes is as shown below.

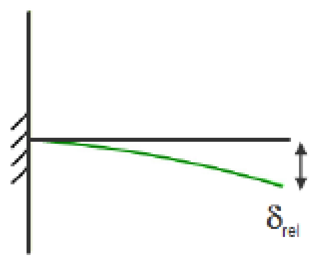


Section axes - (x is into the page along the centroidal axis of the member).

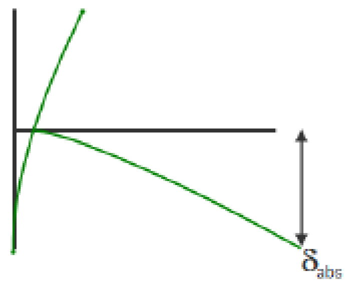
Deflection checks (EC3)

Relative and Absolute Deflections

Tekla Structural Designer calculates both ***relative*** and ***absolute*** deflections. Relative deflections measure the internal displacement occurring within the length of the member and take no account of the support settlements or rotations, whereas absolute deflections are concerned with deflection of the structure as a whole. The absolute deflections are the ones displayed in the structure deflection graphics. The difference between ***relative*** and ***absolute*** deflections is illustrated in the cantilever beam example below.



Relative Deflection



Absolute Deflection

Relative deflections are given in the member analysis results graphics and are the ones used in the member design.

Steel Beam Deflections

Deflections of steel beams in design are calculated from first order linear results since these are SLS deflections that are compared with standard limits such as span/360. This means that the effects of any non-linearity such as a continuous beam sitting on sinking supports i.e. non-linear springs are not taken into account in design. If these springs are linear this is not an issue.

Steel Beam Design to EC3

Design method (Beams: EC3)

Unless explicitly stated all calculations are in accordance with the relevant sections of EC3 (Ref. 1) and any associated National Annex.

A basic knowledge of the design methods for beams in accordance with the code is assumed.

Steel beam limitations and assumptions (Beams: EC3)

The following limitations apply:

- Continuous beams (more than one span) must be co-linear in the plane of the web within a small tolerance (sloping in elevation is allowed),
- Rolled doubly symmetric prismatic sections (i.e. I- and H-sections), doubly symmetric hollow sections (i.e. SHS, RHS and CHS), and channel sections are fully designed,
- Single angles, double angles and tees are designed, but certain checks are beyond scope, (see Angle and Tee Limitations)
- Plated beams are fully designed provided the section type is either “Plated Beam” or “Plated Column”. All other plated section types (“Rolled I Sections with Plates”, “Double Rolled I Sections” etc.) are analysed only but not designed,
- Fabsec beams (with or without openings) are excluded.

The following assumptions apply:

- All supports are considered to provide torsional restraint, that is lateral restraint to both flanges. This cannot be changed. It is assumed that a beam that is continuous through the web of a supporting beam or column together with its substantial moment resisting end plate connections is able to provide such restraint.
- If, at the support, the beam oversails the supporting beam or column then the detail is assumed to be such that the bottom flange of the beam is well connected to the supporting member and, as a minimum, has torsional stiffeners provided at the support.
- In the *Tekla Structural Designer* model, when not at supports, coincident restraints to both flanges are assumed when one or more members frame into the web of the beam

at a particular position and the cardinal point of the centre-line model of the beam lies in the web. Otherwise, only a top flange or bottom flange restraint is assumed. Should you judge the actual restraint provided by the in-coming members to be different from to what has been assumed, you have the flexibility to edit the restraints as required.

- Intermediate lateral restraints to the top or bottom flange are assumed to be capable of transferring the restraining forces back to an appropriate system of bracing or suitably rigid part of the structure.
- It is assumed that you will make a rational and “correct” choice for the effective lengths between restraints for both LTB and compression buckling. **The default value for the effective length factor of 1.0 may be neither correct nor safe**.

Ultimate Limit State - Strength (Beams: EC3)

The strength checks relate to a particular point on the member and are carried out at regular intervals along the member and at *“points of interest”*.

Classification (Beams: EC3)

General

The classification of the cross section is in accordance with EC3 Cl. 5.5 Table 5.2

A steel non-composite beam can be classified as:

- Plastic Class = 1
- Compact Class = 2
- Semi-compact Class = 3
- Slender Class = 4

Class 4 sections are unacceptable and are either failed in check mode or rejected in design mode.

Implementation of the below clauses is as follows:

- Classification is determined using 5.5.2 (6) and not 5.5.2 (7).
- 5.5.2 (9) is not implemented as clause (10) asks for the full classification to be used for buckling resistance.
- 5.5.2 (11) is not implemented.
- 5.5.2 (12) is not implemented.

The note at the end of Cl. 5.5.2 is not implemented. A brief study by CSC (UK) Ltd of UK rolled UBs and UCs showed that flange induced buckling in normal rolled sections is not a concern. No study was undertaken for plated sections.

Hollow sections

The classification rules for SHS and RHS relate to *"hot-finished hollow sections"* and *"cold-formed hollow sections"*.

Shear Capacity (Beams: EC3)

Major and minor axis shear

Checks are performed according to clause 6.2.6 (1) for the absolute value of shear force normal to each axis at the point under consideration.

The following points should be noted:

- No account is taken of fastener holes in the flange or web - see 6.2.6 (7)
- Shear is not combined with torsion and thus the resistance is not reduced as per 6.2.6 (8)

Web Shear buckling

Shear web buckling design applies to rolled and plated I/H sections only.

National Annex Dependency

Plates with unstiffened webs are checked for shear web buckling where:

$h_w/t_w > 72\ \varepsilon/\eta$

where

$\varepsilon \quad = \quad \sqrt{}(235 / f_w$

η is NA dependant and is defined in the table below:

National Annex	η	Applicable to
Eurocode value	1.20	Up to and including S460, else use η = 1.00
UK	1.00	All steel grades
Irish	1.00	All steel grades
Malaysia	1.00	All steel grades
Singapore	1.00	All steel grades

Contribution from flanges

When the flange resistance is not fully utilized in resisting the bending moment ($M_{Ed} < M_{f,Rd}$), the contribution from the flanges is taken as:

$V_{bf,Rd} = (b_f\ t^2{}_f\ f_{yf}) / (c\ \gamma_{M1}) * (1-(M_{Ed}/M_{f,Rd})^2)$

where

bf and tf are taken for the flange which provides the least axial resistance

b_f is not taken as larger than 15εtf on each side of the web

$M_{f,Rd}$ is the moment of resistance of the cross section consisting of the effective area of the flanges only

$c = a\ (0.25 + 1.6 b_f\ t_f^2\ f_{yf} / t\ h_w^2\ f_{yfw})$

a is the distance between stiffeners

As we are only designing for the case where no stiffeners are being used, a → ∞ therefore c → ∞ so $V_{bf,Rd} \rightarrow 0$.

Contribution from the web

The contribution from the web is taken as:

$V_{bw,Rd} = (x_w\ f_{yw}\ h_w\ t) / (\sqrt{3}\gamma_{M1})$

$x_w \leq \eta$

Design Resistance

The design resistance for shear is taken as:

$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \eta\ (f_{yw}\ h_w\ t) / (\sqrt{3}\gamma_{M1})$

Influence of Shear

According to 7.1 of EN 1993-1-5 provided $V_{Ed} \leq 0.5\ V_{bw,Rd}$ the design resistance to bending moment and axial force does not need to be reduced to allow for shear force.

In *Tekla Structural Designer* V_{Ed} is restricted to 0.5 $V_{bw,Rd}$, values above this are deemed beyond scope.

This restriction is only be applicable if $h_w/t_w > 72\ \varepsilon/\eta$

Assumptions

The following points should be noted:

- Non-rigid end post is a more conservative approach than a rigid end post.
- Physical support conditions can be taken as equivalent to "transverse stiffeners at supports only".
- It is assumed there is negligible contribution to the design shear force V_{Ed} from shear from torque
- All hole cut outs are small in accordance to section EN 1993-1-5:2006 2.3
- As the case being designed for is where no stiffeners are being used, a → ∞ therefore c → ∞ so $V_{bf,Rd} \rightarrow 0$.
- If a grade of steel is used other than S335, S355 and S460 η will be taken as 1.00 regardless of National Annex.

Moment Capacity (Beams: EC3)

Major and minor axis bending checks are performed in accordance with Section 6.2.5.

Major axis bending

For the low shear case the calculation uses equation 6.13 for class 1 and 2 cross sections and equation 6.14 for class 3 cross sections. In the high shear case equation 6.29 is used for class 1 and 2 cross sections and equation 6.14 for class 3 cross sections. Where the high shear condition applies, the moment capacity calculation is made less complicated by conservatively adopting a simplified shear area.

Minor axis bending

For the low shear case the calculation uses equation 6.13 for class 1 and 2 cross sections and equation 6.14 for class 3 cross sections. High shear in the minor axis is beyond the current program scope.

Fastener holes in the flange or web are not accounted for in the calculations.

Axial Capacity (Beams: EC3)

Axial Tension

checks are performed according to equation 6.5

Implementation of the below clauses is as follows:

- Cl 6.2.3 (3) - is not considered
- Cl 6.2.3 (4) - is not considered
- Cl 6.2.3 (5) - is not considered
- Eqn 6.7 is not considered for steel non-composite beams.

Axial Compression

Checks are performed according to equation 6.9.

Cross-section Capacity (Beams: EC3)

The cross-section capacity check covers the interaction of axial load and bending.

Class 1 and 2 cross sections

Equation 6.41 is applied. Note that in these calculations the combined effects of axial load and bending are assessed - clause 6.2.9 (4) is not considered

Also note that the current "reduced plastic moduli" approach that is used in the published tables is adopted and not the approximate method given in 6.2.9.1(5). The latter is less conservative than the current approach at low levels of 'n'.

Class 3 cross sections

Equation 6.42 is applied.

Axial and bending interaction checks are beyond the current program scope if coexistent high shear is present in the major axis.

Ultimate Limit State - Buckling (Beams: EC3)

Classification for buckling checks - For rolled I sections, RHS and SHS classification varies along the member length due to the section forces changing along the member length - for Combined Buckling, the worst classification of the whole member should be used. In theory it should be the worst classification in the “check length” considered for buckling. However, the “check lengths” for Lateral Torsional Buckling, minor axis Strut Buckling and major axis Strut Buckling can all be different. It is simpler and conservative therefore to use the worst classification in the entire member length.

Compression buckling (Beams: EC3)

Beams must be checked to ensure adequate resistance to buckling about both the major and minor axes and they must also be checked in the torsional mode over an associated buckling length. Since the axial force can vary throughout the beam and the buckling lengths in the two planes do not necessarily coincide, all buckling modes must be checked. There may be circumstances where it would not be safe to assume that the Combined Buckling check will always govern (see below).

Effective lengths

In all cases *Tekla Structural Designer* sets the default effective length to 1.0L, it does not attempt to adjust the effective length in any way. Different values can apply in the major and minor axis. It is **your** responsibility to adjust the value from 1.0 where you believe it to be justified.

It is assumed that you will make a rational and “correct” choice for the effective lengths between restraints. The default value for the effective length factor of 1.0L may be neither correct nor safe.

Coincident restraint points in the major and minor axis define the 'check length' for torsional and torsional flexural buckling (which also has an effective length factor but is assumed to be 1.0L and cannot be changed).

All intermediate major and minor restraints in a cantilever span are ignored.

Any major or minor Strut Buckling 'check length' can take the type 'Continuous' to indicate that it is continuously restrained over that length. There is no facility for specifying torsional or torsional flexural buckling 'check lengths' as 'Continuous'.

There is no guidance in EC3 on the values to be used for effective length factors for beam-columns.

There is no guidance in EC3 on the values to be used for effective length factors for beam-columns.

Compression resistance

The relevant buckling resistances are calculated from Eqn 6.47.

These consist of the flexural buckling resistance about both the major and minor axis i.e. $N_{b,y,Rd}$ and $N_{b,z,Rd}$ over the buckling lengths L_{yy} and L_{zz} and where required the buckling resistance in the torsional or flexural-torsional modes, $N_{b,x,Rd}$.

The elastic critical buckling load, N_{cr} for flexural buckling about major and minor axes is taken from standard texts. The elastic critical buckling loads for Torsional, $N_{cr.T}$ and for Torsional Flexural buckling, $N_{cr.TF}$ are taken from the NCCI “Critical axial load for torsional and torsional flexural buckling modes” available free to download at www.steel-ncci.co.uk.

All section types are checked for flexural buckling. It is only hollow sections that do not need to be checked for torsional and torsional-flexural buckling.

Lateral Torsional Buckling (Beams: EC3)

Lateral torsional buckling checks are required between supports, or LTB restraints on a flange which is in bending compression, for all lengths that are **not** continuously restrained.

Note that **coincident** LTB restraints (i.e. top & bottom flange) are the equivalent of a support and will define one end of a 'check length' for **both** flanges regardless of whether a particular flange is in compression or tension at the coincident restraint position. However, note also that in a cantilever all intermediate restraints are ignored.

Tekla Structural Designer allows you to 'switch off' LTB checks for either or both flanges by specifying that the entire length between start and end of the beam span is continuously restrained against lateral torsional buckling. If you use this option you must be able to provide justification that the beam is adequately restrained against lateral torsional buckling.

All intermediate LTB restraints in a cantilever span are ignored.

When the checks are required you can set the effective LTB length of each 'check length' by giving factors to apply to the physical length of the beam. Any individual 'check length' less than the full span length can be continuously restrained in which case no LTB check will be carried out for that 'check length' provided **all** segments of the 'check length' have been marked as Continuous. Each 'check length' which is not defined as being continuously restrained for its **whole length** is checked in accordance with clause 6.3.2.3.

The formula for elastic critical buckling moment, M_{cr} is taken from standard texts. The moment factor C_1 that is part of the standard formula has been derived analytically.

LTB does not need to be checked for the following sections,

- circular and square hollow sections,
- equal and unequal flanged I/H sections loaded in the minor axis only.

Effective lengths

The value of effective length factor is entirely the choice of the engineer. The default value is 1.0. There is no specific factor for destabilizing loads - so you will have to adjust the 'normal' effective length factor to allow for such effects.

Combined buckling (Beams: EC3)

Combined buckling in *Tekla Structural Designer* is limited to doubly symmetric sections (I, H, CHS, SHS, RHS). In the context of Combined Buckling beams are assumed to be dominated by moment with axial force.

Restraints are treated as described previously and summarized as follows:

- Each span is assumed to be fully supported at its ends (i.e LTB, y-y and z-z restraint) - this cannot be changed.
- Tension flange LTB restraints are ignored unless they are coincident (see next point).
- Coincident top and bottom flange restraints are considered as 'torsional' restraints i.e. as good as the supports.
- All intermediate LTB and strut restraints in a cantilever span are ignored.

For each span of the beam, the design process is driven from the standpoint of the individual LTB lengths i.e. the LTB lengths and the y-y lengths that are associated with each LTB length and the z-z lengths associated with the y-y length. Thus a 'hierarchy' is formed - see the ***"Design Control"*** section below for details. Both Equ. 6.61 and Equ. 6.62 are evaluated recognizing that the Combined Buckling check is carried out for both the top flange and the bottom flange.

Effective lengths

In all cases *Tekla Structural Designer* sets the default effective length to 1.0L, it does not attempt to adjust the effective length in any way. Different values can apply in the major and minor axis. It is **your** responsibility to adjust the value from 1.0 where you believe it to be justified.

It is assumed that you will make a rational and "correct" choice for the effective lengths between restraints. The default value for the effective length factor of 1.0L may be neither correct nor safe.

Combined buckling resistance

Equations 6.61 and 6.62 are used to determine the combined buckling resistance.

With regard to these equations the following should be noted:

- The "k" factors used in these equations are determined from Annex B only, and reported as follows:
- k_{yy} is reported as components k'_{yy} and C_{my} where k'_{yy} is simply the Annex B term for k_{yy} with C_{my} **excluded**
- k_{yz} is reported as k'_{yz}, a multiple of k'_{zz} (see below)
- k_{zy} is reported as k'_{zy}, a multiple of k'_{yy} (see above), but only for members not susceptible to torsional deformations (i.e. SHS and CHS sections at all times, and I or H sections which have **both** flanges continuously restrained for LTB). For members which are susceptible to torsional deformations k_{zy} is reported per Table B.2 (i.e. with C_{mLT} **included**)
- $k_{zy,LT1}$ is a factor reported for columns only and is the Table B.2 term for k_{zy} with C_{mLT} set to 1.0
- k_{zz} is reported as components k'_{zz} and C_{mz} where k'_{zz} is simply the Annex B term for k_{zz} with C_{mz} excluded.
- The note to Table B.3 that C_m should be limited to 0.9 is not applied.

Equations 6.61 and 6.62 are limited to doubly symmetric sections and do not consider torsional or torsional flexural buckling. **Should either of these buckling modes govern the compression buckling check, you should consider very carefully whether the calculations provided by Tekla Structural Designer for combined buckling can be considered valid.**

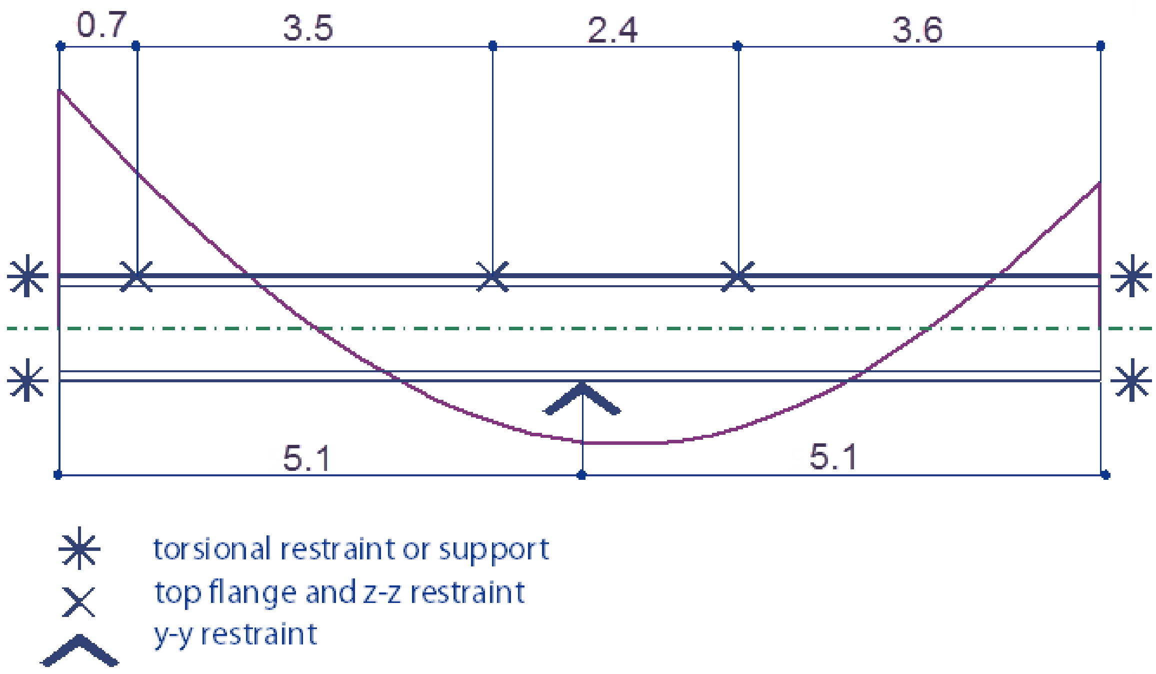
Design Control (Beams: EC3)

Principles

There are multiple check lengths to deal with (LTB, y-y buckling and z-z buckling) all of which can be contained within or overlapped by their associated lengths. Consequently, a 'hierarchy' of checks is defined. In the approach taken the LTB segment length is taken as the driver and the other lengths whether overlapping or contained by this segment are mapped to it.

Design Example

The following example illustrates how the checks are applied to I- and H-sections with equal flanges..



The beam (span) is 10.2 m long and has torsional restraints at each end. The top flange is restrained out-of-plane at 0.7m, 4.2m and 6.6 m – these provide restraint to the top flange for LTB and to the beam as a whole for out-of-plane strut buckling. The bottom flange has one restraint at mid-span and this restrains the bottom flange for LTB and the beam as a whole for in-plane strut buckling. (This is probably difficult to achieve in practice but is useful for illustration purposes.)

Note that the top flange LTB restraints and z-z restraints are coincident in this example but will not always be coincident.

Tekla Structural Designer identifies the following lengths and checks. (in this example all the effective length factors are assumed to be 1.0 for simplicity.)

LTB Segment	**Equation**	**In-plane strut segment**	**Out-of-plane strut segment**
length (m)		**length (m)**	**length (m)**
Top Flange 0 – 4.2 (first restraint ignored since top flange is in tension at this point)	6.61	0 – 5.1	0 – 0.7
	6.62	0 – 5.1	0 – 0.7
	6.61	0 – 5.1	0.7- 4.2
	6.62	0 – 5.1	0.7- 4.2
Top Flange	6.61	0 – 5.1	4.2 – 6.6

4.2 – 6.6	6.62	0 – 5.1	4.2 - 6.6
	6.61	5.1 - 10.2	4.2 - 6.6
	6.62	5.1 - 10.2	4.2 - 6.6
Top Flange 6.6 - 10.2	6.61	5.1 - 10.2	6.6 - 10.2
	6.62	5.1 -10.2	6.6 - 10.2
Bottom Flange 0 – 10.2	6.61	0 – 5.1	0 – 0.7
	6.62	0 – 5.1	0 – 0.7
	6.61	0 – 5.1	0.7- 4.2
	6.62	0 – 5.1	0.7- 4.2
	6.61	0 – 5.1	4.2 - 6.6
	6.62	0 – 5.1	4.2 - 6.6
	6.61	5.1 - 10.2	4.2 - 6.6
	6.62	5.1 - 10.2	4.2 - 6.6
	6.61	5.1 - 10.2	6.6 - 10.2
	6.62	5.1 - 10.2	6.6 - 10.2

Torsion (Beams: EC3)

Torsion design is carried out on request according to SCI P385, but only for single span, pin ended steel and cold formed beams with open and closed section types.

Open sections (I- symmetric rolled)

A torsion design and an angle rotation check can be carried out for applied torsion forces only.

The following should be noted with regard to the torsion design:

- Axial force is not taken into account
- It is assumed that load is applied at the shear centre. The effect of stabilising/destabilizing loads is not considered.

Closed sections (HSS only)

An angle of rotation check can be carried out for applied forces only.

Angle of rotation check

The angle of rotation check is optionally carried out based on the applied torsion loading only.

The check is applied by selecting “Apply rotation limit” (located in the steel beam properties under the Torsion heading). The default limit is also set in the steel beam properties as 2° but can be adjusted to suit.

Web Openings (Beams: EC3)

Circular Openings as an Equivalent Rectangle

Each circular opening is replaced by equivalent rectangular opening, the dimensions of this equivalent rectangle for use in all subsequent calculations are:

d_o'= 0.9*opening diameter

l_o = 0.45*opening diameter

Properties of Tee Sections

When web openings have been added, the properties of the tee sections above and below each opening are calculated in accordance with Section 3.3.1 of SCI P355(Ref. 8) and Appendix B of the joint CIRIA/SCI Publication P068(Ref. 9). The bending moment resistance is calculated separately for each of the four corners of each opening.

Design

The following calculations are performed where required for web openings:

- Axial resistance of tee sections
- Classification of section at opening
- Vertical shear resistance
- Vierendeel bending resistance
- Web post horizontal shear resistance
- Web post bending resistance
- Web post buckling resistance
- Lateral torsional buckling
- Deflections

Deflections

The deflection of a beam with web openings will be greater than that of the same beam without openings. This is due to two effects,

- the reduction in the beam inertia at the positions of openings due to primary bending of the beam,
- the local deformations at the openings due to Vierendeel effects. This has two components - that due to shear deformation and that due to local bending of the upper and lower tee sections at the opening.

The primary bending deflection is established by 'discretising' the member and using a numerical integration technique based on 'Engineer's Bending Theory' - M/I = E/R = σ/y. In this way the discrete elements that incorporate all or part of an opening will contribute more to the total deflection.

The component of deflection due to the local deformations around the opening is established using a similar process to that used for cellular beams which is in turn based on the method for castellated beams given in the SCI publication, "Design of castellated beams. For use with BS 5950 and BS 449".

The method works by applying a 'unit point load' at the position where the deflection is required and using a 'virtual work technique to estimate the deflection at that position.

For each opening, the deflection due to shear deformation, δ_s, and that due to local bending, δ_{bt}, is calculated for the upper and lower tee sections at the opening. These are summed for all openings and added to the result at the desired position from the numerical integration of primary bending deflection.

Note that in the original source document on castellated sections, there are two additional components to the deflection. These are due to bending and shear deformation of the web post. For castellated beams and cellular beams where the openings are very close together these effects are important and can be significant. For normal beams the openings are likely to be placed a reasonable distance apart. Thus in many cases these two effects will not be significant. They are not calculated for such beams but in the event that the openings are placed close together a warning is given.

Composite Beam Design to EC4

Design method (Composite beams: EC4)

The construction stage calculations are performed in accordance with the relevant sections of EC3 (Ref. 1) and the associated UK (Ref. 2) or Irish (Ref. 3) National Annex.

The composite stage design adopts a limit state approach consistent with the design parameters for simple and continuous composite beams as specified in EC4 (Ref. 4) and the associated UK (Ref. 5) or Irish National Annex.

Unless explicitly noted otherwise, all clauses, figures and tables referred to are from EC4.

A basic knowledge of EC3 and the design methods for composite beams in EC4 is assumed.

Overview (Composite beams: EC4)

Construction stage design checks (Composite beams: EC4)

When you design or check a beam for the construction stage (the beam is acting alone before composite action is achieved) the following conditions are examined in accordance with EC3:

- section classification (EC3 Table 5.2),
- major axis shear capacity (EC3 Clause 6.2.6 (1)),
- web shear buckling (EC3 Clause 6.2.6 (6)),
- moment capacity:
  - EC3 Equation 6.13 for the low shear condition,
  - EC3 Equation 6.29 for the high shear condition,
- lateral torsional buckling resistance (EC3 Clause 6.3.2.3),

> *This condition is only checked in those cases where the profile decking does not provide adequate restraint to the beam.*

- construction stage total load deflection check.

Composite stage design checks

When you design or check a beam for the composite stage (the beam and concrete act together, with shear interaction being achieved by appropriate shear connectors) the following Ultimate Limit State and Serviceability Limit State conditions are examined in accordance with EC4, unless specifically noted otherwise.

Ultimate Limit State Checks

- section classification - the classification system defined in EC3 Clause 5.5.2 applies to cross-sections of composite beams,
- vertical shear capacity in accordance with EC3 Clause 6.2.6,
- longitudinal shear capacity allowing for the profiled metal decking, transverse reinforcement and other reinforcement which has been defined,

- number of shear connectors required (EC4 Clause 6.6.1.3 (5)) between the point of maximum moment and the end of the beam, or from and between the positions of significant point loads,
- moment capacity,
- web openings.

Serviceability Limit State Checks

- service stresses - although there is no requirement to check these in EC4 for buildings (EC4 Clause 7.2.2), concrete and steel top/bottom flange stresses are calculated but only reported if the stress limit is exceeded.
- deflections,
    - self-weight,
        - SLAB loadcase,
        - dead load,
        - imposed load,
        - total deflections,
- natural frequency check.

Profiled metal decking (Composite beams: EC4)

You may define the profiled metal decking to span at any angle between 0° (parallel) and 90° (perpendicular) to the direction of span of the steel beam. You can also specify the attachment of the decking for parallel, perpendicular and angled conditions.

Where you specify that the direction of span of the profiled metal decking to that of the steel beam is >=45°, then *Tekla Structural Designer* assumes it is not necessary to check the beam for lateral torsional buckling during construction stage.

Where you specify that the direction of span of the profiled metal decking to that of the steel beam is < 45°, then you are given the opportunity to check the steel beam for lateral torsional buckling at the construction stage.

This check is not mandatory in all instances. For a particular profile, gauge and fixing condition etc. you might be able to prove that the profiled metal decking is able to provide a sufficient restraining action to the steel beam until the concrete hardens. If this is so, then you can specify that the whole beam (or a part of it) is continuously restrained. Where you request to check the beam for lateral torsional buckling during construction then this is carried out in accordance with the requirements of EC3.

Where you specify that the direction of span of the profiled metal decking and that of the steel beam are parallel, you again have the same opportunity to either check the steel beam for lateral torsional buckling at the construction stage, or to set it as continuously restrained.

Precast concrete planks (Composite beams: EC4)

The design of composite beams with precast concrete planks is carried out in accordance with the guidance given in SCI P401. The design basis in P401 is, in general, in accordance with Eurocode 4, supplemented by NCCI derived test data where applicable.

As the implications of applying NCCI PN002 or SCI P405 to composite beams with PC planks have not being considered in the first release, only pure EC design will be carried out regardless of whether **apply NCCI PN002** is selected or not.

Where a choice has been made the condition of the most common application has been taken: shop welded, hollow core unit with partial interaction.

Construction stage design (Composite beams: EC4)

*A*ll checks are performed for this condition in accordance with EC3.

Section classification (Composite beams: EC4)

Cross-section classification is determined using EC3 Table 5.2.

At construction stage the classification of the section must be Class 1, Class 2 or Class 3.

Sections which are classified as Class 4 are beyond scope.

Clause 5.5.2 (6) is implemented, not the alternative 5.5.2 (7).
Clause 5.5.2 (11) is not implemented
Clause 5.5.2 (12) is not implemented.

Member strength checks (Composite beams: EC4)

Member strength checks are performed at the point of maximum moment, the point of maximum shear, the position of application of each point load, and at all other “points of interest” along the beam.

Shear capacity

Shear capacity is determined in accordance with EC3 Clause 6.2.6 (1). Where the applied shear force exceeds 50% of the capacity of the section, the high shear condition applies to the bending moment capacity checks (see below).

The following points should be noted:

- No account is taken of fastener holes in the flange or web - see EC3 6.2.6 (7)
- Shear is not combined with torsion and thus the resistance is not reduced as per EC3 6.2.6(8)

Web Shear buckling

See: Steel Beam Design to EC3 - Web Shear buckling

Bending moment capacity

For low shear this is calculated to EC3 Equation 6.13. In the high shear case Equation 6.29 is used. Where the high shear condition applies, the moment capacity calculation is made less complicated by conservatively adopting a simplified shear area.

Lateral torsional buckling checks (Composite beams: EC4)

You can switch off lateral torsional buckling checks by specifying that the entire length between the supports is continuously restrained.

If you use this option you must be able to provide justification that the beam is adequately restrained against lateral torsional buckling during construction.

When the checks are required you can position restraints at any point within the length of the main beam and can set the effective length of each sub-beam (the portion of the beam between one restraint and the next) either by giving factors to apply to the physical length of the beam, or by entering the effective length that you want to use. Each sub-beam which is not defined as being continuously restrained is checked in accordance with EC3 Clause 6.3.2.3.

Deflection checks (Composite beams: EC4)

Tekla Structural Designer calculates ***relative*** deflections. (see:)

The following deflections are calculated for the loads specified in the construction stage load combination:

- the dead load deflections i.e. those due to the beam self weight, the Slab Wet loads and any other included dead loads,
- the imposed load deflections i.e. those due to construction live loads,
- the total load deflection i.e. the sum of the previous items.

The loads are taken as acting on the steel beam alone.

The “Service Factor” (default 1.0), specified against each load case in the construction combination is applied when calculating the above deflections.

If requested by the user, the total load deflection is compared with either a span-over limit or an absolute value The initial default limit is span/200.

Adjustment to deflections. If web openings have been defined, the calculated deflections are adjusted accordingly. See:

Composite stage design (Composite beams: EC4)

Tekla Structural Designer performs all checks for the composite stage condition in accordance with EC4 unless specifically noted otherwise.

Equivalent steel section - Ultimate limit state (ULS) (Composite beams: EC4)

An equivalent steel section is determined for use in the composite stage calculations by removing the root radii whilst maintaining the full area of the section. This approach reduces the number of change points in the calculations while maintaining optimum section properties.

Section classification (ULS) (Composite beams: EC4)

Tekla Structural Designer classifies the section in accordance with the requirements of EC3, 5.5.2 except where specifically modified by those of EC4.

A composite section is classified according to the highest (least favourable) class of its steel elements in compression. The compression flange and the web are therefore both classified and the least favourable is taken as that for the whole section.

Flanges of any class that are fully attached to a concrete flange are assumed to be Class 1. The requirements for maximum stud spacing according to Clause 6.6.5.5 (2) are checked and you are warned if these are not satisfied.

There are a small number of sections which fail to meet Class 2 at the composite stage. Although EC4 covers the design of such members they are not allowed in this release of *Tekla Structural Designer*.

Member strength checks (ULS) (Composite beams: EC4)

It is assumed that there are no loads or support conditions that require the web to be checked for transverse force. (Clause 6.5)

Member strength checks are performed at the point of maximum moment, the point of maximum shear, the position of application of each point load, and at all other points of interest along the beam.

Shear Capacity (Vertical)

The resistance to vertical shear, V_{Rd}, is taken as the resistance of the structural steel section, $V_{pl,a,Rd}$. The contribution of the concrete slab is neglected in this calculation.

The shear check is performed in accordance with EC3, 6.2.6.

Moment Capacity

For full shear connection the plastic resistance moment is determined in accordance with Clause 6.2.1.2. For the partial shear connection Clause 6.2.1.3 is adopted.

In these calculations the steel section is idealised to one without a root radius so that the position of the plastic neutral axis of the composite section can be determined correctly as it moves from the flange into the web.

Where the vertical shear force, V_{Ed}, exceeds half the shear resistance, V_{Rd}, a (1- ρ) factor is applied to reduce the design strength of the web - as per Clause 6.2.2.4.

Shear Capacity (Longitudinal)

The design condition to be checked is: $v_{Ed} \leq v_{Rd}$

where

v_{Ed} = design longitudinal shear stress

v_{Rd} = design longitudinal shear strength (resistance)

v_{Ed} is evaluated at all relevant locations along the beam and the maximum value adopted.

v_{Rd} is evaluated taking account of the deck continuity, its orientation and the provided reinforcement.

This approach uses the “truss analogy” from EC2. (See Figure 6.7 of EC2).

In these calculations, two planes are assumed for an internal beam, and one for an edge beam. Only the concrete above the deck is used in the calculations.

The values of v_{Rd} based on the concrete “strut” and the reinforcement “tie” are calculated. The final value of v_{Rd} adopted is then taken as the minimum of these two values.

The angle of the strut is minimised to minimise the required amount of reinforcement - this angle must lie between 26.5 and 45 degrees.

In the calculations of v_{Rd} the areas used for the reinforcement are as shown in the following table.

Decking angle	Reinforcement type	Area used
perpendicular	transverse	that of the single bars defined or for mesh the area of the main wires[A]
	other	that of the single bars defined or for mesh the area of the main wires[(a)]
parallel	transverse	that of the single bars defined or for mesh the area of the main wires[(a)]
	other	single bars have no contribution, for mesh the area of the minor

		wires[B]

[A]These are the bars that are referred to as longitudinal wires in BS 4483: 1998 Table 1.
[B]These are the bars that are referred to as transverse wires in BS 4483: 1998 Table 1.

If the decking spans at some intermediate angle (θ_r) between these two extremes then the program calculates:

- the longitudinal shear resistance as if the sheeting were perpendicular, $v_{Rd,perp}$,
- the longitudinal shear resistance as if the sheeting were parallel, $v_{Rd,par}$,
- then the modified longitudinal shear resistance is calculated from these using the relationship, $v_{Rd,perp}\sin^2(\theta_r) + v_{Rd,par}\cos^2(\theta_r)$.

Minimum area of transverse reinforcement (Composite beams: EC4)

The minimum area of transverse reinforcement is checked in accordance with Clause 6.6.6.3.

Shear connectors (ULS) (Composite beams: EC4)

Dimensional Requirements

Various limitations on the use of studs are given in the code.

The following conditions in particular are drawn to your attention:

Parameter	Rule	Clause/Comment
Spacing	Ductile connectors may be spaced uniformly over length between critical cross-sections if: - All critical cross-sections are Class 1 or 2 - The degree of shear connection, η is within the range given by 6.6.1.2 and - the plastic resistance moment of the composite section does not exceed 2.5 times the plastic resistance moment of the steel member alone.	6.6.1.3(3) - not checked
Edge Distance	e_D >= 20 mm	6.6.5.6(2) - not checked
	$e_D \leq 9*t_f*\text{sqrt}(235/f_y)$	6.6.5.5(2) - applies if bare steel beam flange is Class 3 or 4 - not checked

Location	If can't be located in centre of trough, place alternately either side of the trough throughout span	6.6.5.8(3) - not checked
Cover	The value from EC2 Table 4.4 less 5mm, or 20mm whichever is the greater.	6.6.5.2(2) - not checked

The program does not check that the calculated stud layout can be fitted in the rib of the deck.

Design resistance of the shear connectors

For ribs parallel to the beam the design resistance is determined in accordance with Clause 6.6.4.1. The reduction factor, k_l is obtained from Equation 6.22.

For ribs perpendicular to the beam, Clause 6.6.4.2 is adopted. The reduction factor, k_t is obtained from Equation 6.23.

The factor k_t should not be taken greater than the appropriate value of $k_{t,max}$ from the following table;

No of Stud Connectors per rib	**Thickness of sheet, t mm**	**Studs with d $\leq$ 20 mm and welded through profiled steel sheeting, $k_{t,max}$**	**Profiled sheeting with holes and studs with d=19 or 22 mm, $k_{t,max}$**
$n_r = 1$	≤ 1.0	0.85	0.75
	> 1.0	1.00	0.75
$n_r = 2$	≤ 1.0	0.70	0.60
	> 1.0	0.80	0.60

Only the first column of values of $k_{t.max}$ is used from the above table since the technique of leaving holes in the deck so that studs can be welded directly to the beam is not used.

For cases where the ribs run at an angle, θ_r the reduction factor is calculated as:

$k_t*\sin^2\theta_r + k_l*\cos^2\theta_r$

Stud optimization is a useful facility since there is often some over conservatism in a design due to the discrete changes in the size of the section.

If you choose the option to optimise the shear studs, then *Tekla Structural Designer* will progressively reduce the number of studs either until the minimum number of studs to resist the applied moment is found, until the minimum allowable interaction ratio is reached or until the minimum spacing requirements are reached. This results in partial shear connection.

The program can also automatically layout groups of 1 or 2 studs with constraints that you specify.

The degree of shear connection is checked at the point of maximum bending moment or the position of a point load if at that position the maximum utilisation ratio occurs.

During the selection process, in auto design mode point load positions are taken to be "significant" (i.e. considered as positions at which the maximum utilisation could occur) if they provide more than 10% of the total shear on the beam. For the final configuration and for check mode all point load positions are checked.

To determine if the degree of shear connection is acceptable *Tekla Structural Designer* applies the following rules:

- If the degree of shear connection at the point of maximum moment is less than the minimum permissible shear connection, then this generates a *FAIL* status,
- If the point of maximum utilisation ratio occurs at a point that is not the maximum moment position and the degree of shear connection is less than the minimum permissible shear connection, then this generates a *WARNING* status,
- If the degree of shear connection at any other point load is less than the minimum permissible shear connection, then this does not affect the status in any way.

Lateral torsional buckling checks (ULS) (Composite beams: EC4)

The concrete slab is assumed to be laterally stable and hence there is no requirement to check lateral torsional buckling at the composite stage. (Clause 6.4.1).

Section properties - serviceability limit state (SLS) (Composite beams: EC4)

A value of the short term elastic (secant) modulus, E_{cm} is defaulted in *Tekla Structural Designer* for the selected grade of concrete. The long term elastic modulus is determined by dividing the short term value by a user defined factor - default 3.0. The elastic section properties of the composite section are then calculated using these values as appropriate (see the table below).

This approach is used as a substitute for the approach given in EC4 Equation 5.6 in which a knowledge of the creep coefficient, ϕ_t, and the creep multiplier, ψ_L is required. It is envisaged that you will make use of EN 1992-1-1(Ref. 6) when establishing the appropriate value for the factor.

EN 1994-1-1, Clause 7.3.1.(8) states that the effect on deflection due to curvature imposed by restrained drying shrinkage may be neglected when the ratio of the span to the overall beam depth is not greater than 20. This relates to normal weight concrete. Tekla Structural Designer makes no specific allowance for shrinkage curvature but does provide you with a Warning when the span to overall depth exceeds 20 irrespective of whether the concrete is normal weight or lightweight. Where you consider allowance should be made, it is suggested that you include this as part of the 'factor' described above.

Tekla Structural Designer calculates the deflection for the beam based on the following properties:

Loadcase Type	Properties used
self-weight	bare beam
Slab Dry	bare beam
Dead	composite properties calculated using the long term elastic modulus
Live	composite properties calculated using the effective elastic modulus appropriate to the long term load percentage for each load. The deflections for all loads in the loadcase are calculated using the principle of superposition.
Wind	composite properties calculated using the short term elastic modulus
Total loads	these are calculated from the individual loadcase loads as detailed above again using the principle of superposition

Deflection checks (SLS) (Composite beams: EC4)

Tekla Structural Designer calculates ***relative*** deflections. (see:)

The composite stage deflections are calculated in one of two ways depending upon the previous and expected future load history:

- the deflections due to all loads in the Slab Dry loadcase and the self-weight of the beam are calculated based on the inertia of the steel beam alone (these deflections are not modified for the effects of partial interaction).

It is the Slab Dry deflection alone which is compared with the limit, if any, specified for the Slab loadcase deflection.

- the deflections for all loads in the other loadcases of the Design Combination will be based on the inertia of the composite section allowing for the proportions of the particular load that are long or short term (see above). When necessary these will be modified to include the effects of partial interaction.

Tekla Structural Designer reports the deflection due to imposed loads alone (allowing for long and short term effects). It also reports the deflection for the SLAB loadcase, as this is useful for pre-cambering the beam. The beam Self-weight, Dead and Total deflections are also given to allow you to be sure that no component of the deflection is excessive.

Adjustment to deflections. If web openings have been defined, the calculated deflections are adjusted accordingly. See:

Stress checks (SLS) (Composite beams: EC4)

There is no requirement to check service stresses in EC4 for buildings (Clause 7.2.2). However, since the deflection calculations are based on elastic analysis then at service loads it is logical to ensure that there is no plasticity at this load level.

Tekla Structural Designer calculates the worst stresses in the extreme fibres of the steel and the concrete at serviceability limit state for each load taking into account the proportion which is long term and that which is short term. These stresses are then summed algebraically. Factors of 1.00 are used on each loadcase in the design combination (you cannot amend these). The stress checks assume that full interaction exists between the steel and the concrete at serviceability state. The stresses are not reported unless the stress limit is exceeded, in which case a warning message is displayed.

Natural frequency checks (SLS) (Composite beams: EC4)

Tekla Structural Designer calculates the approximate natural frequency of the beam based on the simplified formula published in the *Design Guide on the vibration of floors* (Ref. 7) which states that:

Natural frequency = 18 / √ δ

where δ is the maximum static instantaneous deflection that would occur under a load equivalent to the effects of self-weight, dead loading and 10% of the characteristic imposed loading, based upon the composite inertia (using the short term elastic modulus) but not modified for the effects of partial interaction.

Cracking of concrete (SLS) (Composite beams: EC4)

In Clause 7.4.1(4) simply supported beams in unpropped construction require a minimum amount of longitudinal reinforcement over an internal support. This is not checked by *Tekla Structural Designer* as it is considered a detailing requirement.

Web Openings (Composite beams: EC4)

Circular Openings as an Equivalent Rectangle

Each circular opening is replaced by equivalent rectangular opening, the dimensions of this equivalent rectangle for use in all subsequent calculations are:

d_o'= 0.9*opening diameter

l_o = 0.45*opening diameter

Properties of Tee Sections

When web openings have been added, the properties of the tee sections above and below each opening are calculated in accordance with Section 3.3.1 of SCI P355(Ref. 8) and Appendix B of the joint CIRIA/SCI Publication P068(Ref. 9). The bending moment resistance is calculated separately for each of the four corners of each opening.

Design at Construction stage

The following calculations are performed where required for web openings:

- Axial resistance of tee sections
- Classification of section at opening
- Vertical shear resistance
- Vierendeel bending resistance
- Web post horizontal shear resistance
- Web post bending resistance
- Web post buckling resistance
- Lateral torsional buckling
- Deflections

Design at Composite stage

The following calculations are performed where required for web openings:

- Axial resistance of concrete flange
- Vertical shear resistance of the concrete flange
- Global bending action - axial load resistance
- Classification of section at opening
- Vertical shear resistance
- Moment transferred by local composite action
- Vierendeel bending resistance
- Web post horizontal shear resistance
- Web post bending resistance
- Web post buckling resistance
- Deflections

Deflections

The deflection of a beam with web openings will be greater than that of the same beam without openings. This is due to two effects,

- the reduction in the beam inertia at the positions of openings due to primary bending of the beam,
- the local deformations at the openings due to Vierendeel effects. This has two components - that due to shear deformation and that due to local bending of the upper and lower tee sections at the opening.

The primary bending deflection is established by 'discretising' the member and using a numerical integration technique based on 'Engineer's Bending Theory' - M/I = E/R = σ/y. In this way the discrete elements that incorporate all or part of an opening will contribute more to the total deflection.

The component of deflection due to the local deformations around the opening is established using a similar process to that used for cellular beams which is in turn based on the method for castellated beams given in the SCI publication, “Design of castellated beams. For use with BS 5950 and BS 449".

The method works by applying a 'unit point load' at the position where the deflection is required and using a 'virtual work technique to estimate the deflection at that position.

For each opening, the deflection due to shear deformation, δ_s, and that due to local bending, δ_{bt}, is calculated for the upper and lower tee sections at the opening. These are summed for all openings and added to the result at the desired position from the numerical integration of primary bending deflection.

Note that in the original source document on castellated sections, there are two additional components to the deflection. These are due to bending and shear deformation of the web post. For castellated beams and cellular beams where the openings are very close together these effects are important and can be significant. For normal beams the openings are likely to be placed a reasonable distance apart. Thus in many cases these two effects will not be significant. They are not calculated for such beams but in the event that the openings are placed close together a warning is given.

Precast Concrete Planks:

The effect of web openings on composite beams with PC planks is not within the scope of SCI P401. Web openings can be modelled but are ignored in both design at Construction stage and design at Composite stage when a PC plank is used. Design will be carried out treating the steel beam as one with no web openings.

Application of NCCI PN002 to Partial Shear Connection (Composite beams: EC4)

An **Apply NCCI PN002** check box is available on the *Stud strength* page of the *Beam Properties*. When this option is selected *Tekla Structural Designer* calculates partial shear limits described in PN002 for edge beams and SCI P405 for internal beams.

It should be noted that to obtain the benefits of this NCCI,

- for all deck types and orientation the design live load (γ_q q_k) is limited to 9 kN/m^2
- for all deck types and orientation the beam should be “unpropped” at the construction stage (this is a general assumption in *Tekla Structural Designer* for all composite beams).
- for perpendicular trapezoidal decks the studs should be placed on the “favourable” side or in the central position.
- for perpendicular trapezoidal decks the reinforcement is assumed to be above the head of the stud. Consequently, a reduction is made to the stud resistance in accordance with NCCI PN001.
- for limits of maximum longitudinal stud spacing the relevant NCCI must be satisfied.
- for slab the nominal total depth must not exceed 180mm (depth of concrete over the decking must not exceed 100mm)
- for all deck profiles the nominal height (to shoulder) must not exceed 80mm (applies to SCI P405 only)

It is the user's responsibility to ensure compliance with the above since the program makes no check on these items.

For perpendicular trapezoidal decks the reduction in stud resistance to which point 4 above refers, will be conservative if the reinforcement is placed in a more favourable (lower) position. Even though the NCCI is relevant to the UK this option is also available for all EC head-codes in *Tekla Structural Designer*.

More information is given in the PN001, PN002 and SCI P405 on www.steel-ncci.co.uk and on http://www.steelbiz.org/

Steel Column Design to EC3

Design method (Columns: EC3)

Unless explicitly stated all steel column calculations in *Tekla Structural Designer* are in accordance with the relevant sections of EC3 (Ref. 1) and the associated National Annex.

A full range of strength, buckling and serviceability checks are carried out.

A sway assessment is also performed. This can optionally be de-activated for those columns for which it would be inappropriate, by unchecking the Alpha Crit Check box on the Column Properties dialog.

Simple Columns (Columns: EC3)

A general column could be designated as a “simple column” to indicate that it does not have any applied loading in its length. Simplified design rules exist for such columns as they are only subject to axial forces and moments due to eccentricity of beam reactions, (moments due to frame action or due to member loading are assumed not to occur).

The simple column design rules have not yet been implemented in Tekla Structural Designer: such columns are thus classed as “beyond scope” when they are designed.

Ultimate Limit State Strength (Columns: EC3)

Strength checks relate to a particular point on the member and are carried out at 5^{th} points and “points of interest”, (i.e. positions such as maximum moment, maximum axial etc.)

Classification (Columns: EC3)

The classification of the cross section is in accordance with Table 5.2. General columns can be classified as:

- Plastic Class = 1
- Compact Class = 2
- Semi-compact Class = 3
- Slender Class = 4

Class 4 sections are not allowed.

Implementation of the below clauses is as follows:

- Classification is determined using 5.5.2 (6) and not 5.5.2 (7).
- 5.5.2 (9) is not implemented as clause (10) asks for the full classification to be used for buckling resistance.
- 5.5.2 (11) is not implemented.
- 5.5.2 (12) is not implemented. A brief study of UK rolled UBs and UCs showed that flange induced buckling in normal rolled sections is not a concern.

Axial Capacity (Columns: EC3)

The axial tension and compression capacity checks are performed according to Clause 6.2.3 and Clause 6.2.4 respectively.

The following points should be noted:

- Cl 6.2.3 (3) - is not considered
- Cl 6.2.3 (4) - is not considered

- Cl 6.2.3 (5) - is not considered

Shear Capacity (Columns: EC3)

The shear check is performed at the point under consideration according to Clause 6.2.6(1):

- for the absolute value of shear force normal to the y-y axis, $V_{y,Ed}$, and
- for the absolute value of shear force normal to the z-z axis, $V_{z,Ed}$

The following points should be noted:

- No account is taken of fastener holes in the flange or web - see 6.2.6 (7)
- Shear is not combined with torsion and thus the resistance is not reduced as per 6.2.6 (8)

Shear buckling

When the web slenderness exceeds 72ε shear buckling can occur in rolled sections. *Tekla Structural Designer* designs for shear web buckling with accordance to EN 1993-1-5:2006. The following should however be noted:

- The approach to design assumes a non-rigid end post, this is more conservative than the design that takes the approach assuming a rigid end post.
- Physical support conditions have been assumed to be equivalent to “transverse stiffeners at supports only”.
- All hole cut outs must be small in accordance to section EN 1993-1-5:2006 2.3
- If a grade of steel was to be used other than S335, S355 and S460 η will be taken as 1.00 regardless of National Annex.
- As we are only designing for the case where no stiffeners are being used, a → ∞ therefore c → ∞ so $V_{bf,Rd}$→0, where $V_{bf,Rd}$ is the contribution from the flange - see 5.4(1)
- The design assumes negligible contribution to the design shear force V_{Ed} from shear from torque, therefore V_{Ed} is restricted to 0.5 $V_{bw,Rd}$. Tekla Structural Designer will warn you if this limit is exceeded - see 7.1(1)

Moment Capacity (Columns: EC3)

The moment capacity check is performed at the point under consideration according to Clause 6.2.5(1):

- for the moment about the y-y axis, $M_{y,Ed}$, and
- for the moment about the z-z axis, $M_{z,Ed}$

The moment capacity can be influenced by the magnitude of the shear force (“low shear” and “high shear” conditions). Where the high shear condition applies, the moment capacity calculation is made less complicated by conservatively adopting a simplified shear area.

The maximum absolute shear to either side of a point of interest is used to determine the moment capacity for that direction.

High shear condition about y-y axis

The treatment of high shear is axis dependent. In this release for CHS, if high shear is present, the moment capacity check about the y-y axis is Beyond Scope.

High shear condition about z-z axis

For rolled sections in this release, if high shear is present normal to the z-z axis then the moment capacity check about the z-z axis is Beyond Scope.

For hollow sections, there is greater potential for the section to be used to resist the principal moments in its minor axis. Of course for CHS and SHS there is no major or minor axis and so preventing high shear arbitrarily on one of the two principal axes does not make sense. Nevertheless, if high shear is present normal to the z-z axis then in this release the moment capacity about the z-z axis is not calculated, the check is Beyond Scope.

If high shear is present in one axis or both axes and axial load is also present, the moment capacity check is given a Beyond Scope status.

If high shear and moment is present in both axes and there is no axial load (“biaxial bending”) the moment capacity check is given a Beyond Scope status.

Combined Bending and Axial Capacity (Columns: EC3)

The cross-section capacity check covers the interaction of axial load and bending to Clause 6.2.9 appropriate to the type (for example – doubly symmetric) and classification of the section.

If high shear is present in one axis or both axes and axial load is also present, the cross-section capacity check is given a Beyond Scope status.

If high shear and moment is present in both axes and there is no axial load (“biaxial bending”) the cross-section capacity check is given a Beyond Scope status.

The following additional points should be noted:

- the combined effects of axial load and bending are assessed and clause 6.2.9 (4) is not considered.
- the current “reduced plastic moduli” approach in the published tables is used and not the approximate method given in 6.2.9.1(5). The latter is less conservative than the current approach at low levels of 'n'.

Ultimate Limit State Buckling (Columns: EC3)

Classification for buckling checks -

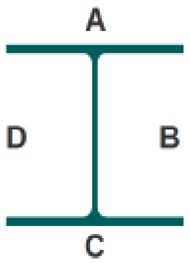
For rolled I sections, RHS and SHS classification varies along the member length due to the section forces changing along the member length - for combined buckling, the worst classification of the whole member (column stack) should be used. In theory it should be the worst classification in the segment length considered for buckling. However, the segment lengths for lateral torsional *buckling, minor axis strut buckling and major axis strut buckling can all be different. It is simpler and conservative therefore to use the worst classification in the entire member length (column stack).*

Compression buckling (Columns: EC3)

General columns must be checked to ensure adequate resistance to buckling about both the major and minor axes and they must also be checked in the torsional mode over an associated buckling length. Since the axial force can vary throughout the column and the buckling lengths in the two planes do not necessarily coincide, all buckling modes must be checked. There may be circumstances where it would not be safe to assume that the Combined Buckling check will always govern (see below).

Restraints

Restraints to strut buckling are determined from the incoming members described within *Tekla Structural Designer*. The buckling checks are based on these.



Restraining members framing into either Face A or C will provide restraint to major axis strut buckling. Members framing into either Face B or D will provide restraint to minor axis strut buckling. *Tekla Structural Designer* determines the strut buckling restraints but you can override these.

The program assumes that any member framing into the major or minor axis of the column provides restraint against strut buckling in the appropriate plane. If you believe that a certain restraint in a particular direction is not effective then you can either override the restraint or adjust the effective length to suit – to 2.0L for example.

Torsional and torsional flexural buckling restraint is only provided at points restrained coincidentally against major **and** minor axis strut buckling.

Provided a level is restrained coincidentally against major and minor axis strut buckling, the program assumes that any member framing into the appropriate faces provides restraint against torsional and torsional flexural buckling at that level. There are a number of practical conditions that could result in torsional restraint not being provided at floor levels. At construction levels this is even more possible given the likely type of incoming member and its associated type of connection. You must consider the type of connection between the incoming members and the column since these can have a significant influence on the ability of the member to provide restraint to one, none or both column flanges. For example, consider a long fin plate connection for beams framing into the column web where the beam stops outside the column flange tips to ease detailing. The fin plate is very slender and the beam end is remote from the column flanges such that it may not be able to provide any restraint to torsional or torsional flexural buckling. The fact that a slab is usually present may mitigate this. You are expected to override the ineffective restraint.

Tekla Structural Designer always assumes full restraint at the base and at the roof level when carrying out buckling design checks – you are warned on validation if your restraint settings do not reflect this. Restraints are considered effective on a particular plane providing they are within ±45° to the local coordinate axis system.

Effective lengths

In all cases *Tekla Structural Designer* sets the default effective length to 1.0L, it does not attempt to adjust the effective length in any way. **You** are expected to adjust the strut buckling effective length factor (up or down) as necessary. Different values can apply in the major and minor axis.

It is assumed that you will make a rational and “correct” choice for the effective lengths between restraints. The default value for the effective length factor of 1.0L may be neither correct nor safe.

The torsional and torsional flexural buckling effective length factor (1.0L) can not be changed.

Any strut buckling effective length can take the type “Continuous” to indicate that it is continuously restrained over that length. There is no facility for specifying torsional, or torsional flexural buckling effective lengths as “Continuous”.

There is no guidance in EC3 on the values to be used for effective length factors for beam-columns.

For general columns - The minimum theoretical value of effective length factor is 0.5 and the maximum is infinity for columns in rigid moment resisting (RMR) frames. Practical values for simple columns are in the range 0.7 to 2.0 (see For simple columns below). In theory, values less than 1.0 can be chosen for non-sway frames or for sway frames in which the effects of sway are taken into account using either the amplified forces method or P-Delta analysis. However, EC3 states that when second-order effects are included in this way then the design “may be based on a buckling length equal to the system length” i.e. an effective length factor of 1.0. The program default of 1.0 matches this requirement but allows you flexibility for special situations.

One such situation might be in RMR frames where the principal moments due to frame action preventing sway are in one plane of the frame. There will often be little or no moment out-of-plane and so, if using the amplified forces method, the amplification of these moments has little effect on the overall design. Nevertheless the stability out-of-plane can still be compromised by the lack of restraint due to sway sensitivity in that direction. In such cases a value of greater then 1.0 (or substantially greater) may be required. Similarly, in simple construction where only eccentricity moments exist, it is only the brace forces that 'attract' any amplification. Thus for the column themselves the reduced restraining effect of a sway sensitive structure may require effective length factors greater than 1.0.

For Simple columns - There is no concept of simple columns in EC3 and hence no information on effective lengths either. However, reference can be made to the “NCCI” on the subject of simple construction but none of this includes the clear guidance on effective lengths of simple columns that was included as Table 22 in BS 5950-1: 2000. Again the program defaults the effective length factor to 1.0

Compression resistance

The relevant buckling resistances are all calculated from Equation 6.47.

These consist of the flexural buckling resistance about both the major and minor axis i.e. $N_{b,y,Rd}$ and $N_{b,z,Rd}$ over the buckling lengths L_{yy} and L_{zz} and where required the buckling resistance in the torsional or flexural-torsional modes, $N_{b,x,Rd}$.

All section types are checked for flexural buckling. It is only hollow sections that do not need to be checked for torsional and torsional-flexural buckling.

Lateral Torsional Buckling (Columns: EC3)

Effective lengths

The value of effective length factor is entirely your choice. The default value is 1.0 and is editable for flanges A & C. Any individual segment (for either flange) can be 'continuously restrained' in which case no lateral torsional buckling (LTB) check is carried out for that flange over that segment.

For a level to be treated as torsional restraint it must have both A and C restraint **and** also be restrained for compression buckling in both the major and minor axis.

There is no specific factor for destabilizing loads - you can however adjust the 'normal' effective length factor to allow for such effects.

Lateral torsional buckling resistance

The LTB resistance is calculated from Equation 6.55.

LTB does not need to be checked for the following sections,

- circular and square hollow sections,
- equal and unequal flanged I/H sections loaded in the minor axis only.

Combined Buckling (Columns: EC3)

The column must be restrained laterally in two directions and torsionally, at the top and bottom of the 'design length'. This equates to LTB restraint to faces A and C and restraint to major and minor axis compression buckling all being coincident. A design length is allowed to have intermediate restraint and if the restraint requirements are not met at a particular floor then the design length does not have to be between adjacent floors. Thus a stack can 'jump' floors or sheeting rails can be attached. It is assumed that the restraints for compression buckling are fully capable of forcing the buckled shape. Hence, the compression buckling resistance is based on the restrained lengths whilst the LTB resistance ignores the intermediate restraint and hence is based on the full design length.

It is conservative to ignore the intermediate restraints in this latter case.

Loading within the design length is allowed.

Effective lengths

Effective lengths for flexural (i.e. strut major and strut minor) and lateral torsional buckling are as described in the appropriate section above.

Combined buckling resistance

The combined buckling resistance is checked in accordance with Equations 6.61 and 6.62. Both equations are evaluated at the ends of the design length and, except for simple columns, at the position of maximum moment, if that lies elsewhere.

Eccentricity moments due to beam end reactions are added to the “real” moments due to frame action:

- in the first case the uniform moment factors are calculated from the real moments and applied to the real moments. Eccentricity moments are only added if they are more critical.
- in the second case all moments are “combined” and all uniform moment factors are based on the combined moments and applied to them.

Caution:
Equations 6.61 and 6.62 are limited to doubly symmetric sections and do not consider torsional or torsional flexural buckling. Should either of these buckling modes govern the compression buckling check, you should consider very carefully whether the calculations provided by Tekla Structural Designer for combined buckling can be considered valid.

Serviceability limit state (Columns: EC3)

The column is assessed for sway and the following values are reported for each stack:

- Sway X (mm) and α_{critx}
- Sway Y (mm) and α_{crity}
- Sway X-Y (mm)

Depending on the reported α_{crit} the column is classified as Sway or Non sway accordingly.

A sway assessment is only performed for the column if the Alpha Crit Check box is checked on the Column Properties dialog. If very short columns exist in the building model these can distort the overall sway classification for the building. For this reason you may apply engineering judgement to uncheck the Alpha Crit Check box for those columns for which a sway assessment would be inappropriate.

Steel Brace Design to EC3

Steel Brace Design to EC3

Design method (Braces: EC3)

Unless explicitly stated all brace calculations are performed in accordance with the relevant sections of BS EN 1993-1-1:2005 (Ref. 1) (herein abbreviated to EC3) and the associated National Annex.

A basic knowledge of the design methods for braces in accordance with the design code is assumed.

Classification (Braces: EC3)

No classification is required for braces in tension.

Braces in compression are classified according to Table 5.2 as either: Class 1, Class 2, Class 3 or Class 4.

Class 4 sections are not allowed.

Axial Tension (Braces: EC3)

An axial tension capacity check is performed according to Clause 6.2.3.(1)

The following points should be noted:

- Cl 6.2.3 (3) - is not considered
- Cl 6.2.3 (4) - is not considered
- Cl 6.2.3 (5) - is not considered

Axial Compression (Braces: EC3)

An axial compression capacity check is performed according Clause 6.2.4.(1)

Compression Buckling (Braces: EC3)

If axial compression exists, the member is also assessed according to Clause 6.3.1.1(1) for flexural buckling resistance about both the major and minor axis i.e. $N_{b,y,Rd}$ and $N_{b,z,Rd}$ over the buckling lengths L_{yy} and L_{zz} and where required the torsional, or flexural-torsional buckling resistance, $N_{b,x,Rd}$.

For single and double angles (both equal and unequal) there is also a compression buckling check about the v-v axis, over the buckling length L_{vv}. For single angles, L_{vv} is the system length L, while for double angles L_{vv} is L/3.

All section types are checked for flexural buckling. It is only hollow sections that do not need to be checked for torsional and torsional-flexural buckling.

Different effective length factors can be applied for flexural buckling in the major and minor axis. For single and double angles an effective length factor can also be applied in the v-v axis. The default effective length is 1.0L in all 3 cases. **You** are expected to adjust the effective length factor (up or down) as necessary.

The torsional and torsional flexural buckling effective length factor (1.0L) can not be changed.

Steel Single, Double Angle and Tee Section Design to EC3

Design Method (Angles and Tees: EC3)

The EC3(Ref. 1) design method adopted is dictated by the member characteristic type:

"Beam", "Truss member top" or "Truss member bottom" characteristic:

Member is designed for axial tension, compression, shear, bending and combined forces - consistent with the method detailed in Steel Beam Design to EC3

"Brace", "Truss internal" or "Truss member side" characteristic:

Member is designed for axial tension, compression and compression buckling only - consistent with the method detailed in Steel Brace Design to EC3

Additional Angle and Tee Limitations have to be considered when designing these sections to the above design methods.

Angle and Tee Limitations (EC3)

In the current version when designing tees, single, and double angles to EC3, the following checks remain beyond scope:

	Tee	**Angle**	**Double Angle**
Classification	ok	ok	ok
Axial tension	ok	ok	ok
Axial compression	ok	ok	ok
Shear	ok	ok	ok
Buckling	ok	ok	ok
Combined strength	ok	ok	ok
LTB	Beyond scope	ok	Beyond scope

Combined buckling	Beyond scope	Beyond scope	Beyond scope
Deflection	ok	ok	ok

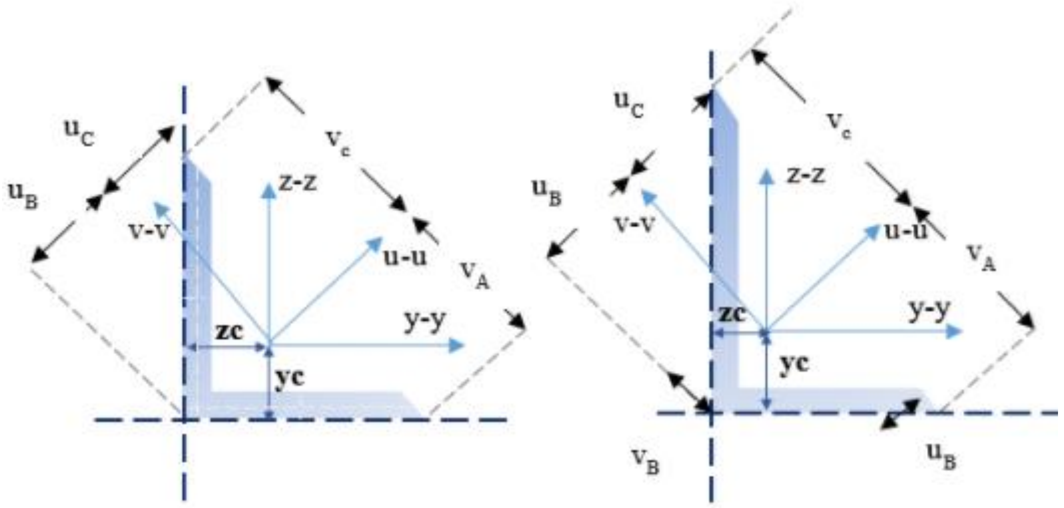
In addition, the following limitations apply:

- All sections and in particular single angles are assumed to be effectively loaded through the shear centre such that no additional torsion moments are developed. In addition no direct allowance is made for 'destabilizing loads'.
- Design excludes bending of the outstand leg of single and double angles loaded eccentrically e.g. supporting masonry.
- Conditions of restraint can be defined as top and bottom flange for lateral torsional buckling LB. It is upon these that the buckling checks will be based. For the current release intermediate LTB restraints are omitted (i.e. only fully restrained for LTB, or unrestrained).
- Single, double angles and tee sections subject to moment with high shear are beyond scope.

Section Axes (Angles and Tees: EC3)

For all sections -

- y-y is the axis parallel to the flanges (major axis)
- z-z is the axis perpendicular to the flanges (minor axis)
- for Single Angles and Double Angles
  - z-z parallel to long side (leg) - single angles
  - z-z parallel to long side (leg) - double angles with long leg back to back
  - z-z parallel to short side (leg) - double angles with short leg back to back
- u-u is the major principal axis for single angles
- v-v is the minor principal axis for single angles



Single Angles - Section Axes

Design Procedures (Angles and Tees: EC3)

This section includes key notes and assumptions made for the EC3(Ref. 1) design of tees and angle sections.

Classification checks

For axial compression and bending both the web and flange (Leg 1 and Leg 2) are classified as Class 1, Class 2, Class 3 or Class 4 and the worst of the two is the resultant classification for that cross section.

The rules from Table 5.2 (sheet 2 of 3) of EC3 are used for tee sections. In particular the rules of “Part subject to compression” are used to classify the tee section since these are more conservative compared to the limits of “Part subject to bending and compression”.

For double angles and single angles the rules from Table 5.2 (sheet 3 of 3) of EC3 are used.

Class 4 section classification is only allowed for tees, double angles and single angles.

Axial Tension check

Section 6.2.3 of EC3 is used for this design check.

Axial Compression check

Section 6.2.4 of EC3 is used for this design check.

Effective length:

The value of effective length factor is entirely at the user's choice. The default value is generally 1.0 although for truss members, there are special settings for the effective length depending upon the type of section and its position in the truss.

Different values can apply in the major and minor axis. Coincident strut restraint points in these two directions define the length for torsional and torsional flexural buckling and this can also have an effective length factor (this is assumed to be 1.0 and cannot be changed).

There is no guidance in EC3 on the values to be used for effective length factors for beam-columns although Annex BB does contain some information on the effective lengths to be used in trusses but not for single, double angles and tees.

It is the responsibility of the user to adjust the value from 1.0 (for the effective length factor) and to justify such a change on the compression page.

For tees:

Check

(1) the buckling length in the major axis – Use L_{yy} = L * major factor

(2) the buckling length in the minor axis – Use L_{zz} = L * minor factor

(3) the buckling length for the torsional mode – Use L_{xx} = 1.00 * minor factor

For single and double angles:

Check

(1) the buckling length in the major axis – Use L_{yy} = L * major factor

(2) the buckling length in the minor axis – Use L_{zz} = L * minor factor

(3) the buckling length for the torsional mode – Use L_{xx} =1.00 * L

  a. Double angles – Check as single angle

    i. Use $L_y = L_{yy}/3$

    ii. Use $L_z = L_{zz}/3$

    iii. Use $L_x = L_{xx}/3$

  b. Double angles – Check as double angle

(4) the buckling length for the principal axis, v-v – Use L_{vv} = 1.00 * L

  a. Double angles – Check as single angle

    i. Use $L_v = L_{vv}/3$

For double angles for (4) & (3a) minor principal axis buckling & torsional buckling respectively – half of the axial force and half of the double angle area is used.

Shear check

Section 6.2.6 of EC3 is used for this design check.

Moment check

Section 6.2.5 of EC3 is used for this design check.

Tees, double angles and single angles are designed as Class 4. Equation 6.15 is used for class 4 slender sections.

Tees, double angles and single angles subject to moment with high shear are beyond scope.

Moment capacity for Class 4 slender sections:

Class 4 sections are designed as Class 3 effective sections.

Hence, additional moments are induced in the member due to the shift of the centroid of the effective cross-section compared to that of the gross section when under axial compression only.

Thus:

$\Delta M_{Ed,y} = e_y \times N_{Ed,max}$

$\Delta M_{Ed,z} = e_z \times N_{Ed,max}$

Where:

$N_{Ed,max}$ is the max compressive force in the span.

For tees and double angles $e_y = 0$. Hence, total minor design moment = minor design moment.

Where,

e_y and e_z = the shift of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section

e_y = abs(cy_{new} – cy)

e_z = abs(cz_{new} – cz)

So finally, a total moment is obtained for which the moment design check is performed:

$M_{total\ y}$ = Abs($M_{Ed,y}$) + Abs($\Delta M_{Ed,y}$)

$M_{total\ z}$ = Abs($M_{Ed,z}$) + Abs($\Delta M_{Ed,z}$)

Single angles - asymmetric sections:

Single angles with continuous lateral – torsional restraint along the length are permitted to be designed on the basis of geometric axis (y, z) bending.

Single angles without continuous lateral – torsional restraint along the length are designed using the provision for principal axis (u, v) bending since we know that the principal axes do not coincide with the geometric ones.

$\Delta M_u = \Delta M_y \times \cos\vartheta + \Delta M_z \times \sin\vartheta$

$\Delta M_v = -\Delta M_y \times \sin\vartheta + \Delta M_z \times \cos\vartheta$

Note that when principal axis design is required for single angles and the classification is Class 4, all moments are resolved into the principal axes (total moment in the principal axes u-u and v-v).

Combined bending and axial check

Section 6.2.9 of EC3 is used for this design check.

For Class 3 - Equation 6.42 is applied:

Abs (N_{Ed} / A) + abs ($M_{y,Ed}$ /$W_{el,min,y}$) + abs ($M_{z,Ed}$/$W_{el,min,z}$) ≤ f_y / γ_{M0}

For Class 4 - Equation 6.43 is applied:

Abs (N_{Ed}/A_{eff}) + (abs ($M_{y,Ed}$) + abs ($\Delta M_{y,Ed}$)) / $W_{eff,min,y}$ + abs ($M_{z,Ed}$) + abs ($\Delta M_{z,Ed}$)) / $W_{eff,min,z}$ ≤ f_y / γ_{M0}

Note that total moments are used when the section classification is Class 4.

For Class 4 cross section capacity - Equation 6.44 is applied.

Lateral torsional buckling check

LTB check for tees and double angles is currently beyond scope.

EC3 is completely silent on LTB check for asymmetric sections such as single angles and mono-symmetric sections such as double angles and tees.

Hence we follow the approach of the Blue Book(Ref. 10):

Firstly we calculate the equivalent slenderness coefficient (φ) (From Blue book) and the equivalent slenderness λLT (BS approach).

Then we find the non-dimensional slenderness in order to follow the EC design approach.

Conservatively we have taken:

All intermediate LTB restraints for single angles, double angles and tees are ignored.

Combined buckling check

In the current version this check is beyond scope for single angles, double angles and tees.

Deflection of Single Angles

If a single angle is continuously restrained the major geometric moment and major geometric section properties are used in the general equation governing the beam deflection.

Single Angle Deflections (continuously restrained, unrestrained)

However, because single angle geometric axes are not coincident with the principal axes; a different procedure is required if the angle is not continuously restrained, the procedure being as follows:

1. External loads are transposed from the geometric axes to the principal axes.
2. The deflection equations are used to calculate deflections in the principal axes.
3. These principal axis deflections are then transposed to geometric axes again.

References (EC3 and EC4)

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8. **The Steel Construction Institute**. *Publication P355. Design of Composite Beams with Large Web Openings*. **SCI 2011**.

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