Tekla Structural Designer 2018i

Reference Guides (British Standards)

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(6.1.06)

Table of Contents

Analysis Verification Examples

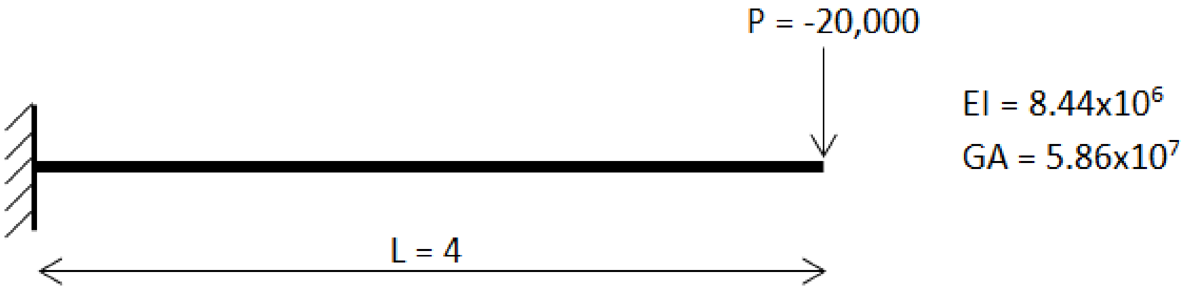
A small number of verification examples are included in this section. Our full automatic test suite for the Solver contains many hundreds of examples which are run and verified every time the Solver is enhanced.

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%

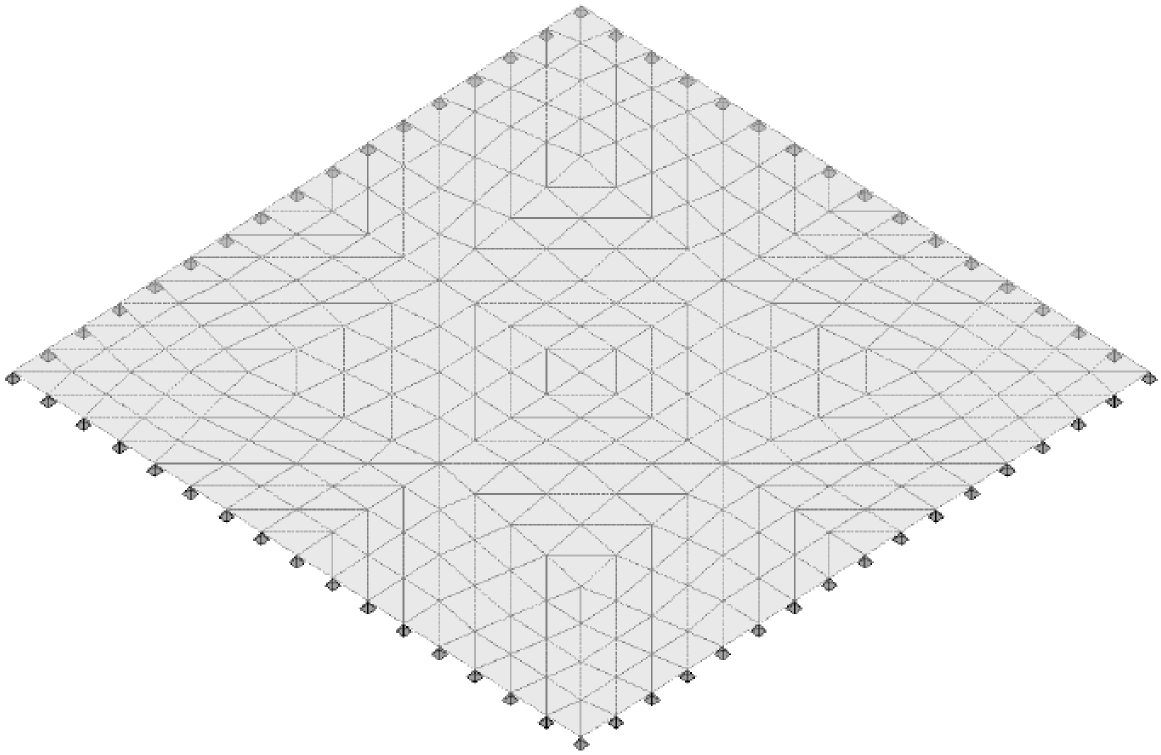
Conclusion

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%

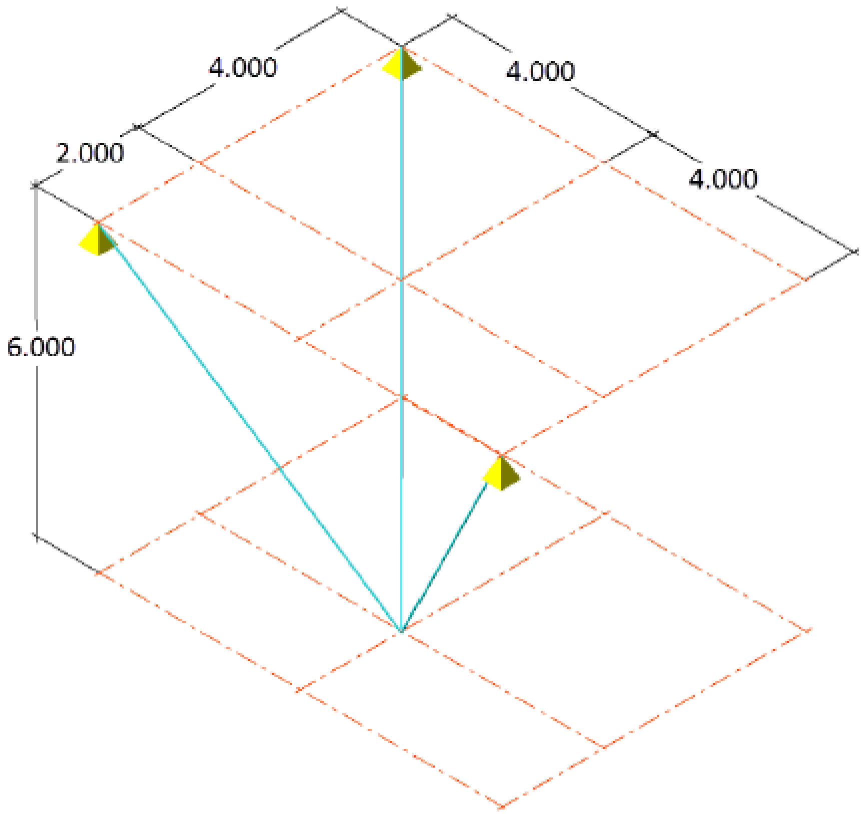
Conclusion

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%

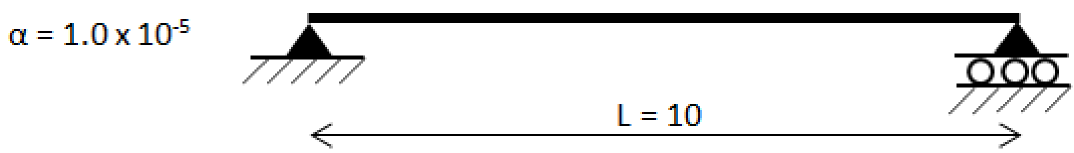
Conclusion

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%

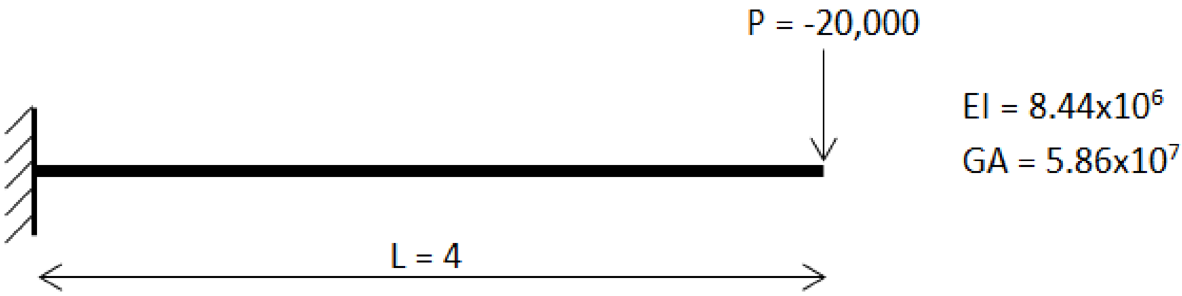
Conclusion

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%

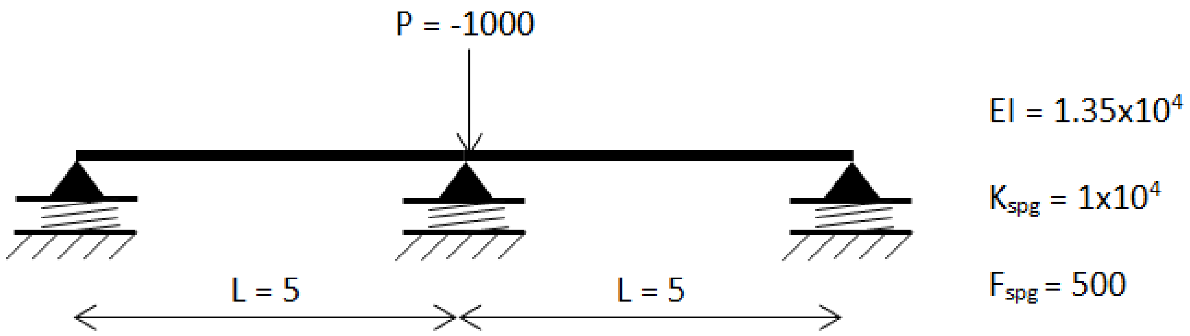
Conclusion

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

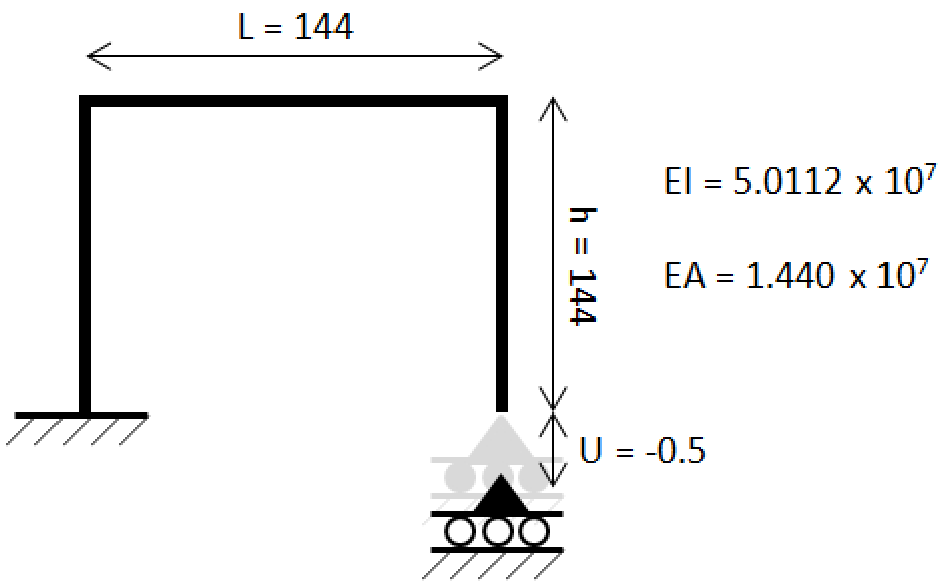
Conclusion

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

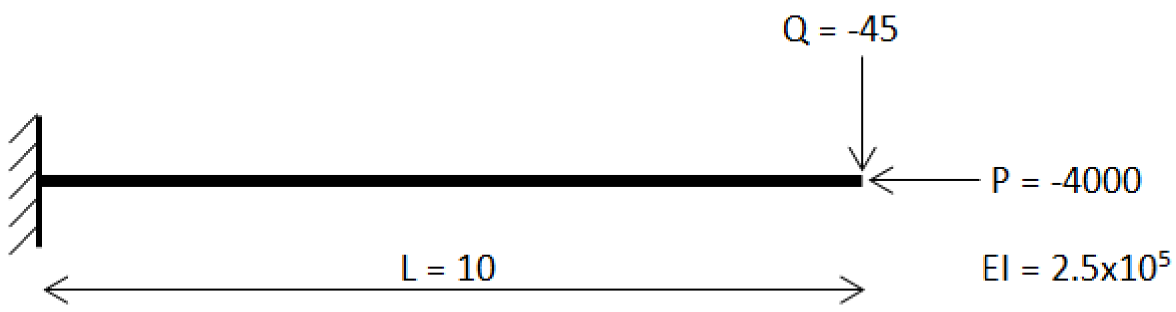
Conclusion

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

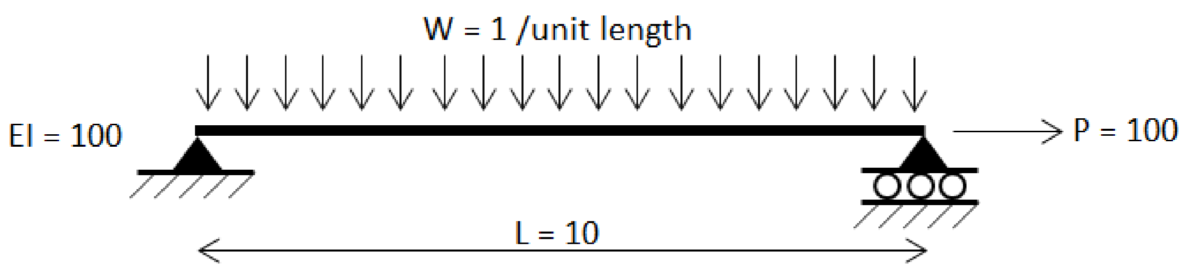
Conclusion

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.

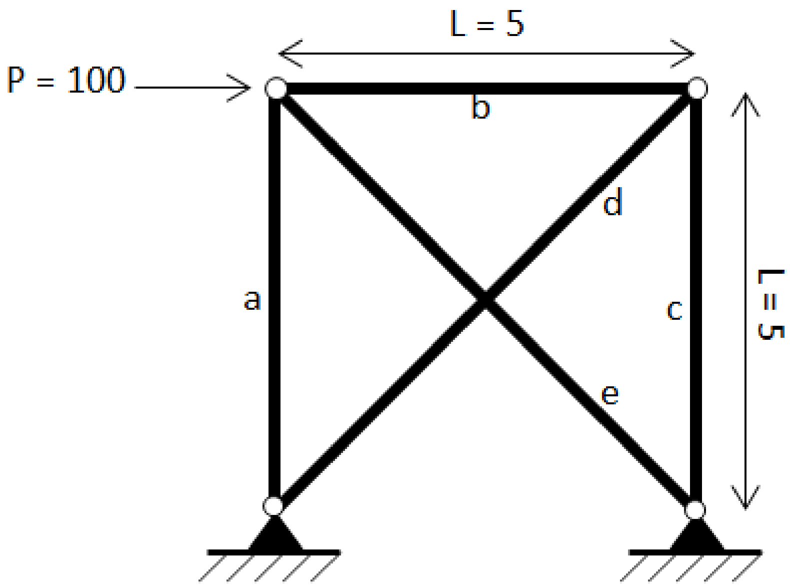
Reference

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 2nd order effects; this requires the value of EA to be sufficiently large to make the 2nd 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

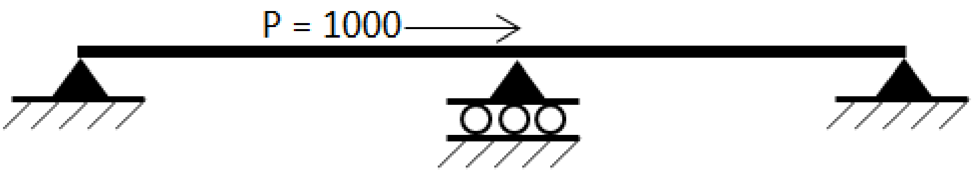
Conclusion

An exact match is observed between the values reported by the solver and the values predicted using statics. A 1st 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

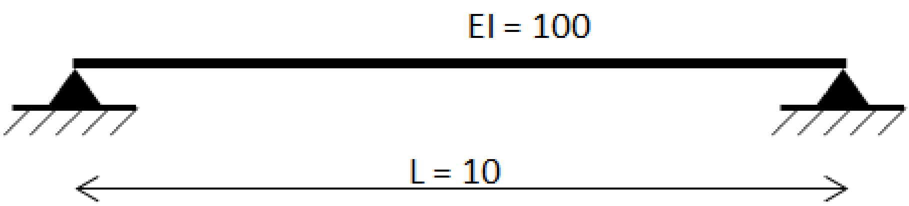
Conclusion

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%

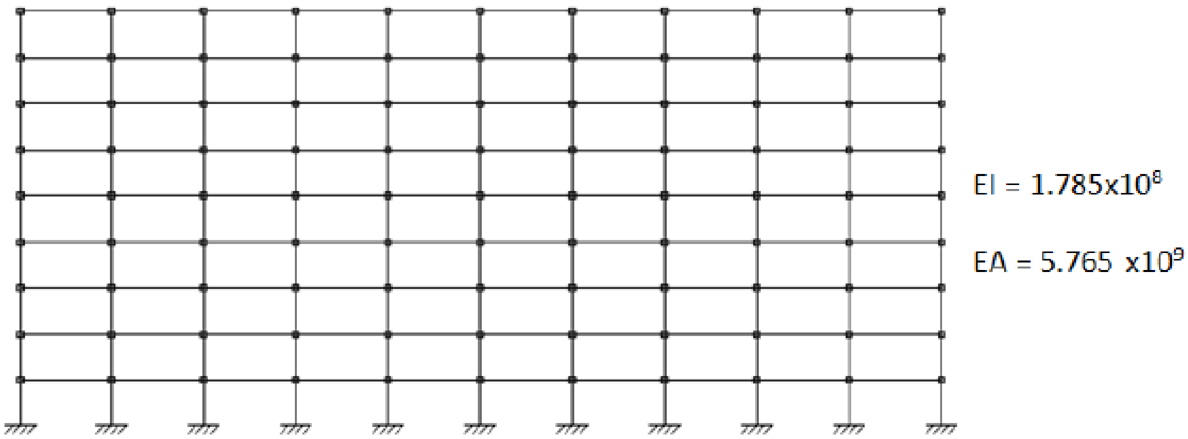
Conclusion

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.

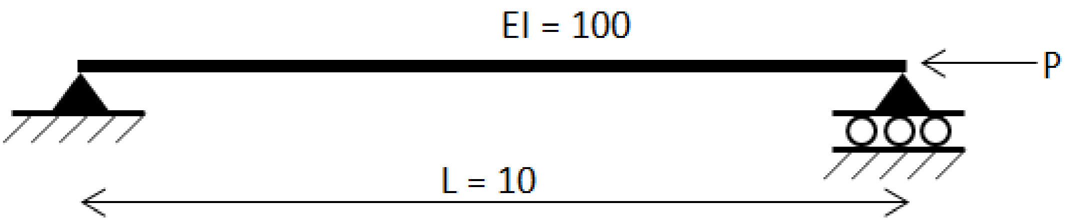
References

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%

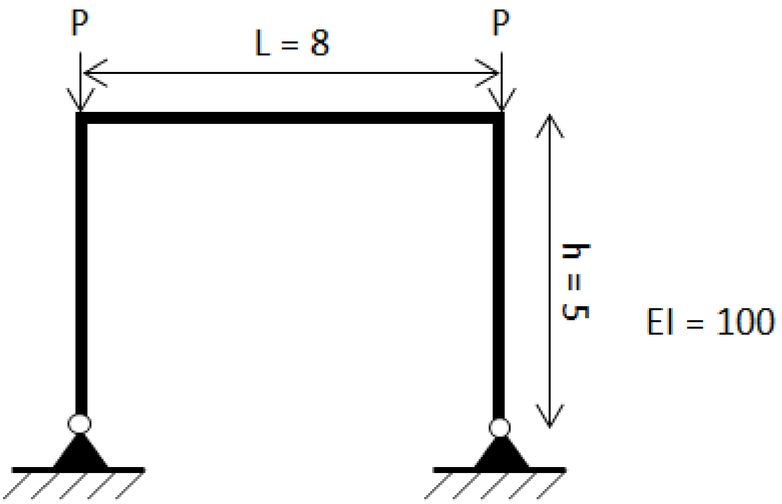
Conclusion

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 - British Standards

British Standards Loading

This handbook provides a general overview of how loadcases and combinations are created in *Tekla Structural Designer* when a British Standards (BS) head code is applied. The Combination Generator for BS loading is also described.

Load Cases (BS)

Loadcase Types (BS)

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 (BS)

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 (BS)

Imposed Load Reductions

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.

Imposed loads are only automatically reduced on:

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

Tekla Structural Designer does not automatically apply imposed load reductions to floors. For concrete beams and slabs 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.

Wind Loads (BS)

The BS6399-2 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 BS6399-2:1997.

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.

For further information on the wind loading capabilities of *Tekla Structural Designer* refer to the Wind Modelling Engineer's Handbook.

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.

Combinations (BS)

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...**

For BS codes we are assuming that the wind load applied in manually defined combinations, or via the combination generator, satisfies the minimum horizontal load requirement (BS5950 Cl 2.4.2.3 (1% of factored dead load) and BS8110 Cl 3.1.4.2 (1.5% characteristic dead weight)).
If this is not the case, i.e. the wind load is less than the minimum proportion of dead load specified in the code, then the user needs to consider manually creating a minimum horizontal load combination.

Manually Defined Combinations (BS)

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

Notional Horizontal Forces (NHFs) (BS)

NHF's are automatically derived from the loadcases within the current combination, their magnitude being calculated in accordance with BS5950 cl 2.4.2.3 as 0.5% of the factored vertical load that passes through any beam/column intersection in the structure.

BS8110 cl 3.1.4.2 has a requirement for notional horizontal load "NHL" This does NOT equate to the NHF requirement described above. The calculation of "NHL" as defined in BS8110 is beyond scope in the current version of Tekla Structural Designer.

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

- NHF Dir1+

- NHF Dir1-
- NHF Dir2+
- NHF Dir2-

The net result is that any combination is able to have up to 2 Notional Loads applied within it - one from Dir1 (+ or -) and one from Dir2 (+ or -). Note however that Dir1+ can not be added with Dir1- (and similarly Dir2+ can not be added with Dir2-).

The Combinations Generator (BS)

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

Combination Generator - Combinations

The first page of the generator lists the combinations applicable (with appropriate strength factors).

The following basic load combinations are created:-

- 1.4(Dead) + 1.6(Imposed or Snow)
- 1.2(Dead) + 1.2(Imposed or Snow) + 1.2 (Wind)
- 1.0(Dead) + 1.4(Wind)

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

The combination names are generated automatically.

Combination Generator - Service

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

The following basic load combinations are created:-

- 1.0(Dead) + 1.0(Live or Snow)
- 1.0(Dead) + 0.8(Live or Snow) + 0.8(Wind)
- 1.0(Dead) + 1.0(Wind)

Combination Generator - NHF

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

Any combination with wind in is automatically greyed.

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

Combination Classes (BS)

Having created your combinations you classify them as either Gravity Combinations, Lateral Combinations, Seismic Combinations, or Vibration Mass Combinations and also (where applicable) indicate whether they are to be checked for strength or service conditions, or both.

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

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

Concrete Design - BS 8110

Introduction to BS8110 Design

This handbook describes how BS 8110-1:1997 (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 8110-1:1997

Within the remainder of this guide BS 8110-1:1997 is referred to as BS8110.

Beam Design to BS 8110

Limitations and Exclusions (Beams: BS 8110)

The following general exclusions apply:

- Bundled bars.
- Design for minor axis bending and shear.
- Design for axial forces.
- Beams with a ratio of clear span/effective depth < 2.0 are classified as deep beams and "Beyond Scope"

Materials (Beams: BS 8110)

Concrete

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

Reinforcement

The reinforcement options are:

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

Slender Beams (Beams: BS 8110)

The clear distance between restraints is taken as below;

For simply supported or continuous beams:[A]

Clear distance between restraints $\leq$ MIN [$60*b_c$, $250*b^2_c/d$]

For cantilevers with lateral restraints only at support:

Clear distance between restraints $\leq$ MIN [$25*b_c$ or $100*b^2_c/d$]

where

b_c = breadth of the compression face of the beam,

d = effective depth of beam

[A] *BS 8110-1:1997 3.4.1.6*

Cover to Reinforcement (Beams: BS 8110)

The greatest nominal cover is selected derived from:

1. Bar Size[1]- The nominal cover to all steel should be such that the resulting cover to main bar is not less than the size of the main bar.

$c_{nom, \phi} \geq \phi - \phi_{link}$

where

ϕ = maximum diameter of bar in longitudinal reinforcement

ϕ_{link} = diameter of link

2. Fire Resistance[2]- For fire protection the values given in Table 3.4 of BS 8110-1:1997 ensure that fire resistance requirements are satisfied.

The BS8110 "Environmental Conditions" and "Fire Resistance" provisions are not implemented in the current version of Tekla Structural Designer.

3. Nominal maximum size of aggregate –[3]

The nominal cover should be not less than the nominal maximum size of the aggregate h_{agg}

$c_{nom,hagg} \geq h_{agg}$

Nominal limiting cover to reinforcement:

$c_{nom,lim} = \text{MAX}\,[c_{nom,\ \varphi}, h_{agg}]$

A minimum value for the nominal cover, $c_{nom,\ u}$, is set by the user with the following default values;

Nominal cover from top surface of beam, $c_{nom,u,top}$	Default value: 30.0mm
Nominal cover from bottom surface of beam, $c_{nom,u,bot}$	Default value: 30.0mm
Nominal cover from side face of beam, $c_{nom,u,side}$	Default value: 30.0mm
Nominal cover from end surface of beam, $c_{nom,u,end}$	Default value: 30.0mm

The nominal limiting cover depends on the diameter of the reinforcement, and the nominal covers to be used for design and detailing purposes are given by;

Nominal limiting cover to reinforcement from the side face of a beam;

$c_{nom,lim,side} = \text{MAX}[\phi_{top} - \phi_{link}, \phi_{bot} - \phi_{link}, \phi_{side} - \phi_{link}, h_{agg}]$

Nominal limiting cover to reinforcement from the top face of a beam;

$c_{nom,lim,top} = \text{MAX}[\phi_{top} - \phi_{link}, h_{agg}]$

Nominal limiting cover to reinforcement from the bottom face of a beam;

$c_{nom,lim,bot} = \text{MAX}[\phi_{bot} - \phi_{link}, h_{agg}]$

Nominal cover to reinforcement from the end of a beam;

$c_{nom,lim,end} = \text{MAX}[\phi_{top} - \phi_{link}, \phi_{bot} - \phi_{link}, \phi_{side} - \phi_{link}, h_{agg}]$

and where

ϕ_{top} = maximum diameter of the longitudinal reinforcing bar nearest to the top surface of the beam

ϕ_{bot} = maximum diameter of the longitudinal reinforcing bar nearest to the bottom surface of the beam

ϕ_{side} = the diameter of the longitudinal reinforcing bar nearest to the side of the beam

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

1. BS 8110-1:1997 3.3.1.2

2. BS 8110-1:1997 3.3.6, Table 3.4

3. BS 8110-1:1997 3.3.1.3

Design Parameters for Longitudinal Bars (Beams: BS 8110)

For each of these parameters, the user defined limits (specified in **Design Options > Beam > Reinforcement Settings**) are considered in addition to any BS 8110 recommendations.

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

$s_{cl,min}$ ≥ MAX[h_{agg} + 5mm, $s_{cl,u,min}$]

where

h_{agg} = maximum size of coarse aggregate

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

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

$s_{cl,min}$ ≥ $2h_{agg}/3$

IF

ϕ > h_{agg} + 5mm,

THEN

$s_{cl,min}$ > ϕ

where

ϕ = the maximum diameter of bar

Maximum Spacing of Tension Bars

When the maximum crack width is limited to 0.3mm and nominal cover to reinforcement does not exceed 50 mm,

$s_{cl,max}$ ≤ $47000/f_s \leq 300$mm

where

$s_{cl,max}$ = maximum **clear** horizontal distance between bars in tension

f_s = the design service stress in the tension reinforcement of a member[A]

f_s = $(5*f_y*A_{s\ req})/(8*A_{s\ prov})*(1/\beta_b)$

$A_{s\ req}$ = required area of tension reinforcement

$A_{s\ prov}$ = provided area of tension reinforcement

β_b = (moment at the section after redistribution) / (moment at the section before redistribution)

[A]*BS 8110-1:1997 equation 8*

In Tekla Structural Designer when designing to BS 8110 the value of crack width is limited to 0.3mm.

Minimum Area of Reinforcement

The minimum area of longitudinal **tension reinforcement**, As,min, is given by;[A]

a) For rectangular sections:

$A_{s,min}$ ≥ 0.0024*b*h (For f_y = 250 N/mm^2)

$A_{s,min}$ ≥ 0.0013*b*h (For f_y = 500 N/mm^2)

b) For flanged beams, web in tension:

i) $b_w/b < 0.4$

$A_{s,min}$ ≥ $0.0032*b_w*h$ (For f_y = 250 N/mm^2)

$A_{s,min}$ ≥ $0.0018*b_w*h$ (For f_y = 500 N/mm^2)

ii) $b_w/b \geq 0.4$

$A_{s,min}$	≥	$0.0024*b_w*h$	(For f_y = 250 N/mm^2)
$A_{s,min}$	≥	$0.0013*b_w*h$	(For f_y = 500 N/mm^2)

c) For flanged beams, flange in tension:

i) T beam

$A_{s,min}$	≥	$0.0048*b_w*h$	(For f_y = 250 N/mm^2)
$A_{s,min}$	≥	$0.0026*b_w*h$	(For f_y = 500 N/mm^2)

ii) L beam

$A_{s,min}$	≥	$0.0036*b_w*h$	(For f_y = 250 N/mm^2)
$A_{s,min}$	≥	$0.002*b_w*h$	(For f_y = 500 N/mm^2)

[A] BS 8110-1:1997 3.12.5.3, Table 3.25

The minimum area of longitudinal **compression reinforcement**, $A_s'_{,min}$, is given by;

a) For rectangular sections:

$A_s'_{,min}$	≥	$0.002*b*h$	(For f_y = 250 N/mm^2)
$A_s'_{,min}$	≥	$0.002*b*h$	(For f_y = 500 N/mm^2)

b) For flanged beam:

i) Flange in compression

$A_s'_{,min}$	≥	$0.004*b*h_f$	(For f_y = 250 N/mm^2)
$A_s'_{,min}$	≥	$0.004*b*h_f$	(For f_y = 500 N/mm^2)

ii) Web in compression

$A_s'_{,min}$	≥	$0.002*b_w*h$	(For f_y = 250 N/mm^2)
$A_s'_{,min}$	≥	$0.002*b_w*h$	(For f_y = 500 N/mm^2)

where

b	=	breadth of section
b_w	=	breadth or effective breadth of the rib for T or L section, average breadth of the concrete below the flange

f_y = characteristic strength of reinforcement

h = overall depth of the cross-section of a reinforced member

h_f = depth of flange

The minimum percentage of reinforcement is given in Table 3.25 of BS 8110-1:1997 for fy 250 Mpa and 500 Mpa. For other grades linear interpolation is applied.

Maximum Area of Reinforcement

The maximum area of longitudinal **tension reinforcement,** $A_{s,max}$, is given by[2];

$A_{s,max} \leq 0.04*A_c$

The maximum area of longitudinal **compression reinforcement**, $A_s'_{,max}$, is given by;

$A_s'_{,max} \leq 0.04*A_c$

where

A_c = the cross sectional area of the beam

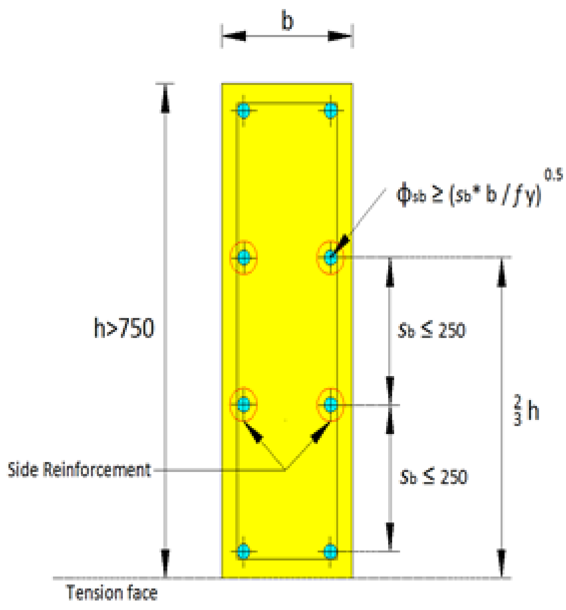
= h*b (For rectangular section)

= $h*b_w$ (For flanged section)

1. BS 8110-1:1997 3.12.11.1

2. BS 8110-1:1997 3.12.6.1

Side Reinforcement in Beams (Beams: BS 8110)



To control cracking in beams with a total depth > 750mm, side bars are provided in the side faces of the beam.[1]

Minimum diameter of bars in side of beam[2], φ_{sb}

$\phi_{sb} \geq (s_b * b / f_y)^{0.5}$

Where

s_b = bar spacing ≤ 250mm

b = breadth of section

IF

b > 500mm

THEN

b = 500

f_y = Characteristic strength of reinforcement

The diameter of the bar to be used is subject to qualification by the user therefore;

$\phi_{sb} = \mathrm{MAX}[\phi_{sb}, \phi_{sb,min}]$

The maximum spacing of side reinforcement bars[3] $s_{b,\,max} \leq 250$mm

The distribution of these bars is over a distance of two-third of the beam's overall depth measured from its tension face.

The number of side bars in each vertical face of the beam is given by;

n_{sb} = $[d-(h/3)]/s_{b, max}$

where

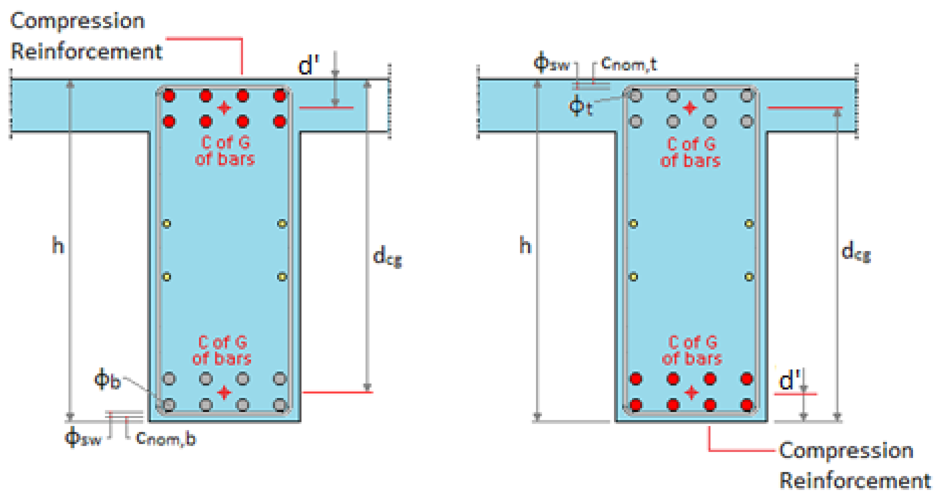
d = effective depth of the tension reinforcement

1. BS 8110-1:1997 3.12.11.2.6
2. BS 8110-1:1997 3.12.5.4
3. BS 8110-1:1997 3.12.11.2.6

Effective Depth of Section (Beams: BS 8110)

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' 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: BS 8110)

Although BS 8110-1:1997 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: BS 8110)

Calculate the value of *K* from;

K = $M/(f_{cu}*b*d^2)$

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

K' = (0.156)

where

M = design ultimate moment

f_{cu} = characteristic strength of concrete

b = width or effective width of the section

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

Calculate the lever arm, *z* from;

z = $MIN\{d*[\ 0.5 + (0.25 - (K/0.9))^{0.5}\ ], 0.95*d\}$

Calculate the depth to neutral axis, *x* from;

x = (d-z)/0.45

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

$A_{s,reqd}$ = $M/(0.87*f_y*z)$

IF K > K' THEN compression reinforcement is required.

The area of compression reinforcement, A'_s is then given by;

A_s' = $(K - K')\ f_{cu}*b*d^2/\ (f'_s\ (d - d'))$

Where

A_s' = area of compression reinforcement

d' = depth to the c of g of compression reinforcement

f'_s = compression stress in steel

= $0.87*f_y$ if $d'/d \leq 0.5*\{1 - (f_y/800)\}$

= $E_s*\varepsilon_c*\{1 - (2d'/d)\}$ if $d'/d > 0.5*\{1 - (f_y/800)\}$[A]

Where

E_s = the elastic modulus for steel = $2*10^5$ N/mm^2

ε_c = strain in concrete at the outermost compression fibre = 0.0035

Calculate the lever arm, *z* from;

$z = MIN\{d*[0.5 + (0.25 - (K'/0.9))^{0.5}], 0.95*d\}$

The total area of tension reinforcement required is given by;

$A_{s,reqd} = (K'*f_{cu}*b*d^2) / (0.87*f_y*z) + A_s'$

[A] *Book of A.H.Allen "Reinforced Concrete Design To BS 8110" section 5.2.2*

Design for Bending for Flanged Sections (Beams: BS 8110)

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[A];

$K = M/(f_{cu}*b_{eff}*d^2)$

Calculate the lever arm, z from;

$z = MIN\{d*[0.5 + (0.25 - (K/0.9))^{0.5}], 0.95*d\}$

Calculate the depth of neutral axis x, from;

$x = (d-z)/0.45$

The depth of the rectangular compression stress block, a is given by;

$a = 0.9*x$

IF $a \le h_f$ 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 $b_w = b_{eff}$.

Compression reinforcement is required if K > K'.

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

NOTE: It is assumed that if thickness of flange (h_f) is less than 0.1*d then the beam will be treated as rectangular.

Calculate the value of β_f as below;

β_f = $\{0.45*(h_f/d)*(1 - (b_w/b_{eff}))*(1 - (h_f/2*d))\} + \{0.15*(b_w/b_{eff})\}$[B]

Calculate the moment of resistance of the section as below;

M_R = $\beta_f * f_{cu} * b_{eff} * d^2$

Calculate the value of K

K = $M / (f_{cu} * b_{eff} * d^2)$

Calculate the limiting value of K, known as K'

K' = 0.156

IF

M ≤ M_R , $h_f < 0.45*d$ and $K \le K'$

THEN

A_s = $\{M + (0.1*f_{cu}*b_w*d)*(0.45*d - h_f)\} / (0.87*f_y*(d - 0.5*h_f))$[C]

ELSE

The ultimate resistance moment of the flange, M_f is given by;

M_f = $(0.67* f_{cu}/\gamma_m)*h_f*(b_{eff}-b_w)*(d-0.5*h_f)$

Where

γ_m = partial safety factor for concrete in flexure = 1.5[D]

The remaining design moment is taken by the web M_w, is given by;

M_w = $M - M_f$

Calculate the value of K_w from;

K_w = $(M_w)/(f_{cu}*b_w*d^2)$

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

K' = (0.156)

IF $K_w \le K'$ THEN compression reinforcement is not required.

Calculate the lever arm, z from;

$z = \mathrm{MIN}\{d*[0.5 + (0.25 - (K_w/0.9))^{0.5}], 0.95*d\}$

The area of tension reinforcement required is then given by;

$A_{s,reqd} = [M_f / (0.87* f_y* (d - 0.5* h_f))] + [M_w / (0.87* f_y*z)]$

Calculate the depth to the neutral axis, x from;

$x = (d - z)/0.45$

IF $K_w > K'$ THEN compression reinforcement is required.

The ultimate resistance moment of the web only, M_{uw} is given by;

$M_{uw} = K'*f_{cu}*b_w*d^2$

The compression reinforcement is required to resist a moment of magnitude $M_w - M_{uw}$

The area of compression reinforcement, A'_s is then given by;

$A_s' = (M_w - M_{uw}) / (f'_s (d - d'))$

Where

f'_s = compression stress in steel

$= 0.87*f_y$ if $d'/d \le 0.5*\{1 - (f_y/800)\}$

$= E_s*\varepsilon_c*\{1 - (2d'/d)\}$ if $d'/d > 0.5*\{1 - (f_y/800)\}$

Calculate the lever arm, z from;

$z = \mathrm{MIN}\{d*[0.5 + (0.25 - (K'/0.9))^{0.5}], 0.95d\}$

Calculate total area of tension reinforcement from;

$A_{s,reqd} = [M_f / (0.87* f_y* (d - 0.5* h_f))] + [M_{uw} / (0.87* f_y*z)] + A_s'$

[A] *BS 8110-1:1997 3.4.4.1*
[B] *BS 8110-1:1997 3.4.4.5 equation 2*
[C] *BS 8110-1:1997 3.4.4.5 equation 1*
[D] *BS 8110-1:1997 Table 2.2*

Design for Shear (Beams: BS 8110)

Design Shear Resistance (Beams: BS 8110)

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

V_{max} = $MIN\{(0.8*(f_{cu})^{0.5}*b_v*d),(5*b_v*d)\}$[AAA]

where

b_v = breadth of section (for a flanged beam this is taken as the average width of the rib below the flange.)

d = effective depth of section

IF

$V_{Ed,max} \leq V_{max}$

where

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

THEN the shear design process can proceed.

ELSE the shear design process FAILS since the section size is inadequate for shear. No further shear calculations can be carried out in the region under consideration and a warning is displayed.

[A]*BS 8110-1:1997 3.4.5.2*

The design value of the shear resistance of a concrete section with no shear reinforcement, V_c[BB] is given by;

V_c = $[0.79*\{100*A_s/(b_v*d)\}^{⅓}*(400/d)^{0.25} / \gamma_{mc}]*(b_v*d)*(f_{cu}/25)^{⅓}$

If value of f_{cu} exceeds more than 40, it is taken as 40 in the above formula.

where

$(400/d)^{0.25}$	$\geq$	0.67	(for members without shear reinforcement)
$(400/d)^{0.25}$	$\geq$	1	(for members with shear reinforcement providing a design shear resistance of ≥ 0.4 N/mm^2)

$0.15 \leq 100*A_s/(b_v*d) \leq 3$

where

A_s = area of longitudinal tension reinforcement which continues for a distance at least equal to "d" beyond the section being considered.

γ_{mc} = partial safety factor of concrete in shear strength without shear reinforcement = 1.25[CC]

[A]
[B] *BS 8110-1:1997 3.4.5.4 Table 3.8*
[C] *BS 8110-1:1997 Table 2.2*

IF

$V_{region} \leq 0.5 * V_c$

Where V_{region} = maximum design shear force in the particular region

THEN provide **nominal shear reinforcement** over the full length of the span[D]

IF

$0.5 * V_c < V_{region} \leq (V_c + (0.4 * b_v * d))$

THEN provide **minimum shear reinforcement** over the full length of the span as specified below;

$(A_{sv} / s_v)_{min,shear} \geq (0.4 * b_v) / (0.87\, f_{yv})$

where

A_{sv} = total cross-section of links at the neutral axis, at a section

f_{yv} = characteristic strength of links ($f_{yv} \leq 500$ N/mm^2)

s_v = spacing of the links along the member

IF

$(V_c + (0.4 * b_v * d)) < V_{region} \leq V_{max}$

THEN provide **design shear reinforcement** in the form of links

$(A_{sv} / s_v)_{design,shear} \geq (V_{region} - V_c) / (0.87\, f_{yv} * d)$

[A]
[B]
[C]
[D] *BS 8110-1:1997 Table 3.7*

Minimum Area of Shear Reinforcement (Beams: BS 8110)

The minimum diameter of link $\phi_{w,min}$ is given by;

For beams without compression reinforcement;

$\phi_{w,min}$ = MAX[6mm, $\phi_{w,min,u}$]

For beams with compression reinforcement;[A]

$\phi_{w,min}$ = MAX[¼ of $\phi_{comp,max}$, 6mm, $\phi_{w,min,u}$]

where

$\phi_{w,min,u}$ = minimum diameter of link specified by user

$\phi_{comp,max}$ = the maximum diameter of the compression reinforcement

The nominal area of shear reinforcement required, $A_{sv,\ nominal}$ is given by

$A_{sv,\ nominal}$ = MAX[As_v, $A_{sv,min,u}$]

where

As_v = actual area of shear reinforcement by considering the spacing of the shear reinforcement along the longitudinal axis of the beam with minimum diameter of the link $\phi_{w,min}$

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

Area of two-legged link is given by;

A_{sv} = $2*(\pi* \phi_{w,min}^2/4)$

[A] *BS 8110-1:1997 3.12.7.1*

Spacing of Shear Reinforcement (Beams: BS 8110)

The longitudinal spacing, s_v between the legs of shear reinforcement is given by;

In case of nominal shear

$s_{v,min,u} \leq s_v \leq s_{v,max,u}$[A]

In case of minimum and design shear

$s_{v,min,u} \leq s_v \leq \text{MIN}(0.75*d,\ s_{v,max,u})$[B]

where

$s_{v,max,u}$ = the maximum longitudinal spacing specified by the user

$s_{v,min,u}$ = the minimum longitudinal spacing specified by the user

If **compression reinforcement** is required for bending, the longitudinal spacing, s_v between the legs of shear reinforcement is given by;[C]

In case of nominal shear

$s_{v,min,u} \leq s_v \leq MIN[12*\phi_{comp,min}, s_{v,max,u}]$

In case of minimum and design shear

$s_{v,min,u} \leq s_v \leq MIN\{MIN[0.75*d, 12*\phi_{comp,min}], s_{v,max,u}\}$

where

$\phi_{comp,min}$ = the minimum diameter of the compression reinforcement

If **torsional reinforcement** is required, spacing between links is given by,

$s_{v,min,u} \leq s_v \leq MIN\{MIN[x_1, 0.5*y_1, 200\ mm], s_{v,max,u}\}$

The transverse spacing, s_t between the legs of shear reinforcement is given by;[D]

s_t ≤ $MIN[d, 300\ mm, s_{t,max,u}]$

where

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

The horizontal spacing between vertical leg of link and longitudinal tension bar should not exceed 150mm.

[A] *BS 8110-1:1997 3.4.5.5*
[B] *BS 8110-1:1997 3.4.5.5*
[C] *BS 8110-1:1997 3.12.7.1*
[D] *BS 8110-1:1997 3.4.5.5*

Design for Torsion (Beams: BS 8110)

The torsional shear stress of **rectangular** sections is given by;[A]

v_t = $2*T / \{h^2_{min}*(h_{max} - (h_{min}/3))\}$

[A] *BS 8110-2:1985 2.4.4.1*

In Tekla Structural Designer, for flanged sections the un-flanged rectangular cross section is considered when checking torsion.

Maximum torsional shear stress (v_t) is given by;[AA]

For sections where $y_1 < 550$ mm

$$v_{tu} = \text{MIN}[0.8*(f_{cu}^{0.5}), 5 \text{ N/mm}^2] * y_1/550$$

IF

$$v_t \leq v_{tu}$$

THEN the torsion design process can proceed.

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

Maximum combined shear stress (shear plus torsion – $v + v_t$) is given by

For sections where $y_1 \geq 550$ mm

$$v_{tu} = \text{MIN}[0.8*(f_{cu}^{0.5}), 5 \text{ N/mm}^2]$$

IF

$$v + v_t \leq v_{tu}$$

THEN the torsion design process can proceed.

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

where

v_{tu} = maximum combined shear stress (shear plus torsion)

y_1 = larger centre-to-centre dimension of rectangular link

v = design shear stress at a cross section

Minimum torsional shear stress of a section is given by;[BB]

$$v_{t,\,min} = \text{MIN}[0.067*(f_{cu}^{0.5}), 0.4 \text{ N/mm}^2]$$

IF

$$v_t < v_{t,\,min}$$

THEN torsion reinforcement **is not required**.

[A] *BS 8110-2:1985 2.4.5*
[B] *BS 8110-2:1985 2.4.6*

Based upon shear and torsional shear stress, reinforcement for Shear and Torsion is calculated as below;[C]

IF

$v \le (v_c + 0.4)$ and $v_t < v_{t,\ min}$

THEN minimum shear reinforcement is provided and no torsion reinforcement.

IF

$v \le (v_c + 0.4)$ and $v_t > v_{t,\ min}$

THEN designed torsion reinforcement is provided not less than minimum shear reinforcement

IF

$v > (v_c + 0.4)$ and $v_t < v_{t,\ min}$

THEN designed shear reinforcement is provided and no torsion reinforcement.

IF

$v > (v_c + 0.4)$ and $v_t > v_{t,\ min}$

THEN designed shear reinforcement and designed torsion reinforcement is provided.

Torsion reinforcement is provided in terms of 2 legged closed stirrups and longitudinal reinforcement.

Area of 2 legged closed links is given by;[D]

$A_{sv} \quad > \quad T^* s_v / 0.8^*x_1^*y_1 (0.87^*f_{yv})$

Spacing of 2 legged closed links is given by

$s_v \quad = \quad \mathrm{MIN}[x_1, 0.5^*y_1 , 200\ \mathrm{mm})$[E]

Area of longitudinal torsion reinforcement is given by;[F]

$A_s \quad > \quad A_{sv}^* f_{yv} (x_1 + y_1) / (s_v^*f_y)$

• This reinforcement is in addition to that required for shear or bending.

The code states that "Longitudinal torsion reinforcement should be distributed evenly round the inside perimeter of the links. The clear distance between these bars should not exceed 300mm and at least four bars, one in each corner of the link, should be used. Additional longitudinal reinforcement required at the level of tension or compression reinforcement may be provided by using larger bars than those required for bending alone."

[A]
[B]
[C] *BS 8110-2:1985 2.4.6 Table 2.4*
[D] *BS 8110-2:1985 2.4.7*
[E] *BS 8110-2:1985 2.4.8*
[F] *BS 8110-2:1985 2.4.7*

Deflection Check (Beams: BS 8110)

The deflection of reinforced concrete beams is not directly calculated and the serviceability of the beam is measured by comparing the calculated limiting basic span/effective depth ratio L/d

The following table gives basic span/effective depth ratio for rectangular and flanged beams for spans up-to 10 m.

Support Conditions	**Rectangular Section**	**Flanged beam with $b_w/b \leq 0.3$**	**Flanged beam with $b_w/b > 0.3$**
Cantilever	7	5.6	Linear interpolated value between 7 and 5.6
Simply Supported	20	16	Linear interpolated value between 20 and 16
Continuous	26	20.8	Linear interpolated value between 26 and 20.8

For spans exceeding 10 m values in the above table are multiplied by (10/Span) except for cantilevers where the design is justified by calculations.[A]

For beams with **tension reinforcement**, the basic values of L/d are multiplied by using the modification factor;

Modification factor[B] (MF_t) = $\text{MIN}\,[0.55 + \{(477 - f_s) / (120*(0.9 + (M/bd^2)))\}, 2.0]$

where

M	=	design ultimate moment at the centre of the span or, for a cantilever, at the support
b	=	width or effective width of the section or flange in the compression zone
d	=	effective depth of the tension reinforcement
f_s	=	the design service stress in the tension reinforcement[C]
	=	$5/8*(f_y*A_{s\ req}) / A_{s\ prov}*1/\beta_b$

NOTE: As pointed out in Reynolds RC Designers Handbook the term 5/8 which is applicable for γ_{ms} = 1.15 is given incorrectly as 2/3 in BS 8110 which is applicable to γ_{ms} = 1.05

f_y	=	characteristic strength of reinforcement
$A_{s\ req}$	=	area of tension reinforcement required at mid-span to resist the moment due to design ultimate loads (at support for cantilevers)
$A_{s\ prov}$	=	area of tension reinforcement provided at mid-span (at support for cantilevers)
β_b	=	ratio of moments, after and before redistribution

NOTE: As redistribution is not implemented, β_b = 1.

For beams with **compression reinforcement**, the basic values of L/d is multiplied by using the modification factor;[D]

Modification factor (MF_c)	=	$MIN[\{1 + ((100*A_s'_{prov})/(b*d))\} / \{ 3 + (100*A_s'_{prov})/(b*d)\}, 1.5]$

where

$A_s'_{prov}$	=	provided area of compression reinforcement

[A] *BS 8110-1:1997 3.4.6.4*
[B] *BS 8110-1:1997 3.4.6.5 Table 3.10 equation 7*
[C] *BS 8110-1:1997 3.4.6.5 Table 3.10 equation 8*
[D] *BS 8110-1:1997 3.4.6.6 Table 3.11 equation 9*

Column Design to BS 8110

Limitations and Exclusions (Columns: BS 8110)

The provisions of clause 3.8 are applied to all members that have been defined as columns. (Compression members whose greater overall dimension exceeds four times their smaller dimension should be defined as walls.)

The following general exclusions also apply:

- Seismic loading and 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: BS 8110)

Concrete

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

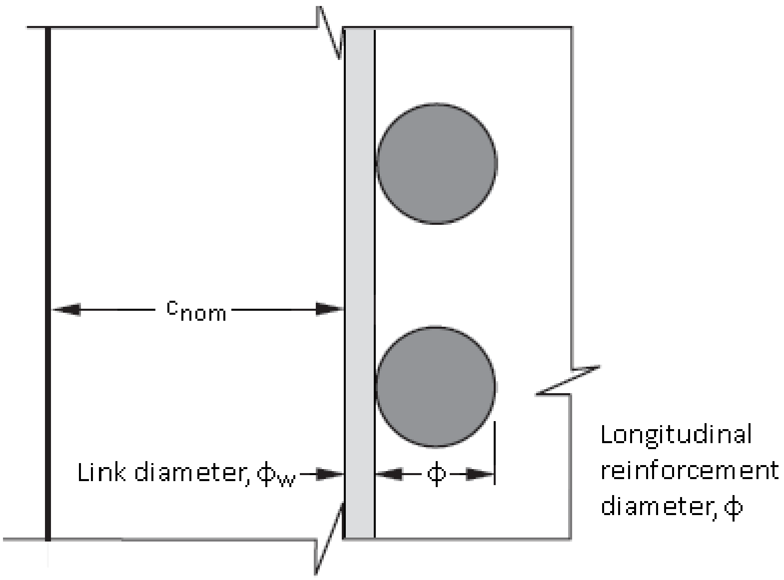
Reinforcement

The reinforcement options are:

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

Cover to Reinforcement (Columns: BS 8110)

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 and maximum nominal aggregate size, h_{agg} (specified in **Design Options > Column > General Parameters**).

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

Design Parameters for Longitudinal Bars (Columns: BS 8110)

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 Spacing

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

$s_{cl,min} \geq \mathrm{MAX}[h_{agg} + 5\text{mm}, s_{cl,u,min}]$

where

h_{agg} = maximum size of coarse aggregate

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

IF

$\phi > h_{agg} + 5\text{mm}$,

THEN

$s_{cl,min} > \phi$

where

ϕ = the maximum diameter of bar

Minimum Longitudinal Total Steel Area

If $N \geq 0$ (i.e. compression)

$A_{sl,min} = 0.4\% * \text{column area}$

Else

$A_{sl,min} = 0.8\% * \text{column area}$ (For $f_y = 250$ N/mm2)

$A_{sl,min} = 0.45\% * \text{column area}$ (For $f_y = 500$ N/mm2)

Maximum Longitudinal Total Steel Area

$A_{sl,max}$ = 8% * column area

1. BS 8110-1:1997 3.12.11.1

Ultimate Axial Load Limit (Columns: BS 8110)

The ultimate axial load carrying capacity N, for **a** column may is obtained as below:

N = $0.4*f_{cu}*A_c + 0.75*A_{sc}*f_y$ [A]

Where

A_c = net cross-sectional area of concrete in a column

f_{cu} = characteristic strength of concrete

A_{sc} = area of vertical reinforcement in a column

f_y = characteristic strength of reinforcement

[A] *BS 8110-1:1997 equation 38*

Effective Length Calculations (Columns: BS 8110)

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_e is calculated automatically - you also have the ability to override the calculated value.

l_e = $\beta*l_o$ [AA]

where

l_o = clear height between end restraints

β = effective length factor

[A] *BS 8110-1:1997 3.8.1.6.1 equation 30*

The value of β may is obtained from the following equations:

For braced columns:

β = $\text{MIN}\ [(0.7 + 0.05*(\alpha_{c,1} + \alpha_{c,2})), (0.85 + 0.05*\alpha_{c,min}), 1.0]$[B]

For unbraced columns:

β = $\text{MIN}\ [(1.0 + 0.15*(\alpha_{c,1} + \alpha_{c,2})), (2.0 + 0.3*\alpha_{c,min})]$[C]

where

$\alpha_{c,1}$ = ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at the lower end of a column

$\alpha_{c,2}$ = ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at the upper end of a column

$\alpha_{c,min}$ = $\text{MIN}\ [\alpha_{c,1}, \alpha_{c,2}]$

[A]
[B] *BS 8110-2:1985 2.5.5*
[C] *BS 8110-2:1985 2.5.6*

In specific cases of relative stiffness the following simplifying assumptions may be used:

a) Flat slab construction – The beam stiffness is based on an equivalent beam of the width and thickness of the slab forming the column strip.

b) Simply supported beams framing into a column - α_c is taken as 10

c) Connection between column and base designed to resist only nominal moments - α_c is taken as 5
(e.g when the base of the column is pinned)

d) Connection between column and base designed to resist column moments - α_c is taken as 1.00

Column Stack Classification (Columns: BS 8110)

Slenderness ratio

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

λ_x = l_{ex} / h

λ_y = l_{ey} / b

where

λ_x	=	slenderness ratio about major axis (x axis)
λ_y	=	slenderness ratio about minor axis (y axis)
h	=	larger dimension of the column
b	=	smaller dimension of the column

For a braced column[A]

l_{ex}	=	effective height in respect of major axis (y axis)
l_{ey}	=	effective height in respect of minor axis (z axis)

IF λ_x and $\lambda_y < 15$

Column is considered as short

Else

Column is considered as slender

For an unbraced column[B]

IF λ_x and $\lambda_y < 10$

Column is considered as short

Else

Column is considered as slender

If column is slender in any direction the additional moments will be induced by the lateral deflection of the loaded column.

[A] *BS 8110-1:1997 3.8.1.3*
[B] *BS 8110-1:1997 3.8.1.3*

limiting slenderness ratio

The limiting value of slenderness ratio λ_{lim} is calculated as below:

For a braced or unbraced columns[A]

l_0 /b ≤ 60

For an unbraced column, if in any given plane one end is unrestrained (e.g a cantilever column)[B]

l_0 /b ≤ MIN[60, 100*b / h]

Where

l_0 = clear height between end restraints

If the slenderness ratio (l_0/b) exceeds the limiting value calculated above then the column design process FAILS.

[A] *BS 8110-1:1997 3.8.1.7*
[B] *BS 8110-1:1997 3.8.1.8*

Design Moment Calculations (Columns: BS 8110)

Minimum Eccentricity

Even though only axial load is considered for the design of column, an allowance must be made for the axial load acting at a nominal eccentricity.

At no section in a column should the design moment be taken as less than that produced by considering the design ultimate axial load as acting at a minimum eccentricity e_{min}

The minimum eccentricity e_{min} is calculated as below:[A]

$e_{min,x}$ = MIN [0.05*h, 20mm]

$e_{min,y}$ = MIN [0.05*b, 20mm]

Where

$e_{min,x}$ = minimum eccentricity along major axis (x axis)

$e_{min,y}$ = minimum eccentricity along minor axis (y axis)

The design moment due to minimum eccentricity is calculated as below:

$M_{x,emin,x}$ = minimum eccentricity moment about major axis (x axis)

= N*$e_{min,x}$

$M_{y,emin,y}$ = minimum eccentricity moment about minor axis (y axis)

= $N*e_{min,y}$

[A] *BS 8110-1:1997 3.8.2.4*

Short columns

The design ultimate moment of **braced short** or **unbraced short** column at top, bottom and mid-fifth span of column is obtained as below:

M_x = MAX [$M_{x,analysis}$, $M_{x,emin,x}$]

M_y = MAX [$M_{y,analysis}$, $M_{y,emin,y}$]

Slender columns

Additional Moments for Slender Columns:

For braced slender column it is necessary to consider **additional moments** induced by the lateral deflection of the loaded column.

The additional moment are obtained as below:

M_{add} = $N*\alpha_u$ [A]

Where

α_u = deflection of a rectangular or circular column at ULS

α_u = β_a*K*h If column bends about major axis (x axis)

α_u = β_a*K*b If column bends about minor axis (y axis)

Where

β_a = $(1/2000)*(l_e/b')^2$ [B]

b' = smaller dimension of column, except for biaxial bending

K = reduction factor that corrects the deflection to allow for the influence of axial load

K = MIN [$(N_{uz} – N)/(N_{uz} – N_{bal})$, 1] [C]

Where

N_{uz} = design ultimate capacity of a section when subjected to axial load only

= $0.45*f_{cu}*A_c + 0.87*f_y*A_{sc}$

N_{bal} = design axial load capacity of a balanced section

= $0.25 * f_{cu} * b * h$ for symmetrically –reinforced rectangular section[D]

= $0.196 * f_{cu} * d^2$ for circular section[E]

Note: Value of N_{bal} for all section shapes can be correctly obtained by interaction curve. Values stated above for rectangular and circular section can be used for checking guidance only.

Where

A_c = net cross-sectional area of concrete in a column

A_{sc} = area of vertical reinforcement in a column

f_{cu} = characteristic strength of concrete

f_y = characteristic strength of reinforcement

[A] *BS 8110-1:1997 equation 35*
[B] *BS 8110-1:1997 equation 34*
[C] *BS 8110-1:1997 equation 33*
[D] *BS 8110-1:1997 3.8.1.1*
[E] *Reynolds's Reinforced Concrete Designer's Handbook - 11th Edition, Section 3.22*

Design for Combined Axial and Bending (Columns: BS 8110)

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:

$\surd(M^2 major) + (M^2 minor) / \surd(M^2 major,res) + (M^2 minor,res) \leq 1.0$

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

Design for Shear (Columns: BS 8110)

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**.

Maximum Span Region Shear Link Spacing

$\varphi_{w,max}$ = MIN[12 * smallest compression bar, user defined maximum link spacing]

Design of Column Shear Reinforcement

Column shear reinforcement design is described as below:

A) Calculate Design Shear Stress:

The design shear stress is calculated as follows (to be done parallel to each axis):

v_i = $V_i / A_{v,i}$

Where

V_i = highest shear force in the stack in this direction,

A_{vi} = shear area parallel to the axis *i*.

B) Check Maximum Shear Capacity:

The maximum allowable shear stress on the section is given by,[A]

v_{max} = MIN [0.8 * $(f_{cu})^{0.5}$, 5 N/mm^2]

IF

v_i ≤ v_{max} (for each direction)

Then the section size is adequate for shear, and the links can be designed or checked.

ELSE the section size must be increased.

[A] *BS 8110-1:1997 3.4.5.2*

C) Check Section Nominal Shear Capacity

It must then be checked whether the nominal shear capacity of the concrete alone is enough to resist the shear force applied. If so, then only nominal links will be used; otherwise the links will be designed to resist shear.

v_c = design concrete shear stress with no shear reinforcement

v_c' = design concrete shear stress

IF
$v_i \le v_c'$ <u>and</u> $M/N \le 0.6 * h$[1]

Then the nominal confinement reinforcement $[(A_{sw} / s)_{nominal}]$ is adequate.

ELSE shear reinforcement is provided as described below.

D) Nominal Shear Capacity Not Adequate ($v > v_{c'}$)

IF

$v_i \le v_c' + 0.4$

Then provide minimum shear reinforcement as specified below,

$(A_{sw} / s)_i$ = MAX $[0.4 * b_{w,i} / (0.87 * f_{yv}), (A_{sw} / s)_{nominal}]$[A]

Else

$(A_{sw} / s)_i$ = MAX $[\{(v_i - v_c') * b_{w,i} / (0.87 * f_{yv})\}, (A_{sw} / s)_{nominal}]$[B]

Where

A_{sw} = area of link provided,

s = required spacing between link centres,

$b_{w,i}$ = effective breadth of the section in direction *i*,

f_{yv} = characteristic strength of links (not to be taken more than 500 N/mm^2)[C]

$(A_{sw} / s)_{nominal}$ = nominal confinement reinforcement at full span

[A] *BS 8110-1:1997 Table 3.7*
[B] *BS 8110-1:1997 Table 3.7*
[C] *BS 8110-1:1997 3.4.5.1*

1. BS 8110-1:1997 3.8.4.6

Wall Design to BS 8110

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: BS 8110)

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,
- Multi-stack reinforcement lifts.

Materials (Walls: BS 8110)

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: BS 8110)

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.

These values are then checked against the nominal limiting cover, $c_{nom,lim}$ which depends on the diameter of the reinforcement and maximum nominal aggregate size, h_{agg} (specified in **Design Options > Wall > General Parameters**).

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

Vertical reinforcement (Walls: BS 8110)

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.

Plain Wall Check

Before placing vertical reinforcement in the wall, the following checks are performed to determine if the given section of wall can act potentially as a plain wall or not.

a) Check for stresses at the corner of a wall

If there are no tensile stresses developed in the wall it is in compression throughout and check passes.

b) Check for bending utilization

Check the bending utilization is less than 10%

i.e $A_{sc,\ required}$ = 0.1*0.0025* A_c = 0.00025* A_c (For f_y=500 N/mm^2)

$A_{sc,\ required}$ = 0.1*0.003* A_c = 0.0003* A_c (For f_y=250 N/mm^2)

The figure 10% stated above is not code dependent value, it is a matter of engineering judgment.

c) Check for limiting axial load

Maximum design load per unit length of a plain wall is assessed from the following equations:

i) Stocky braced plain walls –

$n_w \le 0.3*(h - 2*e_y)*f_{cu}$ [1]

Where

n_w = maximum design ultimate load per unit length of plain wall
h = thickness of wall
f_{cu} = characteristic strength of concrete
e_y = resultant eccentricity of load at right angles to the plane of the wall

$e_{y,min}$ = minimum resultant eccentricity of load at right angles to the plane of the wall = 0.05*h

$e_{y,max}$ = maximum resultant eccentricity of load at right angles to the plane of the wall = 0.5*h

ii) Slender braced plain walls –

n_w = MIN [$0.3*(h - 2*e_y)*f_{cu}$, $0.3*(h - 1.2*e_y - 2*e_a)*f_{cu}$ [2]]

Where

e_a = additional eccentricity due to deflections = $(l_e^2/h)*(1/2500)$
l_e = effective height of wall

iii) Stocky and slender un-braced plain walls –

n_w = MIN [$0.3*(h - 2*e_{y,1})*f_{cu}$[3] , $0.3*\{h - 2*(e_{y,2} + e_a)\}*f_{cu}$[4]]

Where

$e_{y,1}$ = resultant eccentricity at the top of wall
$e_{y,2}$ = resultant eccentricity at the bottom of wall

d) Check for shear

Shear design procedure of plain walls is described as below:[5]

IF $V < 0.25*\, n_w$ or $v_{avg} < 0.45$ N/mm^2

Shear reinforcement is not required

ELSE

It is not a valid plain wall.

Where
V = horizontal design shear force due to ultimate loads
v_{avg} = average design shear stress = V/A_v
A_v = shear area

IF all the checks stated above (a, b, c and d) pass, the given section of wall is classified as plain wall.

Reinforcement area for a Plain wall

The minimum vertical reinforcement area in a plain wall can be given as[6]

$A_{sc,min} = \rho_{v,lim,min}*A_c = 0.003*\, A_c$ (For f_y=250 N/mm^2) = $0.0025*\, A_c$ (For f_y=500 N/mm^2)

For intermediate values of fy, linear interpolation is applicable.

Reinforcement area for a RC wall

The following parameters control the vertical reinforcement area of RC wall,

Limiting minimum ratio of vertical reinforcement area to gross concrete area, $\rho_{v,lim,min}$
Limiting maximum ratio of vertical reinforcement area to gross concrete area, $\rho_{v,lim,max}$

The recommended values are:[7]

IF $n_w \geq 0$ (i.e compression)

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,\,lim,min}*A_c = 0.004*A_c$
Total maximum area of vertical reinforcement, $A_{sc,max} = \rho_{v,lim,max}*A_c = 0.04*A_c$
Where
A_c = Gross area of the concrete wall
n_w = The total design axial load on the wall due to design ultimate loads

Where 2 layers are specified distributed equally to each face, this is a minimum of $0.002*A_c$ placed at each face.

ELSE

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,\ lim,min}*A_c$

$= 0.008*\ A_c$ (For f_y=250 N/mm^2)

$= 0.0045*\ A_c$ (For f_y=500 N/mm^2)

In this case reinforcement should be placed in 2 layers.

For intermediate values of fy, linear interpolation is applicable.

Total maximum area of vertical reinforcement, $A_{sc,max} = \rho_{v,lim,max}*A_c = 0.04*\ A_c$

Spacing of vertical loose bars

Limiting minimum clear horizontal spacing of the vertical bars, $s_{cl,\ v,min}$ is given by, [8]

$s_{cl,v,min} = \text{MAX}\ [h_{agg} + 5\text{mm},\ s_{cl,u,v,min}\ ]$

Where

h_{agg} = maximum size of coarse aggregate

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

$s_{cl,v,min}$ = minimum clear horizontal distance between bars

IF

$\phi > h_{agg} + 5\text{mm}$,

THEN

$s_{cl,v,min} > \phi$

Where

ϕ = maximum diameter of vertical bar

You are also able to specify a limit for maximum spacing of vertical bars in the wall, $s_{cl,v,max}$, although this is not a BS code requirement.

If transverse link reinforcement is required, then there are further restrictions:[9]
All vertical compression bars should be enclosed by a link. No bar should be further than 200 mm from a restrained bar with an included angle of not more than 90^0.

Check for vertical reinforcement when wall in tension

For RC wall –

In the wall properties a "Reinforcement layers" option is present in which user can set 1 or 2 layers.
If a single layer has been selected then it is necessary to check if the axial load acting on the wall is tension or compression.

IF $n_w \geq 0$ (i.e compression)

Then, vertical reinforcement can be placed in single layer.

ELSE

Vertical reinforcement to be placed in double layers.[10]

If the option of double layers have been selected then it is not necessary to check if the axial load on wall is tension or compression.

1. BS 8110-1:1997 equation 43
2. BS 8110-1:1997 equation 44
3. BS 8110-1:1997 equation 45
4. BS 8110-1:1997 equation 46
5. BS 8110-1:1997 cl 3.9.4.18
6. BS 8110-1:1997 cl 3.9.4.19
7. BS 8110-1:1997 Table 3.25
8. BS 8110-1:1997 cl 3.12.11.1
9. BS 8110-1:1997 cl 3.12.7.5
10. BS 8110-1:1997 cl 3.9.3.5

Horizontal reinforcement (Walls: BS 8110)

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

Reinforcement area for a RC wall

The following parameters control the horizontal reinforcement area of RC wall,

Limiting minimum ratio of horizontal reinforcement area to gross concrete area, $\rho_{h,lim,min}$
Limiting maximum ratio of horizontal reinforcement area to gross concrete area, $\rho_{h,lim,max}$

Where the main vertical reinforcement is used to resist compression and does not exceed 2% of the concrete area, at least the following percentage of horizontal reinforcement should be provided, depending upon the characteristic strength of reinforcement.[1]

Minimum area of horizontal reinforcement, $A_{sh,min} = \rho_{h,\ lim,min} * A_c$
$= 0.003 * A_c$ (For $f_y = 250\ N/mm^2$)
$= 0.0025 * A_c$ (For $f_y = 500\ N/mm^2$)

Where
A_c = Gross area of the concrete wall
f_y = Characteristics strength of reinforcement.

For intermediate values of fy, linear interpolation is applicable.

IF

$A_{sc} > 0.02*A_c$

THEN

Links are provided as per section Link/Confinement Reinforcement in addition to the above requirement.[2]

Although not a BS code requirement, an additional check is performed for the maximum area of horizontal reinforcement:

$A_{sc,maxsh,max} = \rho_{h,lim,max}*A_c = 0.04*A_c$

Diameter of horizontal bar

Minimum diameter of horizontal bar, $\phi_{h,min}$ = MAX [$0.25*\phi_v$, $\phi_{h,min,u}$, 6 mm][3]

Where

ϕ_v = maximum diameter of vertical bar

$\phi_{h,min,u}$ = minimum diameter of link specified by user

Spacing of horizontal loose bars

Limiting minimum clear vertical spacing of the horizontal bars, $s_{cl,h,min}$ is given by,[4]

$s_{cl,h,min}$ = MAX [h_{agg} + 5mm, $s_{cl,u,h,min}$]

Where

h_{agg} = maximum size of coarse aggregate

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

$s_{cl,h,min}$ = minimum clear vertical distance between bars

IF

$\phi > h_{agg} + 5mm$,

THEN

$s_{cl,h,min} > \phi$

Where

ϕ = diameter of horizontal bar

Although not a BS code requirement, an additional check is performed (as per the Eurocode) for the maximum spacing of horizontal bars in wall.

Limiting maximum horizontal spacing, $s_{cl,h,max}$ = MIN [3*h , 400mm]

Reinforcement area for a Plain wall

For a plain wall,

$0 \leq \rho_v < 0.004$

The same bar limit checks that are performed for an RC wall are also performed for a plain wall.

1. BS 8110-1:1997 cl 3.12.7.4
2. BS 8110-1:1997 cl 3.12.7.5
3. BS 8110-1:1997 cl 3.12.7.4
4. BS 8110-1:1997 cl 3.12.11.1

Link/Confinement Reinforcement (Walls: BS 8110)

The confinement steel reinforcement provided in the form of link contributes to the shear resistance.

It must be provided when the vertical area of reinforcement in the 2 faces exceeds $0.02*A_c$[1]

Reinforcement area

There is no code requirement to satisfy.

Diameter of confinement reinforcement

Minimum diameter of link, $\phi_{w,min}$ = MAX [$0.25*\phi_v$, 6 mm , $\phi_{w,min,u}$][2]

Where
ϕ_v = maximum diameter of vertical bar
$\phi_{w,min,u}$ = minimum diameter of link specified by user

It is expected that the maximum diameter of link, $\phi_{w,max}$ will not be greater than the vertical bar diameter.

$$\phi_{v,max} \leq \phi_v$$

Spacing of confinement reinforcement

The spacing of links, s_v in horizontal direction is given by;[3]

$s_{v,min,u} \leq s_v \leq$ MIN [2*h, 200mm]

No vertical bar should be further than 200mm from a restrained bar, at which a link passes round the bar with an included angle of not more than 90^0

The spacing of links, s_v in the vertical direction is given by;

$s_{v,min,u} \leq s_v \leq$ MIN [2*h , $16*\phi_v$, $s_{v,max,u}$]

Where
$s_{v,min,u}$ = the minimum link spacing specified by user

$s_{v,max,u}$ = the maximum link spacing specified by user
ϕ_v = maximum diameter of vertical bar

You are given control over these values by specifying upper and lower limits in **Design Options > Wall > Reinforcement Layout**.

1. BS 8110-1:1997 cl 3.12.7.5
2. BS 8100-1:1997 cl 3.12.7.5
3. BS 8100-1:1997 cl 3.12.7.5

Ultimate Axial Load Limit (Walls: BS 8110)

The axial resistance calculations for walls are the same as for columns. (See Column Design -).

Effective Length and Slenderness Calculations (Walls: BS 8110)

The slenderness calculations for walls are generally the same as for columns. (See Column Design - and).

Design Moment Calculations (Walls: BS 8110)

For each combination, a set forces are returned from one or more sets of analyses, in the same way as for columns. For details, see: *Column Design to BS8110>* .

Design for Combined Axial and Bending (Walls: BS 8110)

These calculations are the same whether the design element is a column or a wall.
See Column Design -).

Design for Shear (Walls: BS 8110)

The shear design calculations are the same whether the design element is a column or a wall.
See Column Design -).

Slab Design to BS 8110

Limitations and Exclusions (Slabs: BS 8110)

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: BS 8110)

Concrete

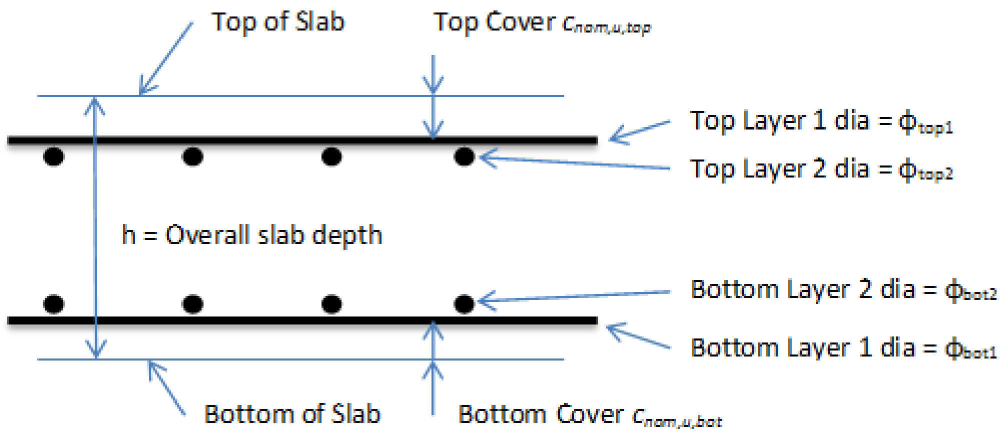
Only normal weight is included in the current release. (Lightweight concrete is excluded).

Reinforcement

The reinforcement options are:

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

Reinforcement Parameters (Slabs: BS 8110)



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 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: BS 8110)

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.

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

Limiting Reinforcement Parameters (Slabs: BS 8110)

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: BS 8110)

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} = 8mm (default)

Maximum Loose Bar Diameter

For “flat slab”:

φ_{min} = 25mm (default)

For “beam and slab”:

φ_{min} = 16mm (default)

Minimum Clear Spacing (Slabs: BS 8110)

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

$s_{cl,min} \geq MAX[h_{agg} + 5mm, s_{cl,u,min}]$

where

h_{agg} =maximum size of coarse aggregate (specified in **Design Options > Slab > General Parameters**).

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

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

$s_{cl,min} \geq 2h_{agg} /3$

If

$\varphi > h_{agg} + 5mm$,

Then

$s_{cl,min} > \varphi$

Where

φ = maximum bar diameter

Maximum Spacing of Tension Bars (all slabs) (Slabs: BS 8110)

To control cracking in slabs, maximum values for clear spacing between bars are set out in BS8110: Part 1, clause 3.12.11.2.7.

Bar spacings are checked against the user defined maximum spacings specified in **Design Options > Slab > Reinforcement Layout**.

Minimum Area of Reinforcement (Slabs: BS 8110)

The minimum area of longitudinal tension reinforcement, $A_{s,min}$, is given by;[1]

$A_{s,min} \geq MAX[0.0024 * b* h$ (for $f_y = 250N/mm^2$)

$A_{s,min} \geq MAX[0.0013 * b* h$ (for $f_y = 500N/mm^2$)

This minimum steel should be provided in both directions

Where,

b = breadth of section (taken as 1000mm)

f_y = characteristic strength of reinforcement

h = overall depth of the cross-section of a reinforced member

1. BS 8110-1:1997 3.12.5.3, Table 3.25

Maximum Area of Reinforcement (Slabs: BS 8110)

The maximum area of longitudinal tension reinforcement, $A_{s, max}$, is given by[1]

$A_{s, max} \leq 0.04 * A_c$

where

A_c = the cross sectional area of the slab design section

= h * b (consider b as 1000m)

1. BS 8110-1:1997 3.12.6.1

Basic Cross Section Design (Slabs: BS 8110)

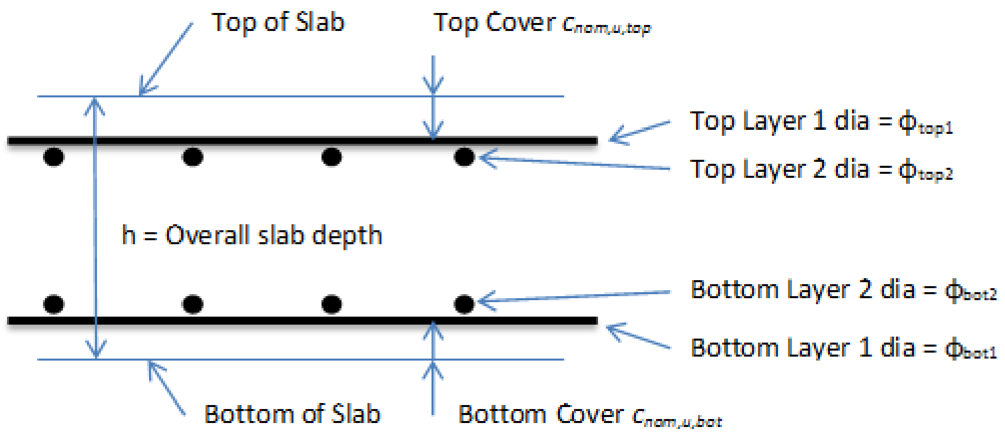
Regardless of whether design is being carried out for a slab panel or a patch, a unit width of slab is always designed for a known design force.

h = overall slab depth

b = unit width

for BS 8110 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: BS 8110)

For the design moment under consideration the appropriate effective depths d and d_2 are calculated. The design is then basically the same as employed for the design of rectangular beams

Calculate the value of *K* from;

$K = M_{Ed}/(f_{cu}*b*d^2)$

b =)1000mm

where

MEd = design ultimate moment

fcu = characteristic strength of concrete

b = width or effective width of the section

The limiting value of K, known as *K'* is:

K' = 0.156

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

Calculate the lever arm, *z* from;

$z = MIN(d*[0.5 + (0.25 – (K/(0.9))^{0.5}], 0.95*d)$

Calculate the depth to the neutral axis, x from;

x =(d-z)/0.45

The area of tension reinforcement required is then given by;

$A_{st,reqd} = M_{Ed}/(0.87 * f_y*z)$

IF K > K' THEN compression reinforcement is required. This is beyond scope for the current release.

Deflection Check (Slabs: BS 8110)

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: BS8110)*

Punching Shear Checks (Slabs: BS 8110)

•

Punching shear limitations and assumptions (Slabs: BS 8110)

Arrangement of reinforcement and related checks (Slabs: BS 8110)

When working to the BS 8110 head code, all the relevant calculations are performed as to evaluate both the strength of the slab under punching shear and estimate the required amount of punching shear reinforcement if its use is found to be appropriate. However the arrangement of this type of reinforcement around the loaded perimeter and any related checks are still beyond scope at this stage.

Applicability of wall punching checks (Slabs: BS 8110)

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: BS 8110)

BS 8110 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: BS 8110)

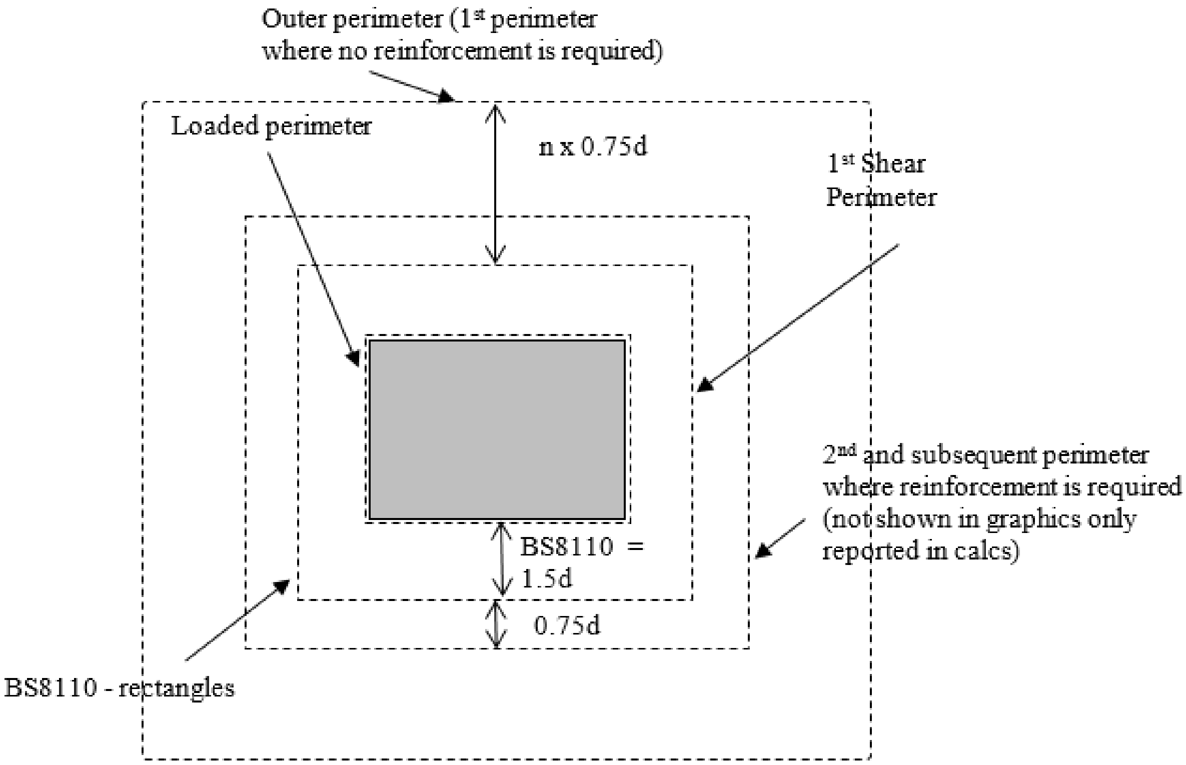
BS 8110 only provides specific design guidance for rectangular columns with this guidance being further limited, for the case of edge and corner columns, to those cases where the edge(s) of the slab coincide with the edge(s) of the column. In the program the equations for other column shapes and scenarios are therefore obtained by modifying the equations. 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.

Overlapping Perimeters (Slabs: BS 8110)

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: BS 8110)



The following definitions apply

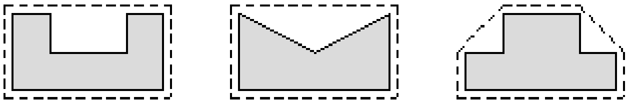
- **Loaded perimeter** - the perimeter of the loaded area eg face of column or wall or drop panel
- **1st Shear Perimeter** - the first punching shear perimeter - at 1.5d from the loaded perimeter in BS8110
- **2nd, 3rd and subsequent Shear Perimeters** - the subsequent punching shear perimeters - at 0.75d intervals out from the 1st shear perimeter where reinforcement is required
- **Outer perimeter** - the first shear perimeter in the above sequence (beyond the 1st perimeter) at which the punching shear check passes with no reinforcement requirement
- **Average effective depth d to the tension reinforcement** d = (dy + dz) / 2 where dy and dz are the effective depths in the two orthogonal directions. There is a value of d for top steel and a different value for bottom steel. Note this definition changes in the presence of a drop panel. This information is only available if the reinforcement is known in each direction.
- **Drop or Drop panel** - a thickening of the slab (either up or down or both) local to a column in a slab

Length of the loaded perimeter u0 (Slabs: BS 8110)

Loaded perimeter for Columns

The length of the loaded perimeter at the column face is calculated as determined below.

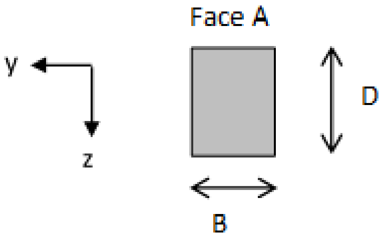
Note - for columns which have a re-entrant corner, ie where an internal angle is greater than 180 degrees, the length of a side and the slab/column interface is adjusted as indicated in the sketches below with the perimeter taken as the shortest distance around the column.



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)



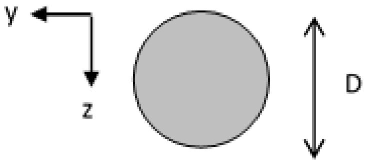
$u_0 = 2 \times (D + B)$

Bounding rectangle $D_{bound} = D$

Bounding rectangle $B_{bound} = B$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

Circular (D)



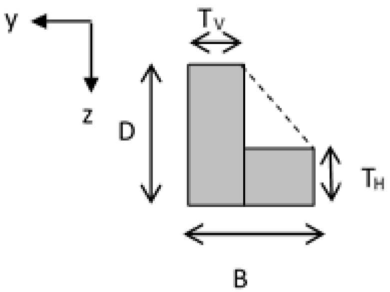
$u_0 = \pi \times D$

Bounding rectangle $D_{bound} = D$

Bounding rectangle $B_{bound} = D$

Bounding rectangle perimeter - $u_{0bound} = 2 \times (D_{bound} + B_{bound})$

L section (D, B, T_H and T_V)



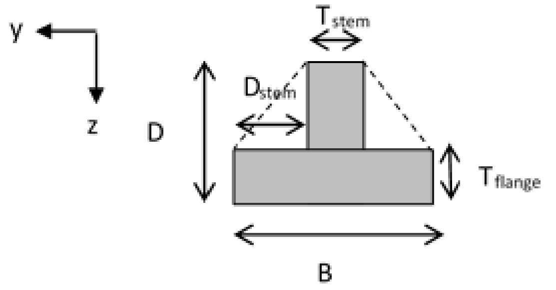
$u_0 = D + B + T_V + T_H + \text{Sqrt}((B-T_V)^2 + (D-T_H)^2)$

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})



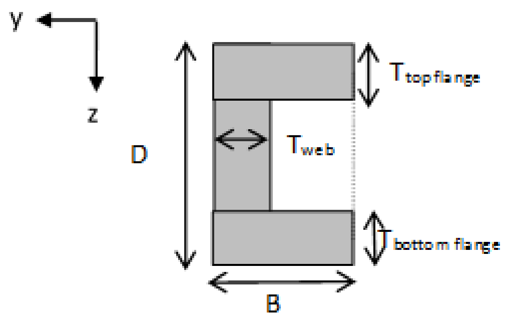
$u_0 = B + 2xT_{flange} + T_{stem} + \text{Sqrt}((D_{stem}{}^2+(D-T_{stem})^2) + \text{Sqrt}((B-T_{stem}-D_{stem})^2+(D-T_{stem})^2)$

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 $T_{bottom\ flange}$)



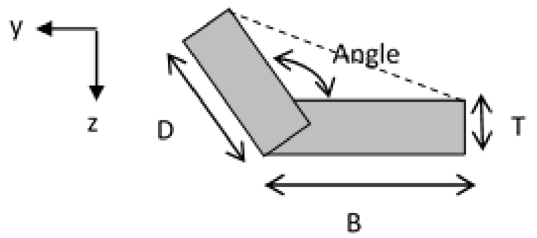
$u_0 = 2 \times (B + D)$

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)



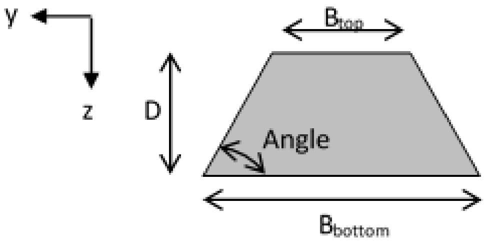
$u_0 = B + D + 2 \times T + \text{Sqrt}((B + D \times \text{Sin}(\text{angle}-90)-T \times \text{Cos}(\text{angle}-90))^2 + (D \times \text{Cos}(90-\text{angle}) - T)^2)$

Bounding rectangle $D_{bound} = D\ \text{Sin}(180-\text{Angle}) \times T\ \text{Cos}(180-\text{Angle})$

Bounding rectangle $B_{bound} = B + D\ \text{Cos}(180-\text{Angle})$

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

Trapezium (D, B_{bottom}, B_{top}, angle)



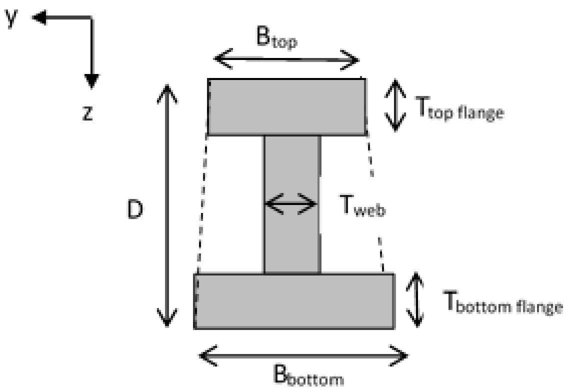
u_0 = B_{top} +B_{bottom} + 2 x D / sin(Angle)

Bounding rectangle D_{bound} = D

Bounding rectangle B_{bound} = max(B_{bottom}, B_{top})

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

I section (D, B_{top}, B_{bottom}, T_{web}, $T_{top\ flange}$, $T_{bottom\ flange}$)



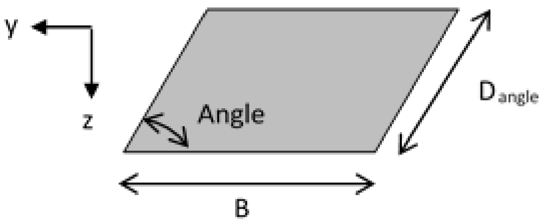
u_0 = B_{bottom} + B_{top} + 2 x Sqrt(((B_{bottom}-B_{top})/2)^2 + D^2) (approx)

Bounding rectangle D_{bound} = D

Bounding rectangle B_{bound} = max(B_{bottom}, B_{top})

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

Parallelogram (D_{angle}, B, angle)



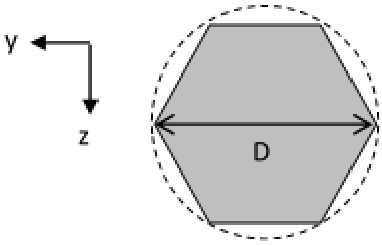
u_0 = 2 x (B + D_{angle})

Bounding rectangle D_{bound} = D x sin(Angle)

Bounding rectangle B_{bound} = B + D_{angle} x cos(Angle)

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

Polygon (D, n) - n >4



u_0 = 2 x n x D/2 x sin (180/n)

Bounding rectangle D_{bound} = D

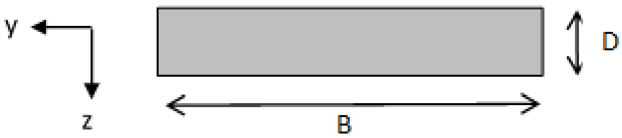
Bounding rectangle B_{bound} = D x cos (180/n)

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

Loaded perimeter for Walls

The length of the loaded perimeter at the wall face may be calculated as determined below.

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})

Loaded perimeter for Point Loads

The length of the loaded perimeter at the point load may be calculated as determined below.

u_0 = 2 x (D_{load} + B_{load})

Bounding rectangle D_{bound} = D_{load}

Bounding rectangle B_{bound} = B_{load}

Bounding rectangle perimeter - u_{0bound} = 2 x (D_{bound} + B_{bound})

Additional Loaded perimeter drops

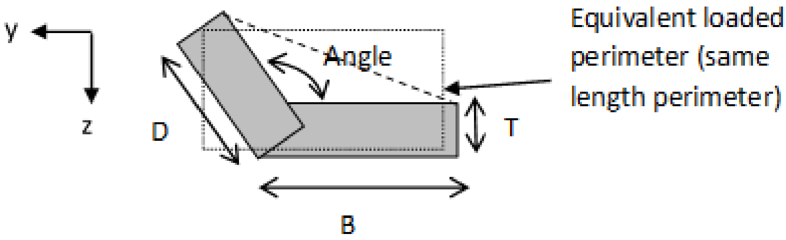
The additional loaded perimeter for a column/wall with a drop is defined by the perimeter of the rectangular drop

u_{0drop} = 2 x B_{drop} x D_{drop}

The equivalent perimeter

For all shapes of column and walls, the equivalent loaded perimeter -

- D_{equiv} = D_{bound} x u_0 / u_{0bound}
- B_{equiv} = B_{bound} x u_0 / u_{0bound}



The equivalent perimeter is used in these situations

- adjustment of the loaded perimeter length/shape u_0 for edge and corner columns/walls
- Reduction in V_t

The shear perimeters

For all shapes of column and walls, the shear perimeters -

- 1st, 2nd and subsequent shear perimeters (each 0.75d out from the previous)
  - D_n = D_{bound} + 3 x d + (n-1) x 1.5 x d
  - B_n = B_{bound} + 3 x d + (n-1) x 1.5 x d
  - u_n = 2 x d + (D_n + B_n)

    Where n is the number of the perimeter

The shear perimeter is used in the design calculations for punching shear for the design shear stress.

Length of the shear perimeter un (Slabs: BS 8110)

Basic shear perimeter without drops

The length of the column/wall basic shear perimeter is the length as determined below.

For all internal column/wall shapes and point loads:

- 1st, 2nd and subsequent shear perimeters (each 0.75d out from the previous)
  - D_n = D_{bound} + 3 x d + (n-1) x 1.5 x d
  - B_n = B_{bound} + 3 x d + (n-1) x 1.5 x d
  - u_n = 2 x d + (D_n + B_n)

    Where n is the number of the perimeter

For all corner column/wall shapes and point loads:

u_n = A + B + C

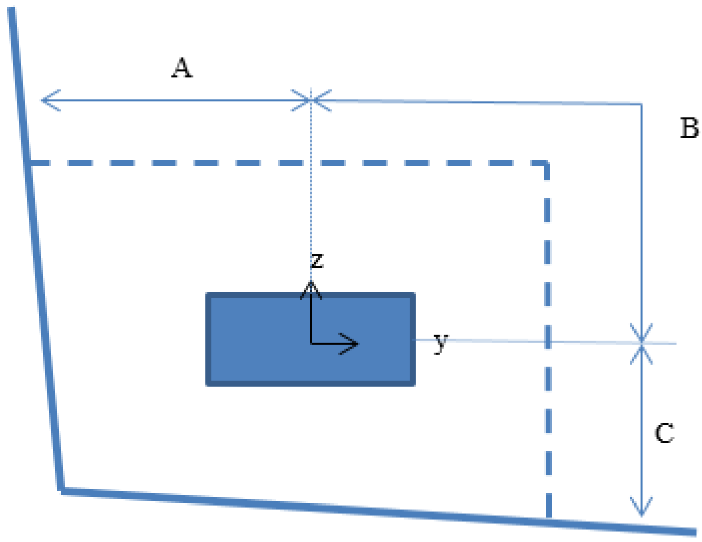
where

for a rectangle

A = dist from centroid to edge along local y

B = $c_1/2 + 3 \times d + c_2/2$

C = dist from centroid to edge along local z



For all edge column/wall shapes and point loads:

$u_n = A + B + C$

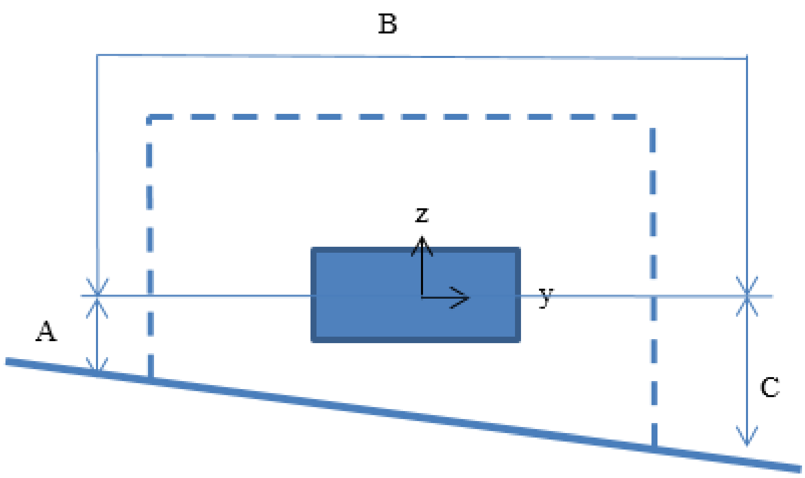
where

For a rectangle

A = dist from centroid to edge along local y or local z

B = $c_1 + 2 \times 3 \times d + c_2$

C = dist from centroid to edge along local y or local z

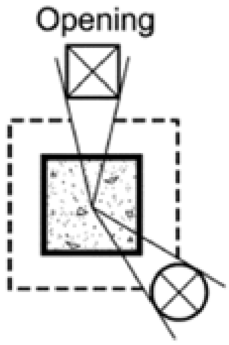


If a shear perimeter passes across a slab edge then only the perimeter length in the slab is counted in u_n.

Note if a slab around a column/wall/point load changes depth, the thinnest slab and its "d" values above is used.

Modification of shear perimeters to take account of slab openings (Slabs: BS 8110)

If any openings are present 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 shear perimeters, u_n, are reduced to take account of the presence of the opening(s) as indicated in the sketch below (fig. 3.18 from BS8110).



User Modification of shear 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:

- Loaded perimeter reduction length (taken off u_0)
- Shear perimeter reduction length (taken off u_n)

User Limit on Vt factor for columns and walls (Slabs: BS 8110)

The 'User factor for Vt' can be used to set a user min value for the factor on V_t

- for internal columns (both axes) = 1.15
- for edge columns moments about axis perpendicular to edge = 1.4

Pad and Strip Base Design to BS 8110

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 BS 8110-Part 1:1997 - Structural use of concrete: Part 1. Code of practice for design and construction.

Checks Performed (Pad and Strip Base: BS 8110)

The checks performed for both directions are:

- Max soil bearing pressure must not exceed allowable bearing pressure.
- Provided steel must be greater than A_s(min) for both vertical directions.
- Provided bar spacing must be inside the limiting spacing
- Provided bar size must be inside the limiting sizes
- Check for bending moment capacity
- Check for shear capacity
- Punching check at column face
- Punching check at 0.25d, 0.5d, 0.75d, 1d,1.25d and 1.5d from column face
- Check for overturning forces - **not in the current release**
- Check for sliding
- Check for uplift

Foundation Bearing Capacity (Pad and Strip Base: BS 8110)

Check for Pad Base Bearing Capacity

Bearing capacity calculations are done by using service (soil) -combinations.

Total base reaction:

$T = F_{swt} + F_{soil} + F_{Gsur} + F_{Qsur} - P$

Moment about X axis:

$M_{x,c} = M_{x,sup} - P^* e_{Py} - h^* F_{y,sup}$

Moment about Y axis:

$M_{y,c} = M_{y,sup} + P^* e_{Px} + h^* F_{x,sup}$

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

A_c = cross section of the column/wall segment

e_{Px} = offset in X direction

e_{Py} = offset in Y direction

ρ_{conc} = density of concrete

ρ_{soil} = density of soil

F_{swt} = $A * h * \rho_{conc}$ = foundation self-weight

F_{soil} = $(A - A_c)*h_{soil}* \rho_{soil}$ = soil self-weight

F_{Gsur} = $(A - A_c)*sc_G$ = Dead load from surcharge

F_{Qsur} = $(A - A_c)*sc_Q$ = Imposed load from surcharge

sc_G = Surcharge in dead load case - user input (kN/m^2)

sc_Q = Surcharge in imposed load case - user input (kN/m^2)

P = = axial load acting on support in SLS

$M_{x,sup}$ = = Moment acting on support around X-axis in SLS

$M_{y,sup}$ = = Moment acting on support around Y-axis in SLS

$F_{x,sup}$ = Horizontal force acting on support X-direction in SLS

$F_{y,sup}$ = Horizontal force acting on support Y-direction in SLS

Eccentricity of base reaction in X-direction:

e_{Tx} = $M_{y,c} / T$

Eccentricity of base reaction in Y-direction:

e_{Ty} = $M_{x,c} / T$

If

$abs(eTx) / Lx + abs(eTy) / Ly \quad \leq \quad 0.167$

Then Base reaction acts within middle third - no loss of contact and:

Pad base pressures:

$q_1 \quad = \quad T/A - 6^* M_{y,c} / (L_x{}^*A) + 6^* M_{x,c} / (L_y{}^*A)$

$q_2 \quad = \quad T/A - 6^* M_{y,c} / (L_x{}^*A) - 6^* M_{x,c} / (L_y{}^*A)$

$q_3 \quad = \quad T/A + 6^* M_{y,c} / (L_x{}^*A) + 6^* M_{x,c} / (L_y{}^*A)$

$q_4 \quad = \quad T/A + 6^* M_{y,c} / (L_x{}^*A) - 6^* M_{x,c} / (L_y{}^*A)$

Max base pressure:

$q_{max} \quad = \quad max\ (q_1, q_2, q_3, q_4)$

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:

If

$abs(eTx) / Lx + abs(eTy) / Ly \quad \leq \quad 0.167$

Then - no loss of contact and

max base pressures for segment:

$q_{max} \quad = \quad T/A + max[- 6^* M_{y,c} / (L_x{}^*A)\ ,\ 6^*M_{y,c} / (L_x{}^*A)]$

Else - loss of contact and

max base pressures for segment:

$q_{max} \quad = \quad 2^*T/[3^* L_y{}^* (L_x /2 - abs(e_{Tx}))]$

where L_y = segment width

Design for Bending (Pad and Strip Base: BS 8110)

Bending design calculations are performed using ULS combinations.

Determination of the Design Moment in Pad Bases

Ultimate base reaction;

$T_u = - P_u$

Ultimate moment around Y-axis in centre of foundation:

$M_{y,c,u} = M_{y,sup,u} + P_u * e_{Px} + h * F_{x,sup,u}$

Ultimate moment around X-axis in centre of foundation:

$M_{x,c,u} = M_{x,sup,u} – P_u * e_{Py} – h * F_{y,sup,u}$

Where

P_u = ultimate axial factored load acting on support – from analysis

$M_{x,sup,u}$ = ultimate moment acting on support around X-axis – from analysis

$M_{y,sup,u}$ = ultimate moment acting on support around Y-axis – from analysis

$F_{x,sup,u}$ = ultimate horizontal force acting on support X-direction – from analysis

$F_{y,sup,u}$ = ultimate horizontal force acting on support Y-direction – from analysis

Eccentricity of ultimate base reaction in X:

$e_{Tx,u} = M_{y,c,u} / T_u$

Eccentricity of ultimate base reaction in Y:

$e_{Ty,u} = - M_{x,c,u} / T_u$

Pad ult. base pressures:

$q_{1,u}$ = $T_u/A – 6* M_{y,c,u} / (L_x*A) +6* M_{x,c,u} / (L_y*A)$

$q_{2,u}$ = $T_u/A – 6* M_{y,c,u} / (L_x*A) - 6* M_{x,c,u} / (L_y*A)$

$q_{3,u}$ = $T_u/A + 6* M_{y,c,u} / (L_x*A) + 6* M_{x,c,u} / (L_y*A)$

$q_{4,u}$ = $T_u/A + 6* M_{y,c,u} / (L_x*A) - 6* M_{x,c,u} / (L_y*A)$

The range and rate of change of base pressure is then determined and the design moment established about each axis.

Determination of the Design Moment in Strip Bases

The principles used in the strip base bending capacity calculations are similar to those for pad foundations. Only the direction X is checked (around Y-axis) using segment widths.

Ultimate base reaction;

$T_u = - P_u$

Ultimate moment around Y-axis in centre of foundation:

$$M_{y,c,u} = M_{y,sup,u} + P_u * e_{Px} + h * F_{x,sup,u}$$

Where

P_u = = ultimate axial factored load acting on support – from analysis

$M_{y,sup,u}$ = ultimate moment acting on support around Y-axis – from analysis

$F_{x,sup,u}$ = ultimate horizontal force acting on support X-direction – from analysis

Eccentricity of ultimate base reaction in X:

$$e_{Tx,u} = M_{y,c,u} / T_u$$

Pad ult. base pressures:

$q_{1,u}$ = $T_u/A - 6 * M_{y,c,u} / (L_x * A)$

$q_{2,u}$ = $T_u/A + 6 * M_{y,c,u} / (L_x * A)$

The range and rate of change of base pressure is then determined and the design moment established around the Y axis.

Bending Capacity Check

The basic design method is identical to that for beams - see: *(Beams: BS8110)*

Checks for Limiting Parameters (Pad and Strip Base: BS 8110)

Limiting reinforcement parameters are specified in **Design Options** > **Foundations** > **Isolated Foundations** > **Reinforcement Layout**

Limits on bar size

The minimum area of longitudinal reinforcement, $A_{s,min}$, is given by;[1]

$A_{s,min}$ ≥ 0.0024*b*h (For $f_y = 250\ N/mm^2$)

$A_{s,min}$ ≥ 0.0013*b*h (For $f_y = 500\ N/mm^2$)

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

$A_{s,max}$ ≤ 0.04*b*h

1. BS 8110-1:1997 3.12.5.3, Table 3.25

2. BS 8110-1:1997 3.12.6.1

Limits on bar spacing

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

$s_{cl,min}$ ≥ MAX[h_{agg} + 5mm, $s_{cl,u,min}$]

where

h_{agg} = maximum size of coarse aggregate

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

When the maximum crack width is limited to 0.3 mm and nominal cover to reinforcement does not exceed 50 mm,

$s_{cl,max}$ ≤ $47000/f_s \leq 300$

where

$s_{cl,max}$ = maximum clear horizontal distance between bars in tension

f_s = the design service stress in the tension reinf. of a member[A]

f_s = $(5 * f_y * A_{s\ req})/(8 * A_{s\ prov})$

$A_{s\ req}$ = required area of tension reinforcement

$A_{s\ prov}$ = provided area of tension reinforcement

[A] *BS 8110-1:1997 equation 8*

1. BS 8110-1:1997 3.12.11.1

Shear Design (Pad and Strip Base: BS 8110)

Pad base shear design check

Design concrete shear stress for each direction[1];

v_c = $0.79*\{100*A_s/(b_v*d)\}^{1/3}*(400/d)^{0.25} * \{\min[\max(f_{cu}, 25\text{MPa}) / 1\text{MPa}, 40] /25\}^{1/3}\text{MPa} / 1.25$

where

$(400/d)^{0.25}$ ≥ 0.67 NOTE: Unit of 400 in this formula is “mm”.

0.15 ≤ $100*A_s/(b_v*d) \leq 3$

A_s = area of longitudinal tension reinforcement per unit width in direction considered

b_v = unit width

d = depth of tension reinforcement in direction considered

Maximum allowable shear stress[2];

v_{max} = $\min(0.8 * \mathrm{sqrt}(f_{cu}), 5N/mm^2)$

where

f_{cu} = characteristic strength of concrete

If;

$v_{su} \leq v_{res} = \min(v_c , v_{max})$

Then the foundation thickness is adequate for shear
Else the shear design check has failed, the foundation thickness is inadequate.

If the thickness is inadequate and the auto-design footing depth option is active then the foundation thickness gets increased.

1. BS 8110-1:1997 3.4.5.4 Table 3.8

2. BS 8110-1:1997 3.4.5.2

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 for Pad Bases (Pad and Strip Base: BS 8110)

Punching shear checks are carried out for pad foundations only.

The punching shear checks for pad bases follow the same basic principle as used for mats.

The main differences between mat and pad base punching shear checks are:

- Checks at multiple perimeters up to 1.5d are required in pad base punching design
- 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: BS 8110)

Checks for overturning forces are beyond scope in the current release of Tekla Structural Designer.

Check for Sliding (Pad and Strip Base: BS 8110)

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.

Resultant Force on foundation:

$H_d = [(F_{x,sup})^2 + (F_{y,sup})^2]^{0.5}$

Resultant Force Angle $\alpha_{Hd} = \tan^{-1} [(F_{y,sup} / F_{x,sup})]$

where

$F_{x,sup}$ = horizontal force acting on support in X-dir. (from analysis)

$F_{y,sup}$ = horizontal force acting on support in Y-dir. (from analysis)

Resistance to sliding due to base friction:

$H_{friction} = [-P + F_{swt}] * \tan\delta$

where

δ = design base friction – user input

Passive pressure coefficient:

$K_p = (1+\sin\Phi') / (1-\sin\Phi')$

where

Φ' = design shear strength of soil – user input

Passive resistance of soil in X direction:

$H_{xpas} = 0.5*K_p*(h^2 +2*h*h_{soil})*L_x* \rho_{soil}$

Passive resistance of soil in Y direction:

$H_{ypas} = 0.5*K_p*(h^2 +2*h*h_{soil})*L_y* \rho_{soil}$

Resultant Passive Resistance:

$H_{res,pas} = abs(H_{xpas}*\cos \alpha_{Hd}) + abs(H_{ypas}*\sin \alpha_{Hd})$

Total resistance to sliding:

$R_{H.d} = (H_{friction} + H_{res,pas}) / 1.5$

If

$R_{H.d} \geq H_d$

The check for stability against sliding passes

Check for Uplift (Pad and Strip Base: BS 8110)

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 BS 8110

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 Capacity (Pile Cap: BS 8110)

Pile capacity passes if:

$R_c \geq F_{char,pn} \geq - R_t$

Where:

R_c = Pile SWL compression capacity

R_t = Pile SWL tension capacity

$F_{char,pn}$ = Pile load

Design for Bending (Pile Cap: BS 8110)

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.

The bending capacity check follows the same basic principle as used for beams.
See: Beam Design - Design for Bending for Rectangular Sections.

Shear Design (Pile Cap: BS 8110)

Shear capacity is calculated at each critical section for both directions (4 checks altogether).

Shear capacity at the critical section[1]:

$v_{c,enh} = [2d / \min(2d , a_v)] * v_c$

where

d = effective depth of reinforcement in direction considered

Maximum shear stress for each direction[2] - side is limited as follows:

v_{max} = min[0.8 $N^{0.5}$/mm * $fcu^{0.5}$, 5 N/mm^2]

One-way shear resistance:

v_{res} = min [$v_{c,enh}$, v_{max}]

Pile cap shear capacity passes if for both sides and both directions:

$v_{su} \leq v_{res}$

where

v_{su} = design shear stress acting on this critical section

1. BS-8110 section 3.4.5.8 and 3.11.4.4

2. BS-8110 section 3.4.5.2

Punching Shear Design (Pile Cap: BS 8110)

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 for Pad Bases

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:
    - $v_{eff,0}$ = b * $F_{pn,max}$ / (u_0*d)

See: Punching Shear Design for Pad Bases

Checks for Limiting Parameters (Pile Cap: BS 8110)

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 $s_{ij} > \min(s_{min}, s_{min,user})$

where

$s_{min,user}$ = user input

s_{min} = perimeter of the pile for non circular friction piles

s_{min} = 3*diameter for circular friction piles

s_{min} = 2*least width of the pile for end bearing piles

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: Pad Base and Strip Footing Design - Checks for Limiting Parameters.

References

1. **British Standards Institution**. *BS 8110-1:1997. Structural use of concrete. Code of practice for design and construction.* **BSI 1997**.

Steel Design - BS 5950

Steel Design to BS 5950

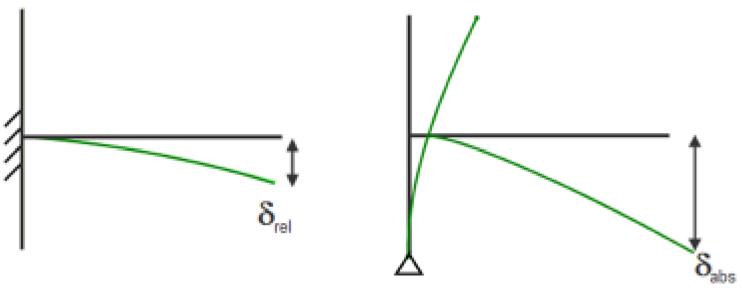
Tekla Structural Designer designs steel members and composite members to a range of international codes. This reference guide specifically describes the design methods applied when the steel design and composite design resistance codes are set as BS 5950-1 and BS 5950-3.1 respectively.

Unless explicitly noted otherwise, all clauses, figures and tables referred to are from BS 5950-1:2000 (Ref. 2); apart from the Composite Beam section, within which references are to BS 5950-3.1:2010 (Ref. 1) unless stated.

Basic Principles (BS 5950)

Deflection checks

Building 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 Design to BS 5950

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Design Method (Beams: BS 5950)

Unless explicitly stated all calculations are in accordance with the relevant sections of BS 5950-1:2000. You may find the handbook and commentary to the Code of Practice published by the Steel Construction Institute useful.

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Steel beam limitations and assumptions (Beams: BS 5950)

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 (that is I- and H-sections), doubly symmetric hollow sections (i.e. SHS, RHS and CHS), and channel sections are fully designed,
- single angle, 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 only analysed 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: BS 5950)

The checks relate to doubly symmetric prismatic sections (that is rolled I- and H-sections), to singly symmetric sections i.e. channel sections and to doubly symmetric hollow sections i.e. SHS, RHS and CHS. Other section types are not currently covered.

The strength checks relate to a particular point on the member and are carried at regular intervals along the member and at *“points of interest”*.

Classification (Beams: BS 5950)

General

The classification of the cross section is in accordance with BS 5950-1: 2000.

Steel beams can be classified as:

- Plastic Class = 1
- Compact Class = 2
- Semi-compact Class = 3
- Slender Class = 4

Class 4 sections are only acceptable for angle and tee sections.

Sections with a Class 3 web can be taken as Class 2 sections (Effective Class 2) providing the cross section is equilibrated to that described in Clause 3.5.6 where the section is given an *“effective”* plastic section modulus, S_{eff}. For rolled I and H sections in the UK, this gives no advantage in pure bending since the web d/t is too small. Hence for beams there is likely to be little advantage in using this approach since the axial loads are generally small, this classification is therefore not implemented.

All unacceptable classifications are either failed in check mode or rejected in design mode.

Hollow sections

The classification rules for SHS and RHS relate to *"hot-finished hollow sections"* only (*"cold-formed hollow sections"* are not included in this release).

Important Note

The classification used to determine M_b is based on the maximum axial compressive load in the relevant segment length. Furthermore, the Code clearly states that this classification should (only) be used to determine the moment capacity and lateral torsional buckling resistance to Clause 4.2 and 4.3 for use in the interaction equations. Thus, when carrying out the strength checks, the program determines the classification at the point at which strength is being checked.

Shear Capacity (Beams: BS 5950)

The shear check is performed according to BS 5950-1: 2000 Clause 4.2.3. for the absolute value of shear force normal to the x-x axis (F_{vx}) and normal to the y-y axis (F_{vy}), at the point under consideration.

Shear buckling

When the web slenderness exceeds 70ε shear buckling can occur in rolled sections. There are very few standard rolled sections that breach this limit. *Tekla Structural Designer* will warn you if this limit is exceeded, but will not carry out any shear buckling checks.

Moment Capacity (Beams: BS 5950)

The moment capacity check is performed according to BS 5950-1: 2000 Clause 4.2.5 for the moment about the x-x axis (M_x) and about the y-y axis (M_y), at the point under consideration. The moment capacity can be influenced by the magnitude of the shear force ("low shear" and "high shear" conditions). The maximum absolute shear to either side of a point load is examined to determine the correct condition for the moment capacity in that direction.

Note

Not all cases of high shear in two directions combined with moments in two directions along with axial load are considered thoroughly by BS 5950-1: 2000. The following approach is adopted by *Tekla Structural Designer*:

- 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. The message associated with this status is "High shear and axial load are present, additional hand calculations are required for cross-section capacity to Annex H.3". *Tekla Structural Designer* does not perform any calculations for this condition.

- 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 and the associated message is, "High shear present normal to the y-y axis, no calculations are performed for this condition."
- if high shear is present normal to the y-y axis and there is no axial load, the y-y moment check and the cross-section capacity check are each given Beyond Scope statuses. The message associated with this condition is, "High shear present normal to the y-y axis, no calculations are performed for this condition."

Axial Capacity (Beams: BS 5950)

The axial capacity check is performed according to BS 5950-1: 2000 Clause 4.6.1 using the gross area and irrespective of whether the axial force is tensile or compressive. This check is for axial compression capacity and axial tension capacity. Compression resistance is a buckling check and as such is considered under Compression Resistance.

Cross-section Capacity (Beams: BS 5950)

The cross-section capacity check covers the interaction of axial load and bending to Clause 4.8.2 and 4.8.3.2 appropriate to the type (for example – doubly symmetric) and classification of the section. Since the axial tension capacity is not adjusted for the area of the net section then the formulae in Clause 4.8.2.2 and 4.8.3.2 are the same and can be applied irrespective of whether the axial load is compressive or tensile.

The *Note* in also applies here.

Ultimate Limit State (Buckling) (Beams: BS 5950)

Lateral Torsional Buckling Resistance, Clause 4.3 (Beams: BS 5950)

For beams that are unrestrained over part or all of a span, a Lateral Torsional Buckling (LTB) check is required either:

- in its own right, Clause 4.3 check,
- as part of an Annex G check,
- as part of a combined buckling check to 4.8.3.3.1, 4.8.3.3.2 or 4.8.3.3.3, (see , , and , respectively)

This check is not carried out under the following circumstances:

- when bending exists about the minor axis only,
- when the section is a CHS or SHS,
- when the section is an RHS that satisfies the limits given in Table 15 of BS 5950-1: 2000.

For sections in which LTB cannot occur (the latter two cases above) the value of M_b for use in the combined buckling check is taken as the full moment capacity, M_{cx}, not reduced for high shear in accordance with Clause 4.8.3.3.3 (c), equation 2 (See).

Effective lengths

The value of effective length factor is entirely at your choice. The default value is 1.0 for "normal" loads and 1.2 for "destabilizing loads". Different values can apply in the major and minor axis.

Lateral Torsional Buckling Resistance, Annex G (Beams: BS 5950)

This check is applicable to I- and H-sections with equal or unequal[1] flanges.

The definition of this check is the out-of-plane buckling resistance of a member or segment that has a laterally unrestrained compression flange and the other flange has intermediate lateral restraints at intervals. It is used normally to check the members in portal frames in which only major axis moment and axial load exist. Although not stated explicitly in BS 5950-1: 2000, it is taken that the lateral torsional buckling moment of resistance, M_b, from the Annex G check can be used in the interaction equations of Clause 4.8.3.3 (combined buckling).

Since this is not explicit within BS 5950-1: 2000 a slight conservatism is introduced. In a straightforward Annex G check the axial load is combined with major axis moment. In this case both the slenderness for lateral torsional buckling and the slenderness for compression buckling are modified to allow for the improvement provided by the tension flange restraints (λ_{LT} replaced by λ_{TB} and λ replaced by λ_{TC}). When performing a combined buckling check in accordance with 4.8.3.3 the improvement is taken into account in determining the buckling resistance moment but not in determining the compression resistance. If the incoming members truly only restrain the tension flange, then you should switch off the minor axis strut restraint at these positions.

The original source research work for the codified approach in Annex G used test specimens in which the tension flange was continuously restrained. When a segment is not continuously restrained but is restrained at reasonably frequent intervals it can be clearly argued that the approach holds true. With only one or two restraints present then this is less clear.BS 5950-1: 2000 is clear that there should be "at least one intermediate lateral restraint" (See Annex G.1.1). *Nevertheless, you are ultimately responsible for accepting the adequacy of this approach.*

For this check *Tekla Structural Designer* sets m_t to 1.0 and calculates n_t. The calculated value of n_t is based on M_{max} being taken as the maximum of M_1 to M_5, and not the true maximum moment value where this occurs elsewhere in the length. The effect of this approach is likely to be small. If at any of points 1 - 5, R >1[2], then the status of the check is set to Beyond Scope.

Reference restraint axis distance, a

The ***reference restraint axis distance*** is measured between some reference axis on the restrained member - usually the centroid - to the axis of restraint - usually the centroid of the restraining member. The measurement is shown diagramatically in Figure G.1 of BS 5950-1: 2000.

Tekla Structural Designer does not attempt to determine this value automatically. Instead, by default, it uses half the depth of the restrained section, and you can specify a value to be added to, or subtracted from, this at each restraint point. **You are responsible for specifying the appropriate values for each restraint position. The default value of 0mm may be neither correct nor safe.**

1. *Unequal flanged sections are not currently included.*
2. *Which could happen since R is based on Z and not S.*

Compression Resistance (Beams: BS 5950)

For most structures, all the members resisting axial compression need checking to ensure adequate resistance to buckling about both the major- and minor-axis. Since the axial force can vary throughout the member and the buckling lengths in the two planes do not necessarily coincide, both are checked. Because of the general nature of a beam, it may not always be safe to assume that the combined buckling check will always govern. Hence the compression resistance check is performed independently from the other strength and buckling checks.

Effective lengths

The value of effective length factor is entirely at your choice. The default value is 1.0 for “normal” loads and 1.2 for “destabilizing loads”. Different values can apply in the major and minor axis.

Beams are less affected by sway than columns but the effectiveness of the incoming members to restrain the beam in both position and direction is generally less than for columns. Hence, it is less likely that effective length factors greater than 1.0 will be required but equally factors less than 1.0 may not easily be justified. Nevertheless, it is your responsibility to adjust the value from 1.0 and to justify such a change.

Please note that the requirements for slenderness limits in (for example $l/r \leq 180$) are no longer included in BS 5950-1: 2000. Consequently *Tekla Structural Designer* does not carry out such checks. Accordingly, for lightly loaded members you should ensure that the slenderness ratio is within reasonable bounds to permit handling and erection and to provide a reasonable level of robustness.

Member Buckling Resistance, Clause 4.8.3.3.1 (Beams: BS 5950)

This check is used for channel sections. Such sections can be Class 1, 2 or 3 Plastic, Compact or Semi-compact (Class 4 Slender sections and Effective Class 2 sections are not allowed in this release).

Note that, whilst this check could be used for any section type dealt with in the subsequent sections, the results can never be any better than the alternatives but can be worse.

Two formulae are provided in Clause 4.8.3.3.1, both are checked; the second is calculated twice – once for the top flange and once for the bottom flange.

See also the *Important Note* at the end of .

Only one value of F is used, the worst anywhere in the length being checked. If the axial load is tensile, then F is taken as zero.

If this check is invoked as part of an Annex G check, and thus M_b is governed by Annex G, then m_{LT} is taken as 1.0.

Member Buckling Resistance, Clause 4.8.3.3.2 (Beams: BS 5950)

This check is used for Class 1, 2 and 3 Plastic, Compact and Semi-compact rolled I- and H-sections with equal flanges (Class 4 Slender sections and Effective Class 2 sections are not included in this release).

Three formulae are provided in Clause 4.8.3.3.2 (c) to cover the combined effects of major and minor axis moment and axial force.These are used irrespective of whether all three forces / moments exist. Clause 4.9 deals with biaxial moment in the absence of axial force, Clause 4.8.3.3.2 (c) can also be used in such cases by setting the axial force to zero.

All three formulae in Clause 4.8.3.3.2 (c) are checked; the second is calculated twice – once for each flange.

Only one value of F is used, the worst anywhere in the length being checked. If the axial load is tensile, then F is taken as zero.

Important Note

Clause 4.8.3.3.4 defines the various equivalent uniform moment factors. The last three paragraphs deal with modifications to these depending upon the method used to allow for the effects of sway. This requires that for sway sensitive frames the uniform moment factors, m_x, m_y and m_{xy}, should be applied to the non-sway moments only. In this release there is no mechanism to separate the sway and non-sway moments, *Tekla Structural Designer* adopts a conservative approach and sets these 'm' factors equal to 1.0 if the frame is sway sensitive (in either direction). This is doubly conservative for sway-sensitive unbraced frames since it is likely that all the loads in a design combination and not just the lateral loads will be amplified. In such a case, both the sway and non-sway moments are increased by k_{amp} and neither are reduced by the above “m” factors. The calculation of m_{LT} is unaffected by this approach, and thus if the second equation of Clause 4.8.3.3.2 (c) governs, then the results are not affected.

Member Buckling Resistance, Clause 4.8.3.3.3 (Beams: BS 5950)

This check is used for Class 1, 2 and 3 Plastic, Compact and Semi-compact hollow sections (Class 4 Slender sections and Effective Class 2 sections are not included in this release).

Four formulae are provided in Clause 4.8.3.3.3 (c) to cover the combined effects of major and minor axis moment and axial force. These are used irrespective of whether all three forces / moments exist. Clause 4.9 deals with biaxial moment in the absence of axial force, Clause 4.8.3.3.3 (c) can also be used in such cases by setting the axial force to zero.

The second and third formulae are mutually exclusive – that is the second is used for CHS, SHS and for RHS when the limits contained in Table 15 are **not** exceeded. On the other hand the third formula is used for those RHS that exceed the limits given in Table 15. Thus only three formulae are checked; the first, second and fourth or the first, third and fourth. Either the second or third (as appropriate) is calculated twice – once for each *"flange"*.

Only one value of F is used, the worst anywhere in the length being checked. If the axial load is tensile, then F is taken as zero.

See also the *Important Note* at the end of .

Web Openings (Beams: BS 5950)

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. 10) and Appendix B of the joint CIRIA/SCI Publication P068(Ref. 5). 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 BS 5950

Design Method (Composite Beams: BS 5950)

Unless explicitly stated all calculations are in accordance with the relevant sections of BS 5950-3.1:1990+A1:2010(Ref. 1). You may find the handbook and commentary to the Code of Practice published by the Steel Construction Institute (Ref. 3 and 4) useful.

Construction stage design checks

When you use *Tekla Structural Designer* to 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 BS 5950-1:2000:

- section classification (Clause 3.5.2),
- shear capacity (Clause 4.2.3),
- moment capacity:
  - Clause 4.2.5.2 for the low shear condition,
- Clause 4.2.5.3 for the high shear condition,
- lateral torsional buckling resistance (Clause 4.3.6),

This condition is only checked in those cases where the profile decking or precast concrete slab (at your request) does not provide adequate restraint to the beam.

- web openings,
- Westok checks,
  - Shear horizontal,
- Web post buckling,
- Vierendeel bending,
- construction stage total load deflection check.

Composite stage design checks

When you use *Tekla Structural Designer* to 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 BS 5950 : Part 3 : Section 3.1 : 1990 (unless specifically noted otherwise).

Ultimate Limit State Checks

- section classification (Clause 4.5.2), depending on whether adequate connection is achieved between the compression flange and the slab. The section classification allows for the improvement of the classification of the section if the appropriate conditions are met,
- vertical shear capacity (BS 5950-1:2000 - Clause 4.2.3),

- longitudinal shear capacity (Clause 5.6) allowing for the profiled metal decking, transverse reinforcement and other reinforcement which has been defined,
- number of shear connectors required (Clause 5.4.7) between the point of maximum moment and the end of the beam, or from and between the positions of significant point loads,
- moment capacity:
  - Clause 4.4.2 for the low shear condition,
- Clause 5.3.4 for the high shear condition,
- web openings.

Serviceability Limit State Checks

- service stresses (Clause 6.2),
  - concrete
- steel top flange and bottom flange
- deflections (Clause 6.1.2)
  - self-weight
- SLAB loadcase,
- dead load,
- imposed load[1],
- total deflections,
- natural frequency check (Clause 6.4).

1. This is the only limit given in BS 5950 : Part 3 : Section 3.1 : 1990.

Construction stage design (Composite Beams: BS 5950)

Tekla Structural Designer performs all checks for this condition in accordance with BS 5950-1:2000(Ref. 2)

Section classification (Composite Beams: BS 5950)

Cross-section classification is determined using Table 11 and Clause 3.5.

The classification of the section must be Plastic (Class 1), Compact (Class 2) or Semi-compact (Class 3).

Sections which are classified as Slender (Class 4) are beyond the scope of *Tekla Structural Designer*.

Member strength checks (Composite Beams: BS 5950)

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 each side of a web opening as well as all other points of interest along the beam.

Shear capacity

Shear capacity is determined in accordance with Clause 4.2.3. Where the applied shear force exceeds 60% of the capacity of the section, the high shear condition applies to the bending moment capacity checks (see below).

Bending moment capacity

This is calculated to Clause 4.2.5.2 (low shear at point) or Clause 4.2.5.3 (high shear at point) for plastic, compact and semi-compact sections.

Lateral torsional buckling checks (Composite Beams: BS 5950)

BS 5950 : Part 3 : Section 3.1 : 1990 states that lateral torsional buckling checks are not required when the angle between the direction of span of the beam and that of the profile decking is greater than or equal to 45°.

When the angle is less than this, then lateral torsional buckling checks will normally be required. *Tekla Structural Designer* allows you to switch off these checks by specifying that the entire length between the supports 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 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 clause 4.3.8 and Annex B of BS 5950-1:2000.

Deflection checks (Composite Beams: BS 5950)

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: BS 5950)

Tekla Structural Designer performs all checks for the composite stage condition in accordance with BS 5950-3.1:1990+A1:2010 unless specifically noted otherwise.

Equivalent steel section - Ultimate limit state (ULS) (Composite Beams: BS 5950)

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: BS 5950)

For section classification purposes the true section is used. Tekla Structural Designer classifies the section in accordance with the requirements of BS 5950-1:2000 except where specifically modified by those of BS 5950-3.1:1990+A1:2010.

There are a small number of sections which fail to meet a classification of compact at the composite stage. Although BS 5950-3.1:1990+A1:2010 covers the design of such members they are not allowed in this release of *Tekla Structural Designer*.

Member strength checks (ULS) (Composite Beams: BS 5950)

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 each side of a web opening as well as all other points of interest along the beam.

Shear Capacity (Vertical)

is determined in accordance with Clause 4.2.3 of BS 5950-1:2000. 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).

Shear Capacity (Longitudinal)

the longitudinal shear resistance of a unit length of the beam is calculated in accordance with Clause 5.6. You can set the position and attachment of the decking and details of the reinforcement that you want to provide. *Tekla Structural Designer* takes these into account during the calculations. The following assumptions are made:

- the applied longitudinal shear force is calculated at the centre-line of the beam, and at the position of the lap (if known). If the position of the lap is not known, then the default value of 0mm should be used (that is the lap is at the centre-line of the beam) as this is the worst case scenario.
- the minimum concrete depth is assumed for calculating the area of concrete when the profile decking and beam spans are parallel,
- the total concrete area is used when the profile decking and beam spans are perpendicular,
- the overall depth of the slab is used for precast concrete slabs. that is the topping is assumed to be structural and any voids or cores are ignored.

In the calculations of the longitudinal shear resistance on the beam centre-line and at the lap, the areas used for the reinforcement are 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 (α) between these two extremes then the program calculates:

- the longitudinal shear resistance as if the sheeting were perpendicular, v_1,
- the longitudinal shear resistance as if the sheeting were parallel, v_2,
- then the modified longitudinal shear resistance is calculated from these using the relationship, $v_1\sin^2(\alpha) + v_2\cos^2(\alpha)$.

Moment Capacity

for the low shear condition the plastic moment capacity is determined in accordance with Clause 4.4.2. For the high shear condition the approach given in Clause 5.3.4 is adopted.

The overall depth of the slab is used for precast concrete slabs. that is the topping is assumed to be structural and any voids / cores are ignored.

In this calculation 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.

Shear connectors (ULS) (Composite Beams: BS 5950)

Tekla Structural Designer checks shear connectors to Clause 5.4.7. It calculates the stud reduction factor based on the number of studs in a group.

Tekla Structural Designer always uses 2*e (and not br) in the calculation of k for perpendicular profiles, and always uses br for parallel cases.

For angled cases two values of k are calculated and summed in accordance with Clause 5.4.7.4. In this instance *Tekla Structural Designer* uses 2*e for the calculation of k1 and br for the calculation of k2.

> *Caution:*
> *The value of e (when used) can have a very significant effect on the value of k. This can have a dramatic effect on the number of studs required for a given beam size. Alternatively for a fixed layout of studs this can have a significant effect on the required beam size.*

Optimise Shear Connection

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 (for example

40% for beams with a span less than 10 m) is reached or until the minimum spacing requirements are reached. This results in partial shear connection.

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.

The percentage degree of shear connection is always calculated by the program as a proportion of the maximum concrete force and not simply Na/Np as in the code.

Section properties - serviceability limit state (SLS) (Composite Beams: BS 5950)

BS 5950-3.1:1990+A1:2010 indicates that the Serviceability Limit State modular ratio for all SLS calculations should be based upon an effective modular ratio derived from the proportions of long term loading in the design combination being considered.

Tekla Structural Designer therefore calculates the deflection for the beam based on the properties as tabulated below.

Loadcase Type	Properties used
self-weight	bare beam
Slab	bare beam
Dead	composite properties calculated using the modular ratio for long term loads

Live	composite properties calculated using the effective modular ratio 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 modular ratio for short term loads
Total loads	these are calculated from the individual loadcase loads as detailed above again using the principle of superposition

Stress checks (SLS) (Composite Beams: BS 5950)

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.

Deflection checks (SLS) (Composite Beams: BS 5950)

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 will not be 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. See:

- 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 in accordance with Clause 6.1.4.

It is the deflection due to imposed loads alone (allowing for long and short term effects) which is limited within the code. Tekla Structural Designer also gives you the deflection for the Slab loadcase which 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.

Natural frequency checks (SLS) (Composite Beams: BS 5950)

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. 6) 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 modular ratio) but not modified for the effects of partial interaction.

Web Openings (Composite Beams: BS 5950)

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. 10) and Appendix B of the joint CIRIA/SCI Publication P068(Ref. 5). 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 resuistance
- 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' - $M/I = E/R = \sigma/y$. 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.

Steel Column Design to BS 5950

Design method (Columns: BS 5950)

Unless explicitly stated all calculations are in accordance with the relevant sections of BS 5950-1: 2000. You may find the handbook and commentary to the Code of Practice published by the Steel Construction Institute useful.

Ultimate Limit State (Strength) (Columns: BS 5950)

The checks relate to doubly symmetric prismatic sections i.e. rolled I- and H-sections and to doubly symmetric hot-finished hollow sections i.e. SHS, RHS and CHS. Other section types are not currently covered.

The 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: BS 5950)

General

The classification of the cross section is in accordance with BS 5950-1: 2000.

Steel columns can be classified as:

- Plastic Class = 1
- Compact Class = 2
- Semi-compact Class = 3
- Slender Class = 4

Class 4 sections are not allowed.

Sections with a Class 3 web can be taken as Class 2 sections (Effective Class 2) providing the cross section is equilibrated to that described in Clause 3.5.6 where the section is given an "effective" plastic section modulus, S_{eff}. This approach is not adopted in the current version of *Tekla Structural Designer*.

All unacceptable classifications are either failed in check mode or rejected in design mode.

Hollow sections

The classification rules for SHS and RHS relate to "hot-finished hollow sections" only ("cold-formed hollow sections" are not included in this release).

Important Note

The classification used to determine M_b is based on the maximum axial compressive load in the relevant segment length. Furthermore, the Code clearly states that this classification should (only) be used to determine the moment capacity and lateral torsional buckling resistance to Clause 4.2 and 4.3 for use in the interaction equations. Thus, when carrying out the strength checks, the program determines the classification at the point at which strength is being checked.

Shear Capacity (Columns: BS 5950)

The shear check is performed according to BS 5950-1: 2000 Clause 4.2.3. for the absolute value of shear force normal to the x-x axis and normal to the y-y axis, F_{vx} and F_{vy}, at the point under consideration.

Shear buckling

When the web slenderness exceeds 70ε shear buckling can occur in rolled sections. There are very few standard rolled sections that breach this limit. *Tekla Structural Designer* will warn you if this limit is exceeded, but will not carry out any shear buckling checks.

Moment Capacity (Columns: BS 5950)

The moment capacity check is performed according to BS 5950-1: 2000 Clause 4.2.5 for the moment about the x-x axis and about the y-y axis, M_x and M_y, at the point under consideration. The moment capacity can be influenced by the magnitude of the shear force ("low shear" and "high shear" conditions). 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 x-x axis

The treatment of high shear is axis dependent. In this release for CHS, if high shear is present, the moment capacity about the x-x axis is not calculated, the check is given a Beyond Scope status and an associated explanatory message.

High shear condition about y-y axis

For rolled sections in the current release, if high shear is present normal to the y-y axis then the moment capacity about the y-y axis is not calculated, the check is given a Beyond Scope status and an associated explanatory message.

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 y-y axis then in this release the moment capacity about the y-y axis is not calculated, the check is given a Beyond Scope status and an associated explanatory message.

Note

Not all cases of high shear in two directions combined with moments in two directions along with axial load are considered thoroughly by BS 5950-1: 2000. The following approach is adopted by *Tekla Structural Designer*:

- 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. The message associated with this status is "High shear and axial load are present, additional hand calculations are required for cross-section capacity to Annex H.3". *Tekla Structural Designer* does not perform any calculations for this condition.
- 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 and the associated message is, "High shear present normal to the y-y axis, no calculations are performed for this condition."
- if high shear is present normal to the y-y axis and there is no axial load, the y-y moment check and the cross-section capacity check are each given Beyond Scope statuses. The message associated with this condition is, "High shear present normal to the y-y axis, no calculations are performed for this condition."

Axial Capacity (Columns: BS 5950)

The axial capacity check is performed according to BS 5950-1: 2000 Clause 4.6.1 using the gross area and irrespective of whether the axial force is tensile or compressive. This check is for axial compression capacity and axial tension capacity. Compression resistance is a buckling check and as such is considered under .

Cross-section Capacity (Columns: BS 5950)

The cross-section capacity check covers the interaction of axial load and bending to Clause 4.8.2 and 4.8.3.2 appropriate to the type (for example – doubly symmetric) and classification of the section. Since the axial tension capacity is not adjusted for the area of the net section then the formulae in Clause 4.8.2.2 and 4.8.3.2 are the same and can be applied irrespective of whether the axial load is compressive or tensile.

The *Note* in also applies here.

Ultimate Limit State (Buckling) (Columns: BS 5950)

Lateral Torsional Buckling Resistance, Clause 4.3 (Columns: BS 5950)

For columns that are unrestrained over part or all of a span, a Lateral Torsional Buckling (LTB) check is required either:

- in its own right, Clause 4.3 check,
- as part of an Annex G check,
- as part of a combined buckling check to 4.8.3.3.2 or 4.8.3.3.3, (see , and).

This check is not carried out under the following circumstances:

- when bending exists about the minor axis only,
- when the section is a CHS or SHS,
- when the section is an RHS that satisfies the limits given in Table 15 of BS 5950-1: 2000.

For sections in which LTB cannot occur (the latter two cases above) the value of M_b for use in the combined buckling check is taken as the full moment capacity, M_{cx}, not reduced for high shear in accordance with Clause 4.8.3.3.3 (c), equation 2 (see).

Destabilising loads are excluded from *Tekla Structural Designer*, this is justified by the rarity of the necessity to apply such loads to a column. If such loads do occur, then you can adjust the “normal” effective length to take this into account although you can not achieve the code requirement to set m_{LT} to 1.0.

Effective lengths

The value of effective length factor is entirely at your choice. The default value is 1.0.

Lateral Torsional Buckling Resistance, Annex G (Columns: BS 5950)

This check is applicable to I- and H-sections with equal or unequal[1] flanges.

The definition of this check is the out-of-plane buckling resistance of a member or segment that has a laterally unrestrained compression flange and the other flange has intermediate lateral restraints at intervals. It is used normally to check the members in portal frames in which only major axis moment and axial load exist. Although not stated explicitly in BS 5950-1: 2000, it is taken that the lateral torsional buckling moment of resistance, M_b, from the Annex G check can be used in the interaction equations of Clause 4.8.3.3 (combined buckling).

Since this is not explicit within BS 5950-1: 2000 a slight conservatism is introduced. In a straightforward Annex G check the axial load is combined with major axis moment. In this case both the slenderness for lateral torsional buckling and the slenderness for compression buckling are modified to allow for the improvement provided by the tension flange restraints (λ_{LT} replaced by λ_{TB} and λ replaced by λ_{TC}). When performing a combined buckling check in accordance with 4.8.3.3 the improvement is taken into account in determining the buckling resistance moment but not in determining the compression resistance. If the incoming members truly only restrain the tension flange, then you should switch off the minor axis strut restraint at these positions.

The original source research work for the codified approach in Annex G used test specimens in which the tension flange was continuously restrained. When a segment is not continuously restrained but is restrained at reasonably frequent intervals it can be clearly argued that the approach holds true. With only one or two restraints present then this is less clear.BS 5950-1: 2000 is clear that there should be “at least one intermediate lateral restraint” (See Annex G.1.1). **Nevertheless, you are ultimately responsible for accepting the adequacy of this approach.**

For this check *Tekla Structural Designer* sets m_t to 1.0 and calculates n_t. The calculated value of n_t is based on M_{max} being taken as the maximum of M_1 to M_5, and not the true maximum moment value where this occurs elsewhere in the length. The effect of this approach is likely to be small. If at any of points 1 - 5, R >1[2], then *Tekla Structural Designer* sets the status of the check to Beyond Scope.

Reference restraint axis distance, a

The ***reference restraint axis distance*** is measured between some reference axis on the restrained member - usually the centroid - to the axis of restraint - usually the centroid of the restraining member. The measurement is shown diagramatically in Figure G.1 of BS 5950-1: 2000.

Tekla Structural Designer does not attempt to determine this value automatically, since such an approach is fraught with difficulty and requires information from you which is only used for this check. Instead, by default, *Tekla Structural Designer* uses half the depth of the restrained section, and you can specify a value to be added to, or subtracted from, this at each restraint point. **You are responsible for specifying the appropriate values for each restraint position. The default value of 0mm may be neither correct nor safe**.

1. Unequal flanged sections are not currently included.
2. Which could happen since R is based on Z and not S.

Compression Resistance (Columns: BS 5950)

For most structures, all the members resisting axial compression need checking to ensure adequate resistance to buckling about both the major- and minor-axis. Since the axial force can vary throughout the member and the buckling lengths in the two planes do not necessarily coincide, both are checked. Because of the general nature of a column, it may not always be safe to assume that the combined buckling check will always govern. Hence the compression resistance check is performed independently from all other strength and buckling checks.

Effective lengths

The value of effective length factor is entirely at your choice. The default value is 1.0. Different values can apply in the major and minor axis.

The minimum theoretical value is 0.5 and the maximum infinity for columns in rigid moment resisting (RMR) frames. Practical values for simple columns are in the range 0.7 to 2.0. 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 the amplified moments method. However, there is a caveat on the value of effective length factor even when allowance is made for sway.

In particular for RMR frames, 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 amplification of these moments has little effect. 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 as given in Table 22 of BS 5950-1: 2000.

Member Buckling Resistance, Clause 4.8.3.3.2 (Columns: BS 5950)

This check is used for Class 1, 2 and 3 Plastic, Compact and Semi-compact rolled I- and H-sections with equal flanges (Class 4 Slender sections and Effective Class 2 sections are not included in this release).

Three formulae are provided in Clause 4.8.3.3.2 (c) to cover the combined effects of major and minor axis moment and axial force.These are used irrespective of whether all three forces / moments exist. Clause 4.9 deals with biaxial moment in the absence of axial force, Clause 4.8.3.3.2 (c) can also be used in such cases by setting the axial force to zero.

All three formulae in Clause 4.8.3.3.2 (c) are checked; the second is calculated twice – once for Face A and once for Face C.

Only one value of F is used, the worst anywhere in the length being checked. If the axial load is tensile, then F is taken as zero.

Important Note

Clause 4.8.3.3.4 defines the various equivalent uniform moment factors. The last three paragraphs deal with modifications to these depending upon the method used to allow for the effects of sway. This requires that for sway sensitive frames the uniform moment factors, m_x, m_y and m_{xy}, should be applied to the non-sway moments only. In this release there is no mechanism to separate the sway and non-sway moments, *Tekla Structural Designer* adopts the only conservative approach and sets these "m" factors equal to 1.0 if the frame is sway sensitive (in either direction). This is doubly conservative for sway-sensitive unbraced frames since it is likely that all the loads in a design combination and not just the lateral loads will be amplified. In such a case, both the sway and non-sway moments are increased by k_{amp} and neither are reduced by the above "m" factors. The calculation of m_{LT} is unaffected by this approach, and thus if the second equation of Clause 4.8.3.3.2 (c) governs, then the results are not affected.

Member Buckling Resistance, Clause 4.8.3.3.3 (Columns: BS 5950)

This check is used for Class 1, 2 and 3 Plastic, Compact and Semi-compact hollow sections (Class 4 Slender sections and Effective Class 2 sections are not included in this release).

Four formulae are provided in Clause 4.8.3.3.3 (c) to cover the combined effects of major and minor axis moment and axial force. These are used irrespective of whether all three forces / moments exist. Clause 4.9 deals with biaxial moment in the absence of axial force, Clause 4.8.3.3.3 (c) can also be used in such cases by setting the axial force to zero.

The second and third formulae are mutually exclusive – that is the second is used for CHS, SHS and for RHS when the limits contained in Table 15 are **not** exceeded. On the other hand the third formula is used for those RHS that exceed the limits given in Table 15. Thus only three formulae are checked; the first, second and fourth or the first, third and fourth. Either the second or third (as appropriate) is calculated twice – once for Face C and once for Face A.

Only one value of F is used, the worst anywhere in the length being checked. If the axial load is tensile, then F is taken as zero.

See also the *Important Note* at the end of .

Serviceability limit state (Columns: BS 5950)

The column is assessed for sway and the following values are reported for each stack:

- Sway X (mm) and λ_{critx}
- Sway Y (mm) and λ_{crity}
- Sway X-Y (mm)

Depending on the reported λ_{crit} the column is classified as Sway or Non sway accordingly.

> *A sway assessment is only performed for the column if the Lambda 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 Lambda Crit Check box for those columns for which a sway assessment would be inappropriate*

Steel Brace Design to BS 5950

Design Method (Braces: BS 5950)

Unless explicitly stated all brace calculations are in accordance with the relevant sections of BS 5950-1:2000 (Ref. 2).

A basic knowledge of the design methods for braces in accordance with the design code is assumed.

Classification (Braces: BS 5950)

No classification is required for braces in tension.

Braces in compression are classified according to Clause 3.5 as either: Class 1, Class 2, Class 3 or Class 4.

Class 4 sections are not allowed.

Hollow sections

The classification rules for SHS and RHS relate to *"hot-finished hollow sections"* only (*"cold-formed hollow sections"* are not included in this release).

Axial Tension (Braces: BS 5950)

An axial tension capacity check is performed according to Clause 4.6.

Axial Compression (Braces: BS 5950)

An axial compression capacity check is performed according Clause 4.7.

Compression Buckling (Braces: BS 5950)

If axial compression exists, the member is also assessed according to Clause 4.7 with all relevant sub-clauses.

The default effective length in each axis is 1.0L.

Steel Single, Double Angle and Tee Section Design to BS 5950

Design Method (Angles and Tees: BS 5950)

The 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 BS 5950

"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 BS 5950

Additional Angle and Tee Limitations have to be considered when designing these sections to the above design methods.

Angle and Tee Limitations (BS 5950)

In the current version when designing tees, single, and double angles to BS 5950, 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
Bending	ok	ok	ok
Combined	ok	ok	ok

strength			
LTB	ok	ok	Beyond scope
Combined buckling	ok	ok	Beyond scope
Deflection	ok	ok	ok

In addition, the following limitations apply:

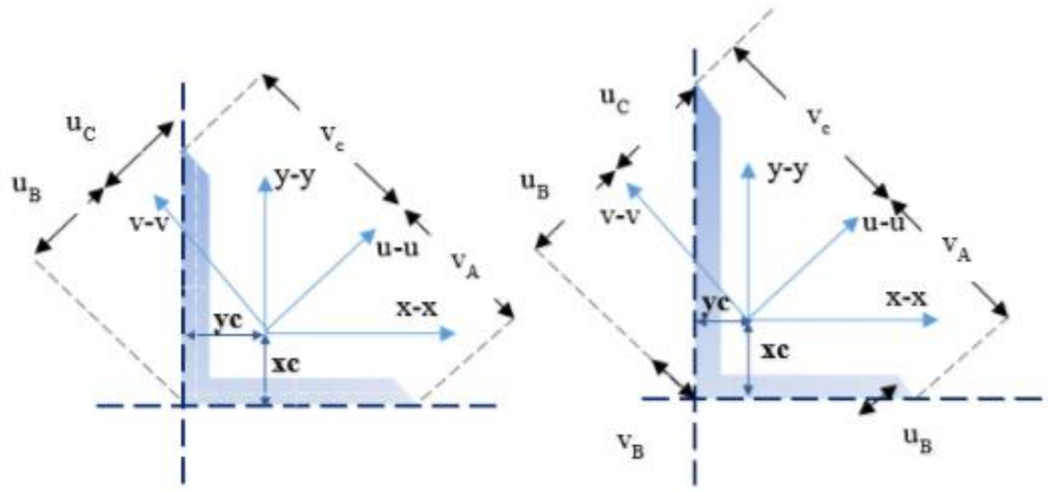
- All sections are assumed to be effectively loaded through the shear centre such that no additional torsion moments are developed. In addition no direct allowance is made for 'destabilizing loads'.
- Design excludes bending of the outstand leg of double angles loaded eccentrically e.g. supporting masonry.
- Conditions of restraint can be defined as top and bottom flange for lateral torsional buckling. It is upon these that the buckling checks are based. For the current release intermediate LTB restraints are omitted (i.e. only fully restrained for LTB, or unrestrained).
- Double angles and tee sections subject to moment with high shear are beyond scope.

Section Axes (Angles and Tees: BS 5950)

For all sections -

- x-x is the axis parallel to the flanges (major axis)
- y-y is the axis perpendicular to the flanges (minor axis)

- for Single and Double Angles
    - y-y parallel to long side (leg) - single angles
    - y-y parallel to long side (leg) - double angles with long leg back to back
    - y-y 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: BS 5950)

This section includes notes and assumptions made specifically for the design of tee, single and double 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 11 and Table 12 of BS 5950-1:2000 apply for the classification of these sections.

Class 4 section classification is only allowed for single angle, double angle and tee sections.

Axial Tension check

Section 4.6 of BS 5950 is used for this design check.

Axial Compression check

Section 4.6 of BS 5950 is used for this design check.

Shear check

Section 4.2.3 of BS 5950 is used for this design check.

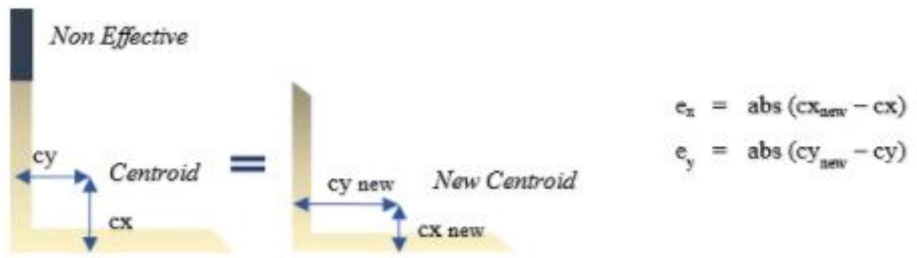
Moment check

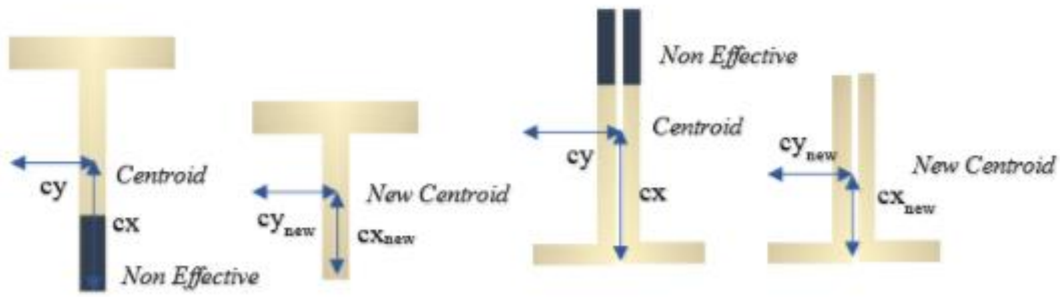
Section 4.2.5 of BS 5950 is used for this design check.

Tees, single angles and double angles are designed as Class 4.

Moment capacity for Class4 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 subject to axial compression only.

Thus:

$\Delta M_{Ed,x}$ = $e_x \times F_c$

$\Delta M_{Ed,y}$ = $e_y \times F_c$

Where:

F_c is the max compressive force in the span.

For tees and double angles e_x = 0. Hence, total minor design moment = minor design moment.

Where,

e_x and e_y = the shift of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section

e_x = $abs(cx_{new} – cx)$

e_y = $abs(cy_{new} – cy)$

So finally, a total moment is obtained for which the moment design check is performed:

$M_{total\ x}$ = $Abs(M_{Ed,x}) + Abs(\Delta M_{Ed,x})$

$M_{total\ y}$ = $Abs(M_{Ed,y}) + Abs(\Delta M_{Ed,y})$

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 (x, y) 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.

$\Delta M_u = \Delta M_x \times \cos\vartheta + \Delta M_y \times \sin\vartheta$

$\Delta M_v = -\Delta M_x \times \sin\vartheta + \Delta M_y \times \cos\vartheta$

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).

Tees, single angles and double angles subject to moment with high shear are beyond scope.

Combined bending and axial check

Section 4.8.3 of BS 5950 is used for this design check.

For Class 3:

$Abs\ (F_c / A_g\ p_y) + abs\ (M_{x,Ed} / M_{cx}) + abs\ (M_{y,Ed}/W_{el,min,y}) \leq 1.0$

For Class 4:

$Abs\ (F_c / A_{eff}\ p_y) + (abs\ (M_{x,Ed}) + abs\ (\Delta M_{x,Ed})) / M_{cx} + (abs\ (M_{y,Ed}) + abs\ (\Delta_{My,Ed})) / M_{cy} \leq 1.0$

Note that total moments are used when the section classification is Class 4.

Lateral torsional buckling check

Section 4.3 of BS 5950 is used for this design check.

This check is beyond scope for double angles.

In the case of a beam with continuous lateral torsional restraint along its length this check is not performed. The lateral torsional resistance is considered adequate.

For beams that are unrestrained, a Lateral Torsional Buckling (LTB) check is required, either:

- In its own right check for LTB, Clause 4.3, and B2.8 for tee sections and B.2.9 for angle sections in BS 5050-1: 2000.

- As part of combined buckling, Clause 4.8 “Members with combined moment and axial force”, 4.8.3.3, for single Angles I3 and I4 sections

This check is not performed when bending exists about the minor axis only

Conditions of restraint can be defined as top and bottom flange for lateral torsional buckling. It is upon these that the buckling checks are based. All intermediate LTB restraints for tees and single angles are ignored.

Combined buckling check

This check is beyond scope for double angles.

Single angles:

Clause I.4 - For beam with continuous lateral torsional restraint or for equal single angle sections with b/t ≤ 15ε a combined buckling check is performed according to Clause I.4.3 - the simplified method.

For any other case Clause 4.8.3.3.1 is used with the moments being resolved into the principal axes u-u and v-v. Two formulae are provided in Clause 4.8.3.3.1, both are checked

Tees:

Clause 4.8.3.3.1 is used. Two formulae are provided in Clause 4.8.3.3.1, both are checked.

- If the axial load is tensile, then F is taken as zero
- Only one value of F is used, the worst anywhere in the length being checked
- Class 4 slender sections are allowed

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 (BS 5950)

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