

Tekla Structural Designer 2016

Reference Guides (ACI/AISC)

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Analysis Verification Examples

Analysis Limitations and Assumptions

Linear analysis of structures containing material nonlinearity

If a structure containing nonlinear springs/elements is subjected to a linear (i.e. 1st or 2nd order linear, 1st order vibration, or 2nd order buckling) analysis, then the nonlinear springs/elements are constrained to act linearly as described below:

Nonlinear spring supports

In each direction in which a nonlinear spring has been specified, a single value of stiffness is applied which is taken as the greater of the specified -ve or +ve stiffness.

Any specified maximum capacities of the spring are ignored.

Tension only, or compression only elements

If either tension only or compression only element types have been specified, they are constrained to act as truss element types instead.

Nonlinear axial spring, or nonlinear torsional spring elements

If either of these element types have been specified, they are constrained to act as linear axial spring, or linear torsional spring element types instead.

A single value of stiffness is applied which is taken as the greater of the specified -ve or +ve stiffness.

Any specified maximum capacities of these spring elements are ignored.

Tension only cross braces

If tension only cross braces have been specified, the program determines which brace in each pair to put into tension by pushing the structure simultaneously in the positive direction 1 and positive direction 2.

The brace that goes into tension retains its full stiffness, while the compression brace becomes inactive.

If the above process fails to determine which of the pair goes into tension then a shear is applied to the structure and the braces are re-assessed.

Analysis of structures containing geometric nonlinearity

It is assumed that where secondary effects are significant (for example the structure is close to buckling), the engineer will elect to undertake a 2nd order analysis. If a 1st order analysis is performed any secondary effects will be ignored.

Analysis of structures containing curved beams

The member analysis for curved members in the plane of the curve is approximated by joining the values at the nodes, which are correct. For detailed analysis of curved members it is your responsibility to ensure sufficient discretization. More refined models can be achieved, if required, by decreasing the maximum facet error.

Story Shears

The story shears that are output are obtained by resolving the loads at column nodes horizontally into Direction 1 and Direction 2. Any loads associated with V & A braces are not included because these occur at mid-beam position and not at column nodes.

Member Deflections

There is a known issue when calculating member deflection profiles in combinations which can affect the following analysis types:

- 2nd Order Linear
- 1st Order Nonlinear
- 2nd Order Nonlinear

This occurs when the structures behaviour is significantly nonlinear because the deflection profile is currently based on linear superposition of the load cases within it. Clearly as structural response becomes more nonlinear the assumption that deflections can be superposed becomes less valid. This can cause a deflected profile to be calculated which deviates from the correct profile. The deviation can become significant if load cases fail to solve, but the combination succeeds in solving, as components of the deflected shape are missing entirely. It is suggested that for the three analysis types listed member deflections in combinations be used with caution and engineering judgment.

It should be noted that this limitation only affects member deflection profiles between solver nodes. All other results, including member force profiles and deflection at the solver nodes are correct

Unstable Structures

Flat Slab Structures

If a concrete structure exists with only flat slabs and columns (i.e. no beams and no shear walls), and the slab is modelled with a diaphragm this is an unstable structure,

assuming that the concrete columns are pinned at the foundation level (current default).

To prevent the instability you should mesh the slabs, as the resulting model does then consider the framing action that results from the interaction of the slabs and columns.

Analysis Verification Examples

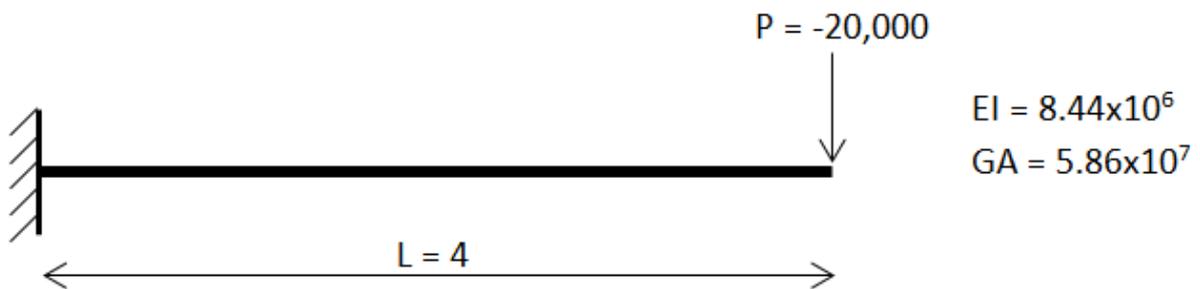
A small number of verification examples are included in this section. Our full automatic test suite for the Solver contains many hundreds of examples which are run and verified every time the Solver is enhanced.

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%

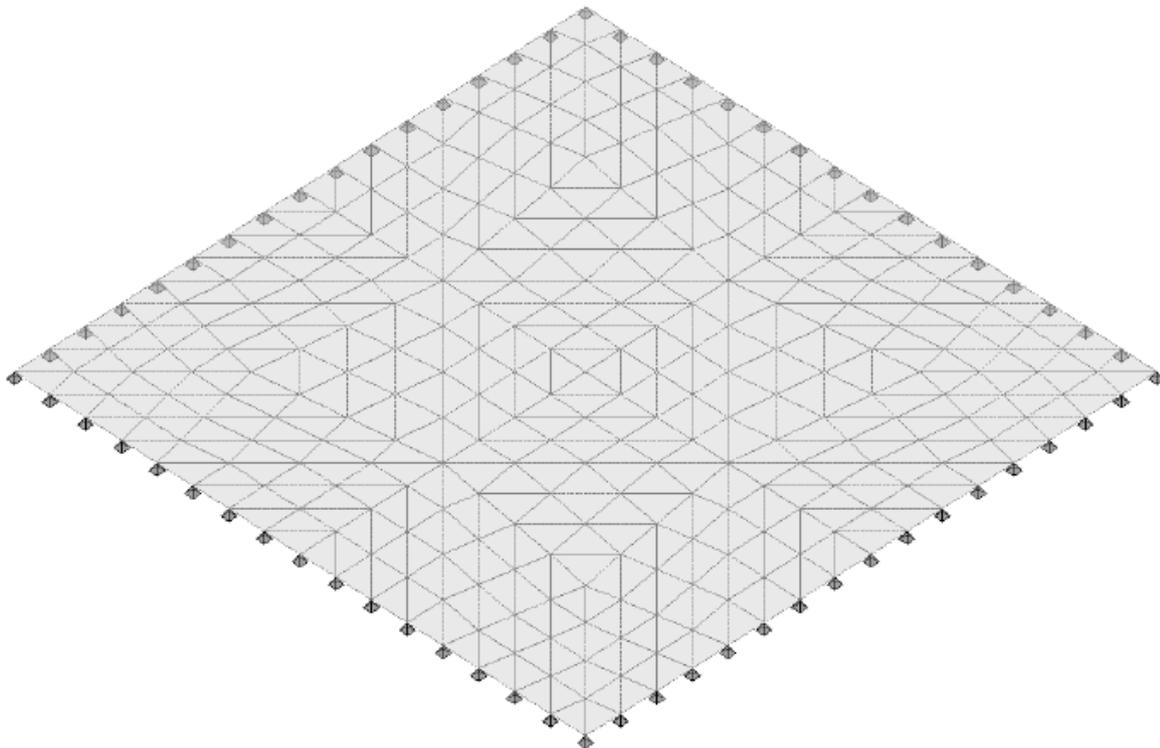
Conclusion

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 $E=2 \times 10^{11}$, $G=7.7 \times 10^{10}$ 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%
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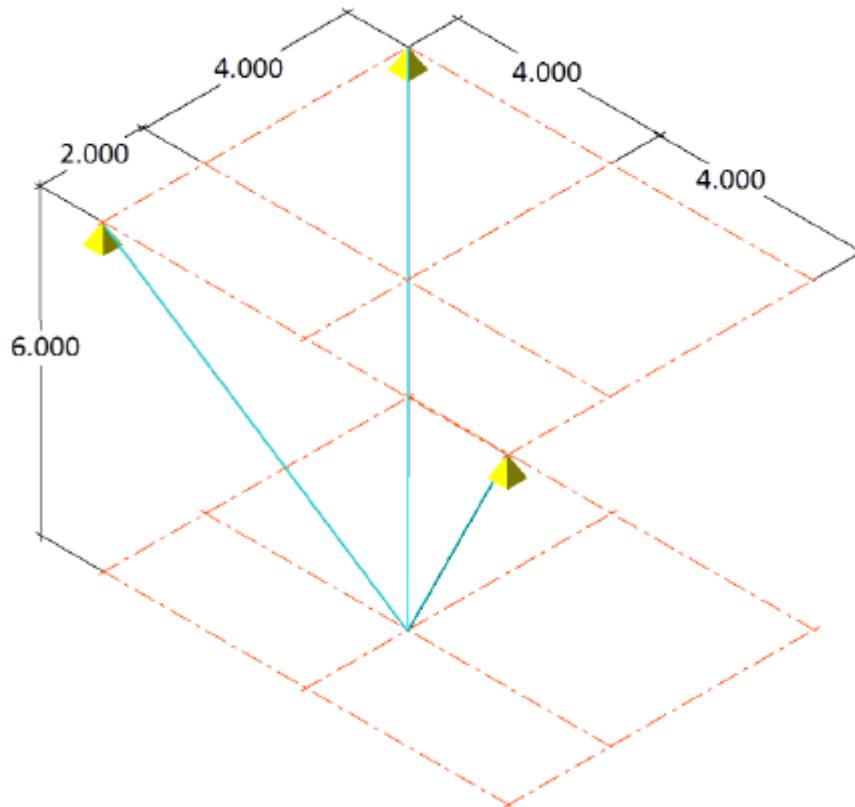
Conclusion

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

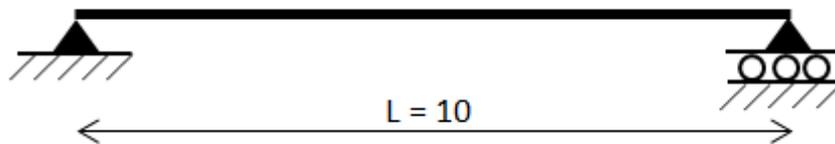
An exact match is observed between the values reported by the solver those reported by Beer and Johnston.

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 1.0×10^{-5} .

$$\alpha = 1.0 \times 10^{-5}$$



Assumptions

The roller pin is assumed to be frictionless.

Key Results

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

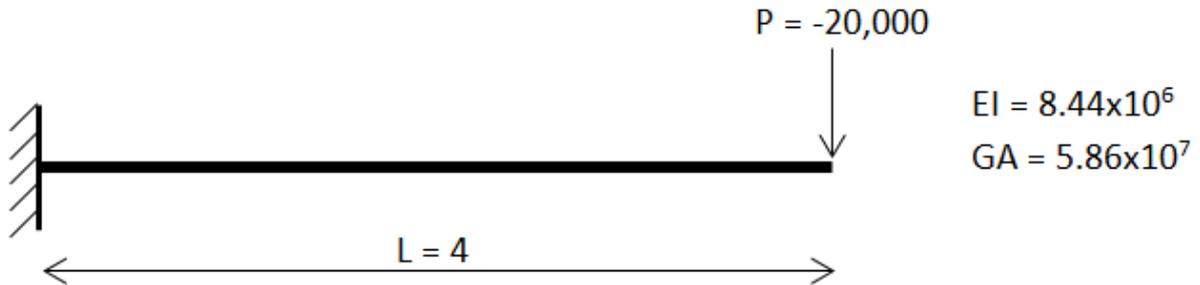
Conclusion

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%

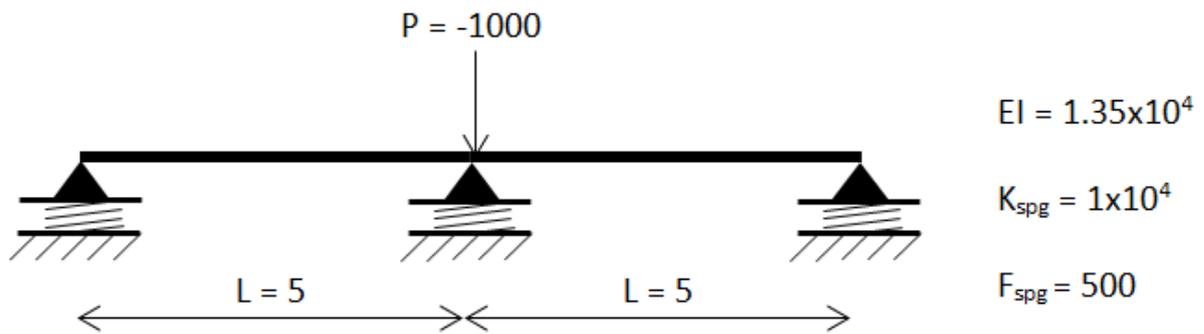
Conclusion

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

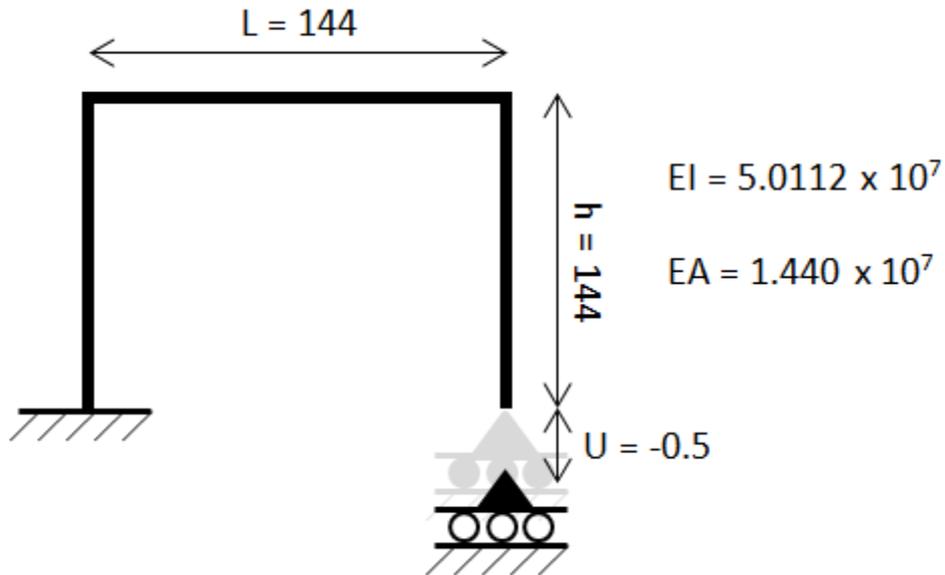
Conclusion

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

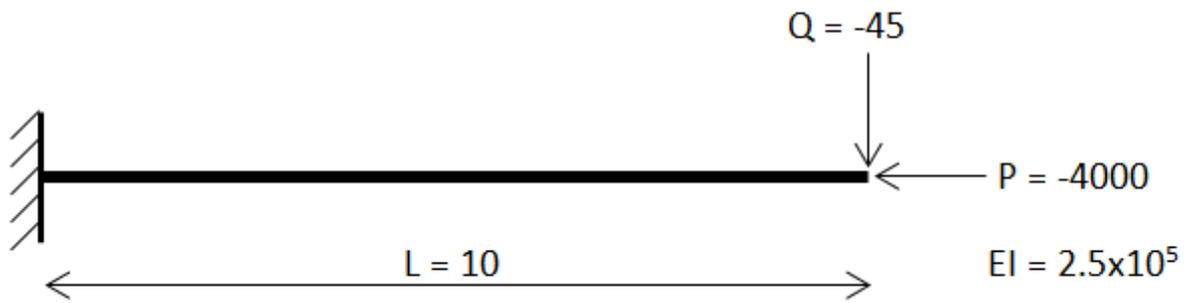
Conclusion

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

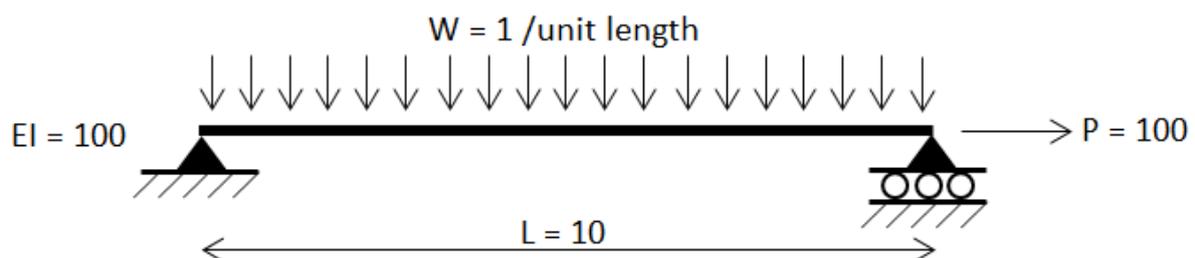
Conclusion

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:

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

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

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.

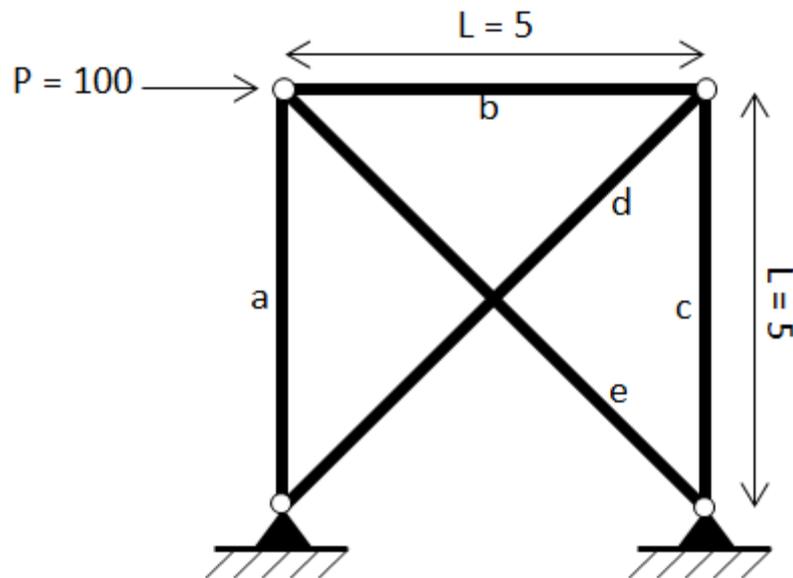
Reference

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
b		-P	-100	0
c		-P	-100	0
d	$\frac{PL^3}{3EI} + \frac{PL}{GA}$	141.42	141.42	0
e		0	0	0

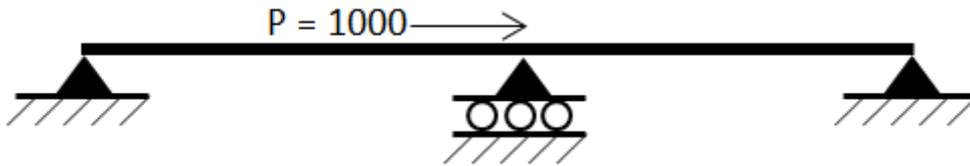
Conclusion

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

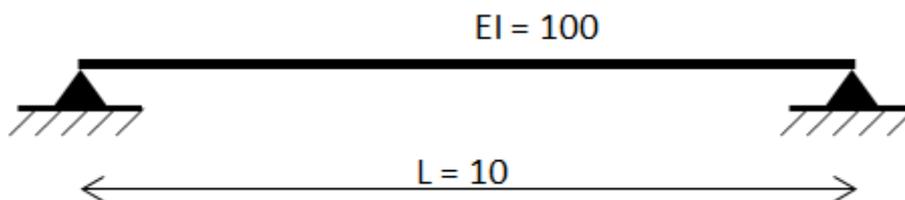
Conclusion

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%

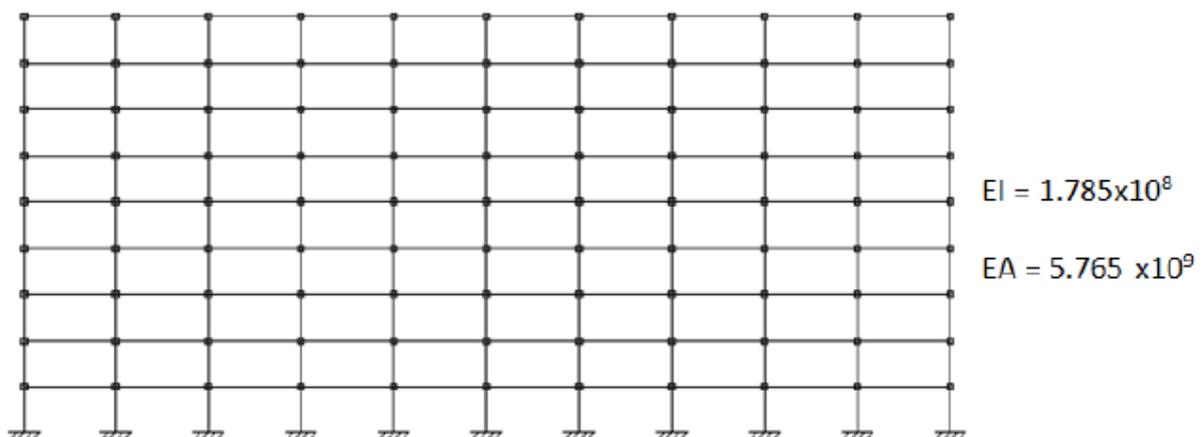
Conclusion

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.

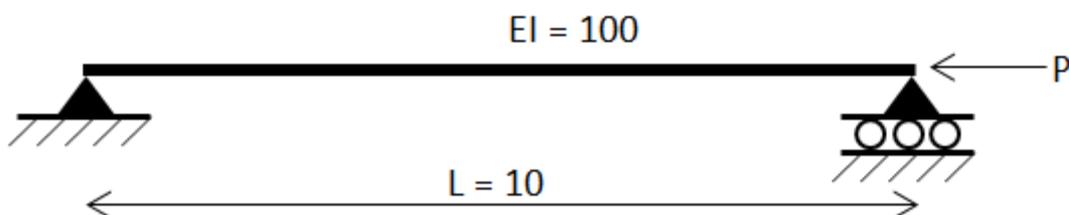
References

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%

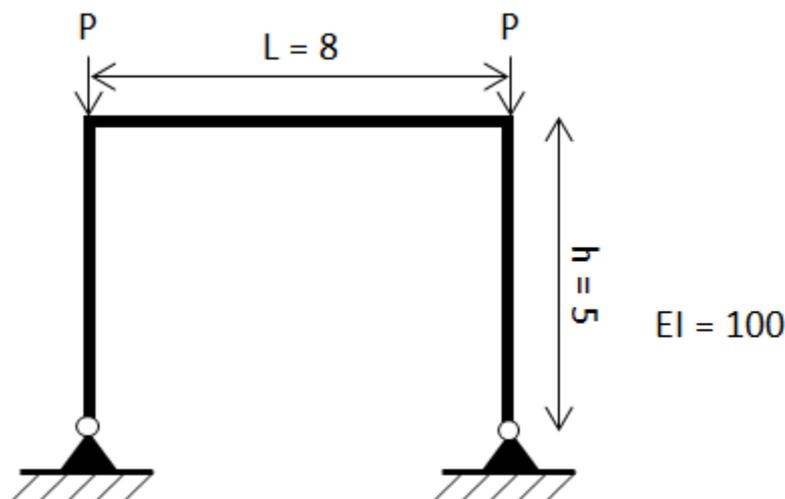
Conclusion

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 -ASCE

ASCE7 Loading

This handbook provides a general overview of how loadcases and combinations are created in *Tekla Structural Designer* when the head code is set to United States(ACI/AISC). The ASCE7 Combination Generator is also described.

Load Cases

Loadcase Types

The following load case types can be created:

Loadcase Type	Calculated Automatically	Include in the Combination Generator	Live Load Reductions	Pattern Load
self weight (beams, columns and walls)	yes/no	yes/no	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
live	N/A	yes/no	yes/no	yes/no
roof live	N/A	yes/no	yes/no	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

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.

Live and Roof Live Loads

Live Load Reductions

Reductions can be applied to roof live and live loads to take account of the unlikelihood of the whole building being loaded with its full design live load. The reduction is calculated based on total floor area supported by the design member. Roof live and live load types each have their own reductions applied in accordance with either Section 4.8 and 4.9 of ASCE 7-05, or Section 4.7 and 4.8 of ASCE 7-10 as appropriate.

Due to the complications associated with live load reduction when considering members at any angle to the vertical or horizontal, reductions are only applied to:

- Horizontal steel beams with vertical webs (major axis horizontal) which are set to be 'gravity only' pin ended only
- Vertical columns only (both RC and steel)
- Vertical walls only (RC)

If a level is not set to be a floor (in the **Construction Levels** dialog) then no reductions are accounted for at that level and it will not be counted as a floor in determining the amount of reduction to make.

Live Load Reduction Factor

The live load reduction factor, R is calculated as follows:

$$R = (0.25 + 15 / \text{Sqrt}(K_{LL} * A_T)) - \text{where } R \leq 1.0 \quad \text{US-units}$$

$$R = (0.25 + 4.57 / \text{Sqrt}(K_{LL} * A_T)) \quad \text{metric-units}$$

K_{LL} comes from Table 4-2 in ASCE7-05/ASCE7-10. Essentially:

Interior and exterior cols (no cantilever slabs) $K_{LL} = 4$

Edge and interior beams (no cantilever slabs) $K_{LL} = 2$

Interior beams (with cantilever slabs) $K_{LL} = 2$

Cantilever beams $K_{LL} = 1$

Edge cols (with cantilever slabs) $K_{LL} = 3$

Corner cols (with cantilever slabs) $K_{LL} = 2$

Edge beams (with cantilever slabs) $K_{LL} = 1$

For all beams and column stacks supporting one floor $R \geq 0.5$

For all column stacks supporting two or more floors $R \geq 0.4$



As it is not possible to automatically assess where cantilever slabs are and what they are attached to - the K_{LL} factor can be manually specified for individual column and wall stacks and beam spans.

Roof Live Load Reduction Factor

The roof live load reduction factor is calculated as follows:

$$R = R_1 * R_2$$

where

$$R_1 = 1.2 - 0.001 * A_T, \text{ where } 1.0 \geq R_1 \geq 0.6 \quad \text{US-units}$$

$$= 1.2 - 0.011 * A_T \quad \text{metric-units}$$

$$R_2 = 1.0 \text{ (conservatively assumes roofs } < 18 \text{ degs)}$$

Wind Loads

The ASCE7 Wind Wizard



The Wind Wizard is fully described in the Wind Modeling Engineer's Handbook.

The **Wind Wizard** assesses wind loading on your building structure via a choice of methods:

- Directional Procedure Part 1 - Rigid Buildings of All Heights (Chapter 27)
- Envelope Procedure Part 1 - Low-Rise Buildings (Chapter 28)

Wind load cases can then be generated and combined with other actions due to dead and imposed loads in accordance with Section 2.3.2 of ASCE7-10

in order to run the wind wizard the following assumptions/limitations exist:-

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

Simple Wind Loading

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

Patterning of Live Loads

ASCE7 pattern loading for LRFD combinations is as follows:

Code Clase	Load Combination	Loaded Spans	UnLoaded Spans
LRFD	$1.2D + 1.6L + 0.5Lr$	$1.2D + 1.6L + 0.5Lr$	$1.2D + 0.5Lr$

Combinations

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...**

Application of Notional Loads in Combinations

Notional loads are applied to the structure in the X and Y global directions but then need to be combined to act in the +Dir1, -Dir1, +Dir2 and -Dir2 directions, because many structures do not have their primary axes along X and Y.

This is achieved by applying the loads themselves in global X and Y and then using the combination factors to set them in Dir1 and Dir2 as required.

So if the angle between X and Dir1 is +60 degs - *Tekla Structural Designer* applies +X factor 0.5 and +Y factor 0.866.

The net result is that any combination is able to have up to 2 Notional Loads applied within it - one from X (+ or -) and one from Y (+ or -).

In addition, you are able to set up the combinations manually and apply factors to each as required.

The Combinations Generator

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



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

Combination Generator - Combinations

The first page of the generator lists suggested ASD and LRFD combinations (with appropriate factors).

The 'Generate' check boxes are used to select those combinations to be considered.

Combination Generator - Service

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

Combination Generator - NL

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

Any combination with wind in is automatically greyed.

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

Combination Classes

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



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

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

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

Construction Stage Combination

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

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



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

Gravity Combinations

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

They are used in the Gravity Sizing processes as follows:

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

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

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

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

Lateral Combinations

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

They are used during the Full Design processes as follows:

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

Seismic Combinations

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

Vibration Mass Combinations

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



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

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

Concrete Design - ACI

Beam Design to ACI 318

Limitations and Exclusions

The following general exclusions apply.

the current release will not:

- design beams as 'deep beams' - beams classified as 'deep' are designed as if they are regular beams and a warning is displayed.



Deep beams according to ACI 318 are:

(a) Members with clear spans equal to or less than 4 times overall member depth

(b) Members with concentrated loads within twice the member depth from the support

- design beams in lightweight concrete
- design beams with coated reinforcement
- design beams with stainless steel
- design prestressed concrete
- design structures subject to very aggressive exposure
- design watertight structures

material limitations for concrete:

- for structural concrete compressive strength of concrete f_c' shall not be less than 17 MPa (2500psi)
- durability requirements are not implemented

material limitations for reinforcement:

- the values of specified yield strength of reinforcement; f_y and f_{yt} used in calculations shall not exceed 550 MPa (80000psi)
- specified yield strength of non-prestressed reinforcement; f_y and f_{yt} shall not exceed 420 MPa (60 000 psi) in design of shear or torsion reinforcement
- wire reinforcement design is not implemented

Slender Beams

Spacing of lateral supports for a beam shall not exceed $50 \cdot b$. In the program the lateral supports will be taken as the distance between the faces of the supports.

Spacing of lateral supports $< 50 \cdot b$

where

b = the least width of compression flange or face

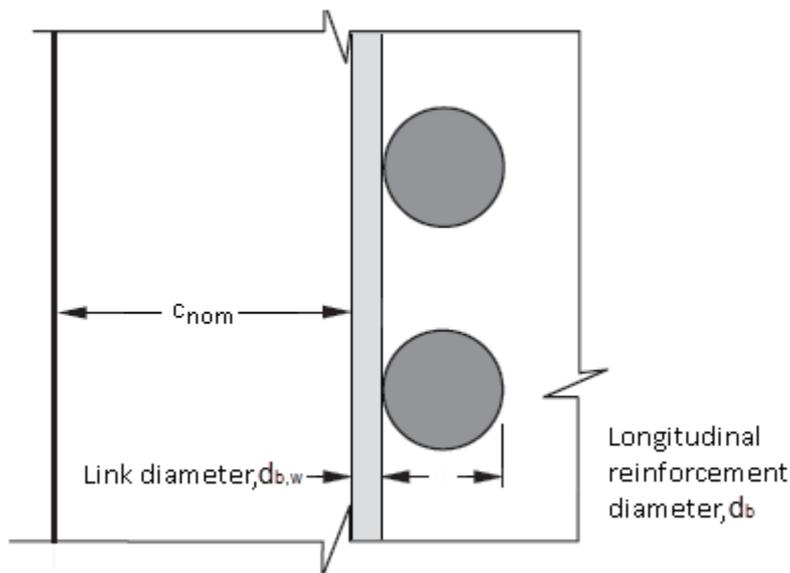
b = web width b_w - for simplification

IF above check fails then a Warning and a text 'Slender span' or 'Over wide distance between lateral supports' is displayed.

Effects of lateral eccentricity of load are considered in determining spacing of lateral supports.

Cover to Reinforcement

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



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

These values are then checked against the nominal limiting cover, $c_{nom,lim}$

If $c_{nom,u} < \text{MAX}(c_{nom,lim}, d_b)$ then a warning is displayed in the calculations.

Design Parameters for Longitudinal Bars

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

Minimum and Maximum Diameter of Reinforcement

IF torsional reinforcement is required, there shall be at least one longitudinal bar in every corner of the stirrups. Longitudinal bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than 9 mm (3/8 in).

The maximum diameters of reinforcement to be used in the various locations is set by the user.

Standard hooks for stirrups and ties are limited to No.8 bars, $d_b=25\text{mm}$ (1.0in.) and smaller.

And the 90-degree hook with $6d_b$ extension is further limited to No. 5, $d_b=16\text{mm}$ (0.625in.) bars and smaller.

For primary reinforcement there is no limit on bar size.

Minimum Distance between Bars

The minimum clear spacing between parallel bars in a layer, $s_{cl,min}$, is given by;

$$s_{cl,min} \geq \text{MAX}[d_b, 25\text{mm}] \quad \text{metric-units}$$

$$s_{cl,min} \geq \text{MAX}[d_b, 1\text{in}] \quad \text{US-units}$$

IF the above check fails then a Warning is displayed.

Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above the bars in the bottom layer with clear distance between layers not less than 25mm (1in.).

Maximum Spacing of Tension Bars

The spacing of reinforcement closest to the tension face, s is given by;¹

$$s \leq \text{MIN}[380\text{mm} \cdot 280\text{MPa}/f_s - 2.5 \cdot c_c, 300\text{mm} \cdot (280\text{MPa}/f_s)] \quad \text{metric-units}$$

$$s \leq \text{MIN}[15\text{in} \cdot 40000\text{psi}/f_s - 2.5 \cdot c_c, 12\text{in} \cdot (40000\text{psi}/f_s)] \quad \text{US-units}$$

where:

c_c = the least distance from surface of reinforcement to the tension face

f_s = calculated stress in reinforcement at service load; it shall be permitted to take

$$=(2/3) \cdot f_y \cdot (A_{s,reqd}/A_{s,prov})$$

IF the above check fails then a Warning is displayed

IF torsional reinforcement is required:

the longitudinal reinforcement required for torsion shall be distributed around the perimeter of the closed stirrups with maximum spacing of 300mm (12 in.)²

1. ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 10.6.4

2. ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 10.5.6.2

Minimum Area of Reinforcement

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

$$A_{s,min} \geq \text{MAX}[(f_c^{0.5}/(4*f_y))*b_w*d, 1.4\text{MPa}*b_w*d/f_y] \quad \text{metric-units}$$

$$A_{s,min} \geq \text{MAX}[(3*f_c^{0.5}/f_y)*b_w*d, 200\text{psi}*b_w*d/f_y] \quad \text{US-units}$$

where

f_c' = specified compressive strength of concrete

f_y = specified yield strength of reinforcement

b_w = web width; for statically determinate members with a flange in tension $b_w = \text{MIN}(2*b_w, b_{eff})$ ^A

d = distance from extreme compression fiber to centroid of longitudinal compression reinforcement

^A Assumption; the member is statically determinate in design

Eq. above is to be provided wherever reinforcement is needed, except where such reinforcement is at least one-third greater than that required by analysis;

IF $A_{s,prov} < 4/3 * A_s$

THEN $A_{s,min}$ is calculated as eq. above

ELSE $A_{s,min}$ not required

IF the above check fails then a Warning is displayed.

1. ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 10.5.1

Maximum Area of Reinforcement

IF flexural members with factored axial compression $< 0.10*f_c'*A_g$ (*Assumption; this always applies*)

THEN net tensile strain in extreme layer of longitudinal tension steel, ϵ_t shall not be less than 0.004;

$$\epsilon_t \geq 0.004$$

$$A_{s,max} \leq 0.85*(f_c'/f_y)*\beta_1*b_w*d*[0.003/(0.003+0.004)]$$
^A

$$\leq 0.85*(f_c'/f_y)*\beta_1*b_w*d*(3/7)$$

where

A_g = the gross area of the concrete section

β_1 = stress block depth factor^B

metric units

$$\begin{aligned} &= 0.85 && \text{for } f'_c \leq 28\text{MPa} \\ &= 0.85 - 0.05 * [(f'_c - 28\text{MPa})/7\text{MPa}] && \text{for } 28\text{MPa} < f'_c < 55\text{MPa} \\ &= 0.65 && \text{for } f'_c \geq 55\text{MPa} \end{aligned}$$

US-units

$$\begin{aligned} &= \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) \\ &= 0.85 && \text{for } f'_c \leq 4000 \text{ psi} \\ &= 0.85 - 0.05 * [(f'_c - 4\text{ksi})/1\text{ksi}] && \text{for } 4000 \text{ psi} < f'_c < 8000\text{psi} \\ &= 0.65 && \text{for } f'_c \geq 8000 \text{ psi} \end{aligned}$$

^A Notes on ACI 318-08 Chap. 6 Section 10.3.5

^B ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 10.2.7.3

ELSE a Warning is displayed.

Side Skin Reinforcement in Beams

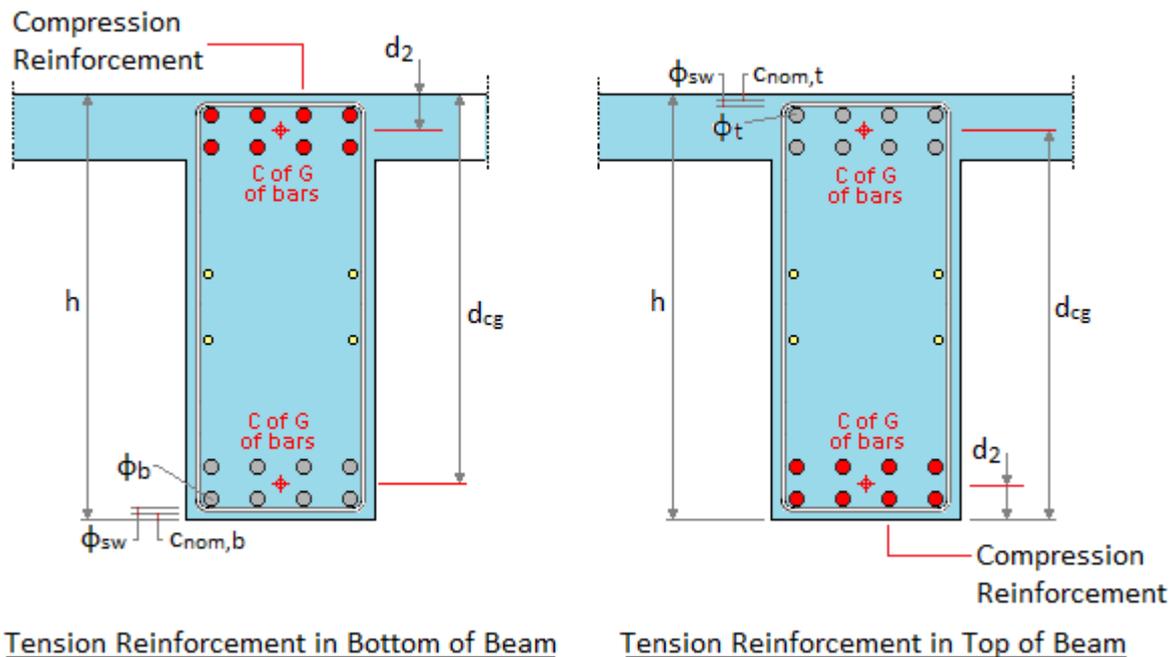
Where h of a beam or joist exceeds 900mm (36 in.), longitudinal skin (side) reinforcement shall be uniformly distributed along both side faces of the member.

The code requires that skin reinforcement shall extend for a distance $h/2$ from the tension face. Regardless of this in the first release the skin reinforcement is provided to the full height of the beam.

Effective Depth of Section

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

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



Design for Bending

Design Check Regions

Design checks are carried out in various regions along the length of the beam with the regions being user defined proportions of the clear span of the beam.

The design value of the bending moment used for the design in a region is the maximum factored bending moment arising from each load combination obtained from the building and grillage analysis in the region under consideration.

In monolithic construction the reinforcement in the top of the beam at the support should be designed for a bending moment, $M_{u,top}$ given by;

$$M_{u,top} = \beta_1 * M_{u,maxspan}$$

where

β_1 = the appropriate fixity coefficient

$M_{u,maxspan}$ = the maximum positive moment in the beam span (excluding support positions)

The design moment to be used for the first and last support regions for **all** beams is therefore given by;

$$M_{u,T1} = \text{MAX}(M_{u,T1}, \beta_1 * M_{u,maxspan})$$

and

$$M_{u,T5} = \text{MAX}(M_{u,T5}, \beta_1 * M_{u,maxspan})$$

Design for Bending for Rectangular Sections

Determine if compression reinforcement is needed¹;

Nominal strength coefficient of resistance is given;

$$R_n = M_u / (\phi * b * d^2)$$

where

M_u = factored moment at section

d = depth to tension reinforcement

b = width of the compression face of the member

ϕ = strength reduction factor^A

= 0.9 (corresponds to the tension-controlled limit)

^A ACI 318-08:2008 and ACI 318-11:2011 Section 9.3

IF $R_n \leq R_{nt}$ THEN compression reinforcement is not required.

IF $R_n > R_{nt}$ THEN compression reinforcement is required.

where

R_{nt} = Limit value for tension controlled sections without compression reinforcement for different concrete strength classes^A

$$= \omega_t * (1 - 0.59 \omega_t) * f'_c$$

f'_c = compressive strength of concrete

$$\omega_t = 0.319 * \beta_1$$

β_1 = stress block depth factor^B

metric-units

$$= 0.85$$

for $f'_c \leq 28\text{MPa}$

$$= 0.85 - 0.05 * [(f'_c - 28\text{MPa}) / 7\text{MPa}]$$

for $28\text{MPa} < f'_c < 55\text{MPa}$

$$= 0.65$$

for $f'_c \geq 55\text{MPa}$

Reference Guides (ACI AISC)

US-units

$$\begin{aligned}
 &= \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) \\
 &= 0.85 && \text{for } f'_c \leq 4000 \text{ psi} \\
 &= 0.85 - 0.05 \times [(f'_c - 4 \text{ ksi}) / 1 \text{ ksi}] && \text{for } 4000 \text{ psi} < f'_c < 8000 \text{ psi} \\
 &= 0.65 && \text{for } f'_c \geq 8000 \text{ psi}
 \end{aligned}$$

^A Notes on ACI 318-08 Section 10.3.4

^B ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 10.2.7.3

Compression reinforcement is not required

The tension reinforcement ratio is given by²;

$$\rho = 0.85 \times f'_c / f_y \times [1 - (1 - 2 \times R_n / (0.85 \times f'_c))^{0.5}] \leq \rho_t = 0.319 \times \beta_1 \times f'_c / f_y$$

where

$$f_y = \text{yield strength of reinforcement}$$

The area of tension reinforcement required is then given by;

$$A_s = \rho \times b \times d$$

Compression reinforcement is required

The area of compression reinforcement required is then given by³;

$$A_s' = M_n' / [(d - d') \times f_s']$$

where

$$M_n' = M_n - M_{nt}$$

$$= (M_u / \phi) - M_{nt}$$

$$M_{nt} = \text{nominal moment resisted by the concrete section⁴}$$

$$= R_n \times b \times d^2$$

The area of tension reinforcement required is then given by⁵;

$$A_s = A_s' \times f_s' / f_y + \rho \times b \times d$$

where

$$f_s' = \text{MIN}[E_s \times (\epsilon_u \times (c - d') / c), f_y]$$

$$\rho = \rho_t \times (d_t / d)$$

$$\rho_t = 0.319 \times \beta_1 \times f'_c / f_y$$

$$\epsilon_u = 0.003^6$$

$$c = 0.375 * d_t$$

1. Notes on ACI 318-08 Chap 7.

2. Notes on ACI 318-08 Section 7 Eq.(3)

3. Notes on ACI 318-08 Section 7

4. Notes on ACI 318-08 Section 10.3.4

5. Notes on ACI 318-08 Chap. 7

6. ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 10.2.3

Design for Bending for Flanged Sections

IF $h_f < 0.5 * b_w$ THEN treat the beam as rectangular¹

where

b_w = web width

Depth of the equivalent stress block is given²:

$$a = \rho * d * f_y / (0.85 * f'_c) (= 1.18 * \omega * d)$$

where

$$\rho = 0.85 * f'_c / f_y * [1 - (1 - 2 * R_n / (0.85 * f'_c))^{0.5}]$$

$$R_n = (M_u / \phi) / (b_{eff} * d^2) \quad \text{assumption } \phi = 0.9$$

IF $a \leq h_f$ THEN the rectangular compression block is wholly in the depth of the flange and the section can be designed as a rectangular section with tension reinforcement only by setting $b = b_{eff}$ and checking the ϕ -factor as followed;

IF $(a/\beta_1)/d < 0.375$ **THEN** $\phi=0.9$ (section tension controlled)

IF $0.375 > (a/\beta_1)/d < 0.600$ **THEN** $\phi=0.7 + (\epsilon_t - 0.002) * (200/3)$

IF $(a/\beta_1)/d > 0.6$ **THEN** $\phi=0.65$ (section comp. controlled)

where

$$\epsilon_t = [(d * \beta_1) / a - 1] * 0.003$$

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;

Required reinforcement is given;

$$A_{sf} = 0.85 * f'_c * (b_{eff} - b) * h_f / f_y$$

Nominal moment strength of **flange**;

$$M_{nf} = [A_{sf} * f_y * (d - h_f / 2)]$$

Required nominal moment strength to be carried by the beam **web** is given;

$$M_{nw} = M_u - M_{nf}$$

Can be written as;

$$M_{nw} = M_u - [(0.85 * f'_c * (b_{eff} - b) * h_f / f_y) * f_y * (d - h_f / 2)] = M_u - [(0.85 * f'_c * (b_{eff} - b) * h_f) * (d - h_f / 2)]$$

Reference Guides (ACI AISC)

Reinforcement A_{sw} required to develop the moment strength to be carried by the web;

$$A_{sw} = \omega_w * f'_c * b * d / f_y$$

where

$$\omega_w = \rho * f_y / f'_c = 0.85 * f'_c / f_y * [1 - (1 - 2 * (M_{nw} / (b * d^2)) / (0.85 * f'_c)^{0.5})] * f_y / f'_c$$

Can be written as;

$$A_{sw} = b * d * 0.85 * f'_c / f_y * [1 - (1 - 2 * (M_{nw} / (b * d^2)) / (0.85 * f'_c)^{0.5})]$$

Total required reinforcement is given;

$$A_s = A_{sf} + A_{sw}$$

Check to see if the section is tension-controlled;

IF

$$\rho_w \leq \rho_t \text{ section is tension- controlled } (\phi=0.9)$$

ELSE add compression reinforcement where

$$\rho_w = \omega_w f'_c / f_y \quad \rho_t = 0.319 * \beta_1 * f'_c / f_y$$

Can be simplified as;

$$\omega_w \leq 0.319 * \beta_1 \text{ section is tension- controlled } (\phi=0.9)$$

ELSE add compression reinforcement

1. ACI 318-08:2008 and ACI 318-11:2011 Section 8.12.4

2. Notes on ACI 318-08 Section 7 (1)

Design for Shear

Shear Strength

Determine shear strength provided by the concrete¹;

Members subject to axial compression not applied at this stage.

$$\phi V_c = \phi * 0.17 * \lambda * f'_c{}^{0.5} * b_w * d = \phi * 0.17 * \lambda * f'_c{}^{0.5} * b_w * d \quad \text{metric- units}$$

$$\phi * 2 * \lambda * f'_c{}^{0.5} * b_w * d = \phi * 2 * \lambda * f'_c{}^{0.5} * b_w * d \quad \text{US-units}$$

where

$$\phi = 0.75 \text{ for shear}^A$$

$$\lambda = 1.0 \text{ for normal weight concrete}^B$$

$$f'_c{}^{0.5} = \text{square root of specified compressive strength of concrete.}^C$$

^A ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 9.3.2.3

^B ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 8.6.1

^C ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.1.2.1

Note; If the structure is defined as a joist construction V_c shall be permitted to be 10% more than that specified in above².

$$\phi V_{c,j} = 1.1 * \phi V_c$$

IF

$$\begin{aligned} V_u - \phi V_c &\leq \phi * 0.66 * f_c^{0.5} * b_w * d^A = \phi * 0.66 * f_c^{0.5} * b_w * d && \text{metric-} \\ &&& \text{units} \\ &= \phi * 8 * f_c^{0.5} * b_w * d = \phi * 8 * f_c^{0.5} * b_w * d && \text{US-units} \end{aligned}$$

where

V_u = the maximum design shear force acting anywhere on the beam

^A ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.7.9

THEN the shear design process can proceed.

ELSE the shear design process **FAILS** since the section size or strength of the concrete is inadequate for shear. No further shear calculations are carried out in the region under consideration and the user is warned accordingly.

The design shear capacity of the minimum area of shear links actually provided, $V_{s,min}$ is given by³;

$$V_{s,min} = (A_{v,min}/s) * \phi * d * f_{yt}$$

where

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

For each beam determine the following;

$V_{u,maxL}$ = the maximum vertical shear force at the face of the left hand support

$V_{u,dL}$ = the vertical shear force at a distance dL from the face of the left hand support

$V_{u,maxR}$ = the maximum vertical shear force at the face of the right hand support

$V_{u,dR}$ = the vertical shear force at a distance dR from the face of the right hand support

$V_{u,S2L}$ = the maximum vertical shear force at the extreme left of region S2

$V_{u,S2R}$ = the maximum vertical shear force at the extreme right of region S2

Reference Guides (ACI AISC)

where

dL = the minimum effective depth of the beam in regions T1 and B1

dR = the minimum effective depth of the beam in regions T5 and B3

For region S1;

IF

$$\text{ABS}(V_{u,\text{maxL}}) - \text{ABS}(V_{u,dL}) \leq 0.25 * \text{ABS}(V_{u,\text{maxL}})$$

THEN

$$V_{u,S1} = \text{ABS}(V_{u,dL})$$

ELSE

$$V_{u,S1} = \text{ABS}(V_{u,\text{maxL}})$$

For region S3;

IF

$$\text{ABS}(V_{u,\text{maxR}}) - \text{ABS}(V_{u,dR}) \leq 0.25 * \text{ABS}(V_{u,\text{maxR}})$$

THEN

$$V_{u,S3} = \text{ABS}(V_{u,dR})$$

ELSE

$$V_{u,S3} = \text{ABS}(V_{u,\text{maxR}})$$

For region S2;

IF

$$\text{ABS}(V_{u,\text{maxL}}) - \text{ABS}(V_{u,dL}) \leq 0.25 * \text{ABS}(V_{u,\text{maxL}})$$

$$V_{u,S2L} = \text{MIN}[\text{ABS}(V_{u,S2L}), \text{ABS}(V_{u,dL})]$$

ELSE

$$V_{u,S2L} = V_{u,S2L}$$

IF

$$\text{ABS}(V_{u,\text{maxR}}) - \text{ABS}(V_{u,dR}) \leq 0.25 * \text{ABS}(V_{u,\text{maxR}})$$

$$V_{u,S2R} = \text{MIN}[\text{ABS}(V_{u,S2R}), \text{ABS}(V_{u,dR})]$$

ELSE

$$V_{u,S2R} = V_{u,S2R}$$

The absolute maximum vertical shear force in the region is then given by;

$$V_{u,S2} = \text{MAX}[\text{ABS}(V_{u,S2L}), \text{ABS}(V_{u,S2R})]$$

In any region, *i*;

IF

$$V_{u,i} \leq V_{s,min} + \phi V_c$$

where

$V_{u,i}$ = the maximum shear in region i from the above routines

OR

The structure is defined as a joist construction⁴.

THEN

Minimum shear reinforcement shall be used;

And the nominal shear strength is given;

$$\phi V_n = \phi V_c + V_{s,min}$$
⁵

ELSE

$$V_{u,i} > V_{s,min} + \phi V_c$$

THEN shear links are required in the region.

The area of shear reinforcement required is then given⁶;

metric-units;

$$(A_v/s)_{Si} = \text{MAX}[(V_u - \phi V_c) / (\phi f_{yt} d), 0.062 f_c^{0.5} b_w / f_{yt}, 0.35 \rho_a b_w / f_{yt}]$$

US-units;

$$(A_v/s)_{Si} = \text{MAX}[(V_u - \phi V_c) / (\phi f_{yt} d), 0.75 f_c^{0.5} b_w / f_{yt}, 50 \rho_a b_w / f_{yt}]$$

$$V_s = (A_v/s) \phi d f_{yt}$$

IF

$$V_s \leq 0.66 f_c^{0.5} b_w d \quad \text{metric -units} \quad 8 f_c^{0.5} b_w d \quad \text{US-units}$$

THEN the shear design process passes.

And the nominal shear strength is given;

$$\phi V_n = \phi V_c + V_s$$

ELSE the shear design process FAILS since the section size or strength of the concrete is inadequate for shear.

^{1.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.2.1.1

^{2.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 8.13.8

^{3.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.7.2

^{4.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.6.1 - Terms (d) and (e) not applied at this stage.

^{5.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.1

^{6.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.6.3

^{7.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.7.9

Minimum Area of Shear Reinforcement

The minimum area of shear reinforcement required, $A_{v,min}$ is given by¹;

$$A_{v,min} = \text{MAX}(0.062 * f_c^{0.5} * b_w * s / f_{yt}, 0.35 \text{MPa} * b_w * s / f_{yt}, A_{v,min,u}) \text{ metric-units}$$

$$\text{MAX}(0.75 * f_c^{0.5} * b_w * s / f_{yt}, 50 \text{psi} * b_w * s / f_{yt}, A_{v,min,u}) \text{ US-units}$$

where

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

f_{yt} = yield strength of transverse reinforcement

$A_{v,min,u}$ = 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

^{1.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.6.3

Spacing of Shear Reinforcement

For Longitudinal spacing, s between the legs of shear reinforcement is given by¹;

IF

$$V_u - \phi * V_c \leq \phi * 0.33 * f_c^{0.5} * b_w * d \text{ metric-units}$$

$$\phi * 4 * f_c^{0.5} * b_w * d \text{ US-units}$$

THEN

$$s_{min,u} \leq s \leq \text{MIN}[0.5 * d, 600 \text{mm (24in.)}, s_{max,u}]$$

ELSE

$$s_{min,u} \leq s \leq \text{MIN}[0.25 * d, 300 \text{mm (12in.)}, s_{max,u}]$$

where

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

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

Moreover IF **compression reinforcement** is required the compression reinforcement shall be enclosed by ties². This is an additional limit, not an alternative.

Vertical spacing of ties is then given by³;

$$s \leq \text{MIN}(16 * d_b, 48 * d_{b,w}, b_w, h)$$

where

d_b = the nominal diameter of the bar

$d_{b,w}$ = the nominal diameter of the link reinforcement

^{1.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 11.4.5

^{2.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 7.11.1

^{3.} ACI 318-08:2008, ACI 318-11:2011 and ACI 318M-11:2011 Section 7.10.5.2

Column Design to ACI 318

Limitations and Exclusions

The following general exclusions apply to the first release:

- 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

Concrete

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

Reinforcement

The reinforcement options are:

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

Cover to Reinforcement

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

Minimum Longitudinal Total Steel Area¹

For design to ACI

1% * column area

Maximum Longitudinal Total Steel Area

For design to ACI

8% * column area

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

Ultimate Axial Load Limit

The strength of a column under truly concentric axial load is

$$P_{no} = 0.85 \cdot f_c \cdot (A_g - A_{st}) + f_y \cdot A_{st}$$

For nonprestressed compression members with tie reinforcement,

$$\phi P_{nmax} = 0.80 \cdot \phi \cdot [0.85 \cdot f_c \cdot (A_g - A_{st}) + f_y \cdot A_{st}]$$

where

$0.85f_c$ = maximum concrete stress permitted in column design

A_g = gross area of the section (concrete and steel)

f_y = yield strength of the reinforcement

A_{st} = total area of reinforcement in the cross section

ϕ = strength reduction factor = 0.65

Effective Length Calculations**Unsupported Length**

The unsupported length, l_u , of a column is the clear distance between lateral supports.

If, at an end of the compression member (stack), no effective beams, flat slab or slab on beams to include is found, then the clear height includes the (compression member) 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). If there is no stack beyond this restraint (i.e. this is the end of the column), the clear height ends at the end of the column.

Effective Length

The effective length, l_e , is calculated automatically from ACI R10.10.1. You have the ability to override the calculated value.

Tekla Structural Designer will impose the following limits for stacks that are designated as braced:

$$0.5 \leq l_e / l_u \leq 1$$

When both ends of an unbraced compression member are hinged (pinned), a "Beyond Scope" warning is displayed.

The effective length of the stack (compression member) is given by:

$$l_e = k * l_u$$

The program uses the bottom end of the stack (compression member) as end 1 and the top as end 2.

Any beams framing into the end of the compression member (stack) within 45 degrees of the axis being considered are said to be restraining beams for the stack in that direction. No adjustment is made to the restraint provided by the beam for the angle (i.e. the full value of "E * I / I" is used for all beams within 45 degrees of the axis).

A beam is to be considered as a restraining beam in the direction considered if:

$$-45^\circ < \beta \leq 45^\circ$$

where

β is the angle from the axis in the direction considered to the beam, measured anti-clockwise when viewed from above (i.e. back along the length of the stack from the end towards the start).

There is a lower limit¹ for the value of ψ that is used in the equations for calculating k:

$$\psi \geq 0.20$$

Additionally, *Tekla Structural Designer* will impose an upper limit:

$$\psi \leq 1000$$

Fixed Column Base

$\psi \geq 0.20$ for fixed bases in *Tekla Structural Designer*. Suggested value is from RC Mechanics by McGregor and Wright, 6e.

Pinned Column End

In any situation where the end of a column anywhere in the structure is pinned, $\psi = 1000$.

No Effective Beams Found

If no effective beams are found to restrain the end of the stack in the direction in question, then the program will consider whether there is a flat slab restraining the stack at this end. If a flat slab is found it will either be considered as a restraint, or not,

in each direction at each end of the stack - this is controlled by checking the option **Use slab for calculation...** located as a 'Stiffness' setting in the column properties.

If there are no effective beams and there is no flat slab (or any flat slab is not to be considered), then the program will look for a slab on beams. If a slab on beams is found, this acts as a restraint at the position. Slabs on beams will only be considered if the 'Use slab for calculation...' option is selected, as is the case for flat slabs.

If no beams and no flat slab or slab on beams is found, then the program will look for the far end of the stack on the other side of the joint, and look at the restraints there, and so on until a restraint with an effective beam, flat slab or slab on beams to be considered is found.

If the stack is restrained by a flat slab, then the slab will be considered to act as a beam in this direction - note that it is one beam in the direction and NOT a beam on each side of the column. The beam's length will be said to be four times its width, and therefore $(E * I / l)$ can be calculated in this direction.

If the stack is restrained by a slab on beams, this will have a zero contribution to the stiffness. This has the effect of setting $\psi = \text{infinity}$, though it is limited to 1000 before being used in the calculations.

If the stack is an end stack and there are no supports, beams, flat slabs or slabs on beams considered to restrain the stack at this end in the direction, the end is therefore free in this direction and $\psi = 1000$.

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

Column Stack Classification

Slenderness ratio

The slenderness ratio, $k l_u / r$, of the restrained length (note: not necessarily the stack length - it will be longer if there is no restraint at either end of the stack) about each axis is calculated as follows:

$$(k l_u / r)_y = k * l_{u_y} / (\sqrt{I_y / A})$$

$$(k l_u / r)_z = k * l_{u_z} / (\sqrt{I_z / A})$$

where

slenderness ratio = $k * l_u / r$,

k is an effective length factor

l_{u_y} is the unsupported column length in respect of major axis (y axis)

l_{u_z} is the unsupported column length in respect of minor axis (z axis)

r_y is the radius of gyration of the column in the y-direction

r_z is the radius of gyration of the column in the z-direction

I_y is the second moment of area of the stack section about the major axis (y axis)

Reference Guides (ACI AISC)

I_z is the second moment of area of the stack section about the major axis (y axis)

A_g is the cross-sectional area of the stack section

For unbraced columns

IF $(k l_u/r)_y \leq 22$

THEN slenderness can be neglected and column can be designed as short column

ELSE, column is considered as slender

IF $(k l_u/r)_z \leq 22$

THEN slenderness can be neglected and column can be designed as short column

ELSE, column is considered as slender

For braced columns

IF $(k l_u/r)_y \leq \text{MIN}((34-12*M_1/M_2), 40)$

THEN slenderness can be neglected and column can be designed as short column

ELSE, column is considered as slender

IF $(k l_u/r)_z \leq \text{MIN}((34-12*M_1/M_2), 40)$

THEN slenderness can be neglected and column can be designed as short column

ELSE, column is considered as slender

where

M_1 = the smaller factored end moment on the column, to be taken as positive if member is bent in single curvature and negative if bent in double curvature

= $\text{MIN} [\text{ABS}(M_{\text{top}}), \text{ABS}(M_{\text{bot}})]$

M_2 = the larger factored end moment on the column always taken as positive

= $\text{MAX} [\text{ABS}(M_{\text{top}}), \text{ABS}(M_{\text{bot}})]$

Design Moments

M_{min} = design moment

= $P_u*(0.6+0.03*h)$ in

US units

= $P_u*(15+0.03*h)$ mm

metric
units

where

h = the major dimension of the column in the direction under consideration

P_u = The max compression force at any design position in the stack under consideration. If stack is in tension set to zero

P_c = critical buckling load

$$= \pi^2(EI)/(k*lu)^2$$

where EI can be computed by Eq. (10-14) or Eq (10-15)

Design for Shear

Design Parameters for Shear Design

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

Minimum Shear Link Diameter

For Ties, minimum shear reinforcement size

IF maximum longitudinal bar \leq 1.27 in. (32.3mm)

shear reinforcement diameter = 0.375 in. (9.5mm)

Minimum shear reinforcement diameter = 0.50 in. (12.7mm)

Maximum Span Region Shear Link Spacing

Controlled by seismic detailing requirements.

Maximum Support Region Shear Link Spacing

Controlled by seismic detailing requirements.

Column Containment

The ACI requirement is that every alternate longitudinal bar should be restrained by a link corner or bar tie, and no bar should have more than 6" (150 mm) clear distance from a restrained bar.

Wall Design to ACI 318

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.

The reference codes are ACI 318-08, ACI 318-11, ACI 318M-11 together with the PCA Notes on ACI 318-08.

Limitations and Exclusions

The design of walls is limited to load bearing and shear walls. Other wall types: non-load bearing, tilt-up and plates that resist in-plane compression are beyond scope.

The following general exclusions also apply to the first release:

- 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,
- Multi-stack reinforcement lifts.

Materials

Concrete

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

Reinforcement

The reinforcement options are:

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

Cover to Reinforcement

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 horz bar or any link/containment transverse reinforcement that may be present.

You are required to set a minimum value for the nominal cover for each wall in the wall properties.

Design Parameters for Vertical Bars

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



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

Minimum and Maximum Vertical Bar Diameter

There are no code provisions, but user defined limits can be applied to the minimum and maximum bar diameters - specified in **Design Options > Wall > Reinforcement Layout**

Minimum and Maximum Vertical Loose Bar Spacing

Limiting minimum horizontal spacing of the vertical bars, $s_{v, \text{lim}, \text{min}}$ is controlled by the diameters of the 2 adjacent bars and aggregate size¹.

$$s_{v, \text{lim}, \text{min}} = 0.5 * (d_{bv, i} + d_{bv, (i+1)}) + c_{\text{gap}}$$

where

c_{gap} = min. clear distance bet. bars

$$c_{\text{gap}} = \max (1.5 * d_{bv, i}, 1.5 * d_{bv, (i+1)}, 1.33 * h_{\text{agg}}, 1.5 \text{ in.}) \quad \text{US units}$$

$$= \max (1.5 * d_{bv, i}, 1.5 * d_{bv, (i+1)}, 1.33 * h_{\text{agg}}, 38 \text{ mm}) \quad \text{metric units}$$

where

$d_{bv, i}$ and $d_{bv, (i+1)}$ = the diameters of the two adjacent vertical bars

h_{agg} = aggregate size

Limiting maximum horizontal spacing of the vertical bars, $s_{v, \text{lim}, \text{max}}$ is controlled by the wall thickness.

$$s_{v, \text{lim}, \text{max}} = \min (3 * h_w, 18 \text{ in.}) \quad \text{US units}$$

$$= \min (3 * h_w, 450 \text{ mm}) \quad \text{metric units}$$

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

Minimum and Maximum Reinforcement Area²

The code provisions which control the vertical reinforcement area are,

- 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 controlling values are:

IF $d_{bv} \leq$ No. 5 (No. 16) with $f_y \geq 60,000$ psi (420 MPa) OR WWR \leq W31 or D31

then $\rho_{v,lim,min} = 0.0012$ else 0.0015 for all other deformed bars

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

Total maximum area of vertical reinforcement, $A_{s,max} = \rho_{v,lim,max} * A_{cg} = 0.08 * A_{cg}$

where A_{cg} = Gross area of the concrete wall.

Where 2 layers are specified, this should be distributed equally to each face.

You are given further control over the minimum and maximum reinforcement ratio values via user limits in **Design Options > Wall > Reinforcement Layout**. These will be used if they are more onerous than the code limits.

[1.](#) Clause 7.6.3

[2.](#) 14.3.2

Design Parameters for Horizontal Bars

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

Minimum and Maximum Reinforcement Area¹

The code provisions which control the horizontal reinforcement area are,

- 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,
 $\rho_{h,lim,max}$

The controlling values are:

IF $d_{bv} \leq$ No. 5 (No. 16) with $f_y \geq 60,000$ psi (420 MPa) OR WWR \leq W31 or D31

THEN $\rho_{h,lim,min} = 0.002$ ELSE 0.0025 for all other deformed bars

Total minimum area of horizontal reinforcement, $A_{s,min} = \rho_{h,lim,min} * A_{cg}$

Total maximum area of vertical reinforcement, $A_{s,max} = \rho_{h,lim,max} * A_{cg} = 0.08 * A_{cg}$

where A_{cg} = Gross area of the concrete wall.

You can select a minimum ratio which will be the start point for the design in **Design Options > Wall > Reinforcement Layout**.

Minimum and Maximum Horizontal Bar Spacing

This is identical in principle to min vertical bar spacing.

Minimum and Maximum Containment Bar Spacing

There are Code provisions that control the maximum spacing:

The recommended values are,

Limiting maximum transverse spacing, $S_{w, \text{lim, max}} = \min(16*d_{bv}, 48*d_{bw, hw})$

[1.](#) 14.3.3

Ultimate Axial Load Limit

The axial resistance calculations for walls are the same as for columns.

Effective Length and Slenderness Calculations

The slenderness calculations for walls are generally the same as for columns, except that for walls:

Where the criteria for each axis is:

If $\lambda < \lambda_{\text{lim}}$, section is 'non-slender'

Elseif $\lambda \geq \lambda_{\text{lim}}$, section is 'slender'

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

In-plane,

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

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

Effective length, $l_{0,y}$

Length of wall panel, l_w

Out-of-plane,

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

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

Effective length, $l_{0,z}$

Thickness of wall panel, h_w

Pre-selection of Bracing Contribution:

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

In-plane direction, a wall is usually considered to be a bracing member.

Out-of-plane direction, a wall is usually considered to be braced by other stabilizing members.

These are the default settings but can be edited.

Resistance Moment Calculations

For each combination, a set of member end forces are returned from one or more sets of analyses, in the same way as for columns.

Design for Shear

In the plane of the wall the factored shear must be equal to or less than the design shear strength of the wall

$$V_u \leq \Phi V_n$$

The design shear strength of the wall is equal to the design shear strength of the concrete plus that of the shear reinforcing

$$V_u \leq \Phi V_c + \Phi V_s$$

The shear strength, V_n , may not be taken greater than $10\sqrt{f_c} h d$.

$$V_n \leq 10 \sqrt{f_c} h d \quad \text{US units}$$

$$V_n \leq 0.83 \sqrt{f_c} h d \quad \text{metric units}$$

where

h = wall thickness

$$d = 0.8 l_w$$

l_w = length of the wall

Out of plane the shear design calculations are the same whether the design element is a column or a wall.

Slab Design to ACI 318

Materials

Concrete

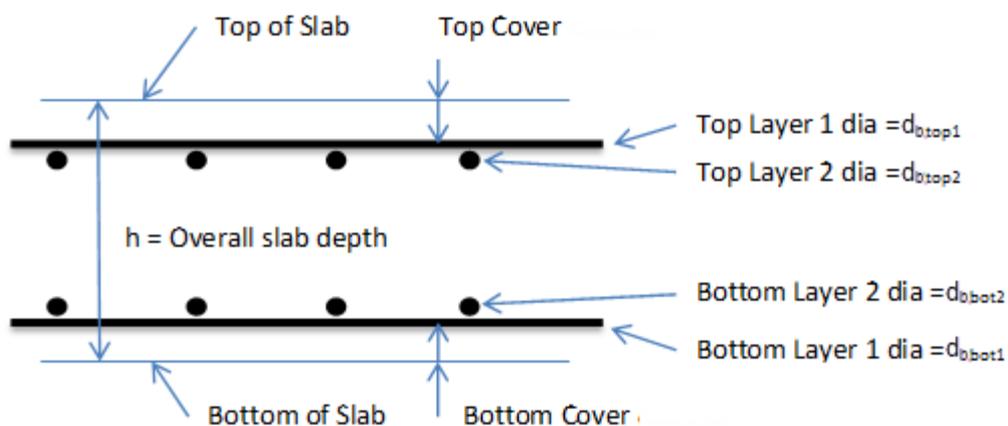
Only normal weight is included in the first release. (Lightweight concrete is excluded).

Reinforcement

The reinforcement options are:

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

Reinforcement Parameters



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:

d_b = the nominal diameter of the bar

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

$d_{b,top2}$ = the diameter of the **largest** longitudinal reinforcing bar in top layer 2

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

$d_{b,bot2}$ = 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

Reference Guides (ACI AISC)

b = unit width

For metric units:

the unit width of slab is 1m, and so the design cross section will always be a rectangular section where $b = 1000\text{mm}$

For US-customary units:

the unit width of slab is 1ft, and so the design cross section will always be a rectangular section where $b = 12\text{in.}$

Cover to Reinforcement

Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirements applies.

You are required to set a minimum value for the nominal cover for each slab panel. These values for top and bottom cover are specified in 'Reinforcement properties' section of the slab panel properties.

This value is then checked against the nominal limiting cover, which depends on the diameter of the reinforcement.

If the nominal cover is less than the limiting cover then a warning is displayed in the calculations.

Limiting Reinforcement Parameters

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 Clear Spacing

The minimum clear spacing between parallel bars in a layer, $s_{cl,min}$, is given by;

$$s_{cl,min} \geq \text{MAX}[[d_b, 4/3*d_g, 1\text{in}, s_{cl,u,min}] \text{ (US units)}$$

$$s_{cl,min} \geq \text{MAX}[[d_b, 4/3*d_g, 25\text{mm}, s_{cl,u,min}] \text{ (metric units)}$$

where

d_g = the maximum size of aggregate

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

Minimum Area of Reinforcement

For ACI 318-08 and ACI 318-11¹

For structural slabs of uniform thickness the minimum area of tensile reinforcement in the direction of the span is:

For US-units:

IF Grade 40 to 50 deformed bars are used

$$A_{s,min,reqd} \geq b \cdot h \cdot 0.0020$$

IF Grade 50 to 60 deformed bars or welded wire reinforcement are used

$$A_{s,min,reqd} \geq b \cdot h \cdot 0.0018$$

For metric units:

IF Grade 280 to 350 deformed bars are used

$$A_{s,min,reqd} \geq b \cdot h \cdot 0.0020$$

IF Grade 350 to 420 deformed bars or welded wire reinforcement are used

$$A_{s,min,reqd} \geq b \cdot h \cdot 0.0018$$

IF yield stress exceeding 420 MPa

$$A_{s,min,reqd} \geq b \cdot h \cdot [\text{MAX}(0.0014, 0.0018 \cdot 420/f_y)]$$

¹ ACI 318-08:2008 and ACI 318-11:2011 and ACI 318M-11:2011 Section 7.12.2.1 and 10.5.4

Maximum Area of Reinforcement

The maximum area of longitudinal tension reinforcement is calculated in the same way as for beams - see: in **Beam Design to ACI 318**.

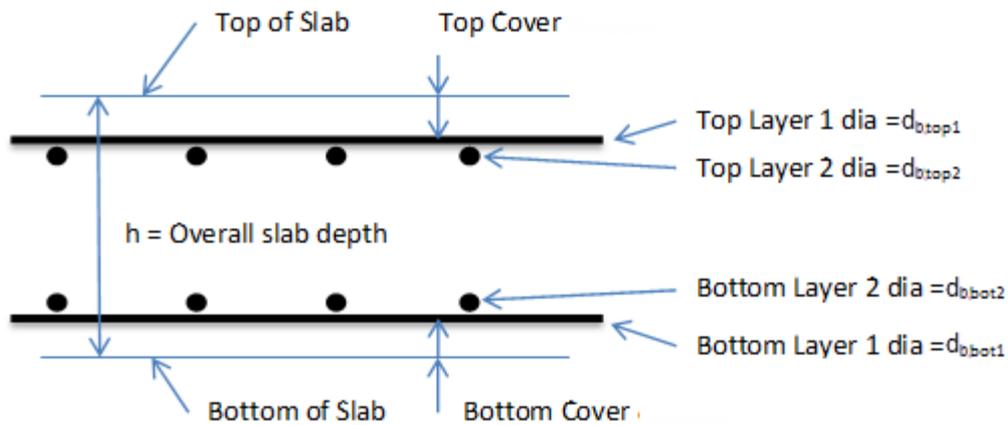
Basic Cross Section Design

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

Matching Design Moments to Reinforcement Layers



In any panel or patch potentially up to 4 sets of Design Bending Moments are established:

- M_{dx-top} - used to determine the reinforcement requirement of the x-direction bars in the top of the slab.
- M_{dy-top} - used to determine the reinforcement requirement of the y-direction bars in the top of the slab
- M_{dx-bot} - is used to determine the reinforcement requirement of the x-direction bars in the bottom of the slab.
- M_{dy-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

The basic design method for slabs is identical to that for beams - see: in **Beam Design to ACI 318**

Punching Shear Checks

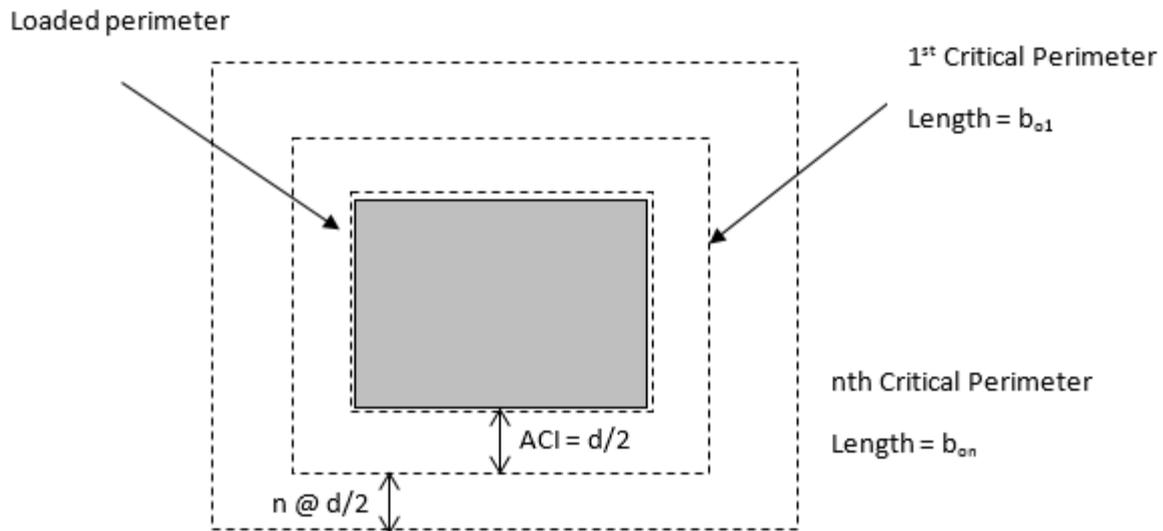
Limitations

ACI318 refers to two forms of slab shear strength

- Beam action - long or narrow slabs acting as a beam
- Two way action - punching along a truncated cone around a concentrated load or reaction area.

In *Tekla Structural Designer* we only consider Punching shear (two way action) and not Beam action.

Punching Shear Basics



There are a number of perimeters associated with Punching Shear (ref ACI 11.11.1.2)

- Loaded perimeter - perimeter around the loaded area - e.g. face of the wall or column
- 1st Critical perimeter - is the check punching shear perimeter $d/2$ from the loaded perimeter
- nth Critical perimeter - is the check punching shear perimeter $n \times d/2$ from the loaded perimeter
- b_o is the length of the 1st critical perimeter
- b_{on} is the length of the nth critical perimeter
- d = depth to tension steel

Length of the loaded perimeter u_0

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, i.e. where an internal angle is greater than 180 degrees, the length of a side and the slab/column interface is adjusted

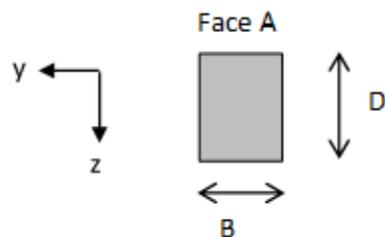
as indicated in the sketches below with the perimeter taken as the shortest distance around the column.



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)



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

$$\text{Bounding rectangle } D_{\text{bound}} = D$$

$$\text{Bounding rectangle } B_{\text{bound}} = B$$

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

$$a_n = \max(D, B)$$

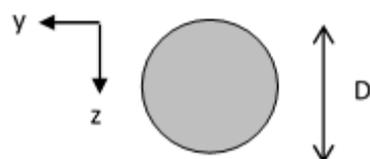
$$b_n = \min(D, B)$$

$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_0 = 2 \times (D_{\text{bound}} + B_{\text{bound}} + 2 \times d)$$

Note $d = d_{\text{drop}}$ if a drop is present

Circular (D)



$$\text{Loaded perimeter } u_0 = \pi \times D$$

$$\text{Bounding rectangle } D_{\text{bound}} = D$$

$$\text{Bounding rectangle } B_{\text{bound}} = D$$

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

$$a_n = \max(D, B)$$

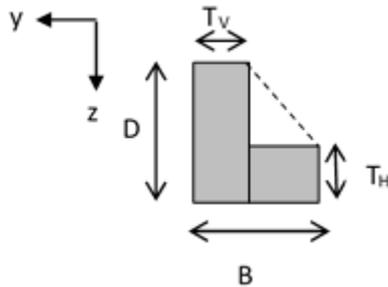
$$b_n = \min(D, B)$$

$$\beta = a_n / b_n$$

Critical perimeter $b_0 = 2 \times (D_{\text{bound}} + B_{\text{bound}} + 2 \times d)$

Note $d = d_{\text{drop}}$ if a drop is present

L section (D, B, T_H and T_V)



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

Bounding rectangle $D_{\text{bound}} = D$

Bounding rectangle $B_{\text{bound}} = B$

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

$a_n = \sqrt{(D^2 + B^2)}$

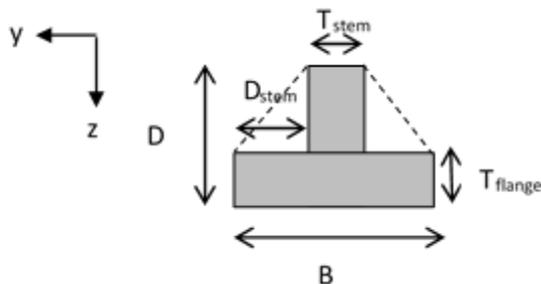
$b_n = D \times B / \sqrt{(D^2 + B^2)} + T_V \times T_H / \sqrt{(T_V^2 + T_H^2)}$

$\beta = a_n / b_n$

Critical perimeter $b_0 = (D + d) + (B + d) + (T_V + 3/4d) + (T_H + 3/4d) + \sqrt{((B+d) - (T_V + 3/4d))^2 + ((D + d) - (T_H + 3/4d))^2}$

Note $d = d_{\text{drop}}$ if a drop is present

T section (D, B, T_{stem}, T_{flange} and D_{stem})



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

Bounding rectangle $D_{\text{bound}} = D$

Bounding rectangle $B_{\text{bound}} = B$

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

$a_n = \max(D, B)$

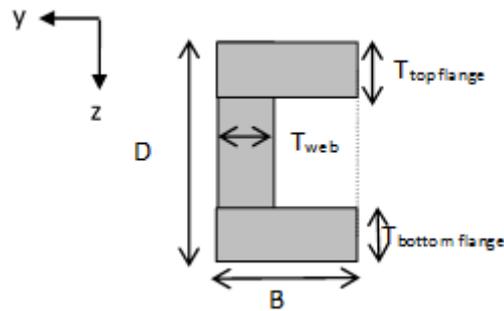
$b_n = \min(D, B)$

$\beta = a_n / b_n$

Critical perimeter $b_0 = (B + d) + 2x(T_{\text{flange}} + d) + (T_{\text{stem}} + d) + \sqrt{((D_{\text{stem}} + d)^2 + (D - (T_{\text{stem}})^2) + \sqrt{((B + d) - (T_{\text{stem}} + d) - (D_{\text{stem}} - d))^2 + (D - T_{\text{stem}})^2})}$ (approx)

Note $d = d_{\text{drop}}$ if a drop is present

C section (D, B, T_{web}, T_{top flange} and T_{bottom flange})



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

Bounding rectangle $D_{\text{bound}} = D$

Bounding rectangle $B_{\text{bound}} = B$

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

$$a_n = \max(D, B)$$

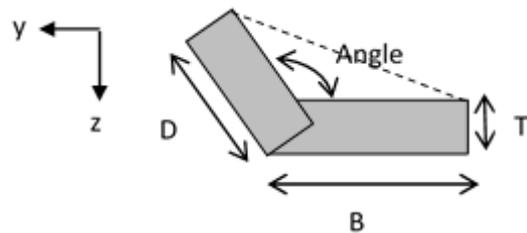
$$b_n = \min(D, B)$$

$$\beta = a_n / b_n$$

Critical perimeter $b_0 = 2 \times (B + D + 2 \times d)$

Note $d = d_{\text{drop}}$ if a drop is present

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

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

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

$$a_n = \max(B + D \text{ Cos}(180-\text{Angle}), D \text{ Sin}(180-\text{Angle}) + T \text{ Cos}(180-\text{Angle}))$$

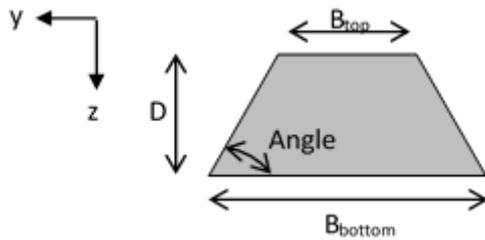
$$b_n = \min(B + D \text{ Cos}(180-\text{Angle}), D \text{ Sin}(180-\text{Angle}) + T \text{ Cos}(180-\text{Angle}))$$

$$\beta = a_n / b_n$$

Critical perimeter $b_0 = (B+d) + (D+d) + 2 \times (T+d) + \sqrt{(((B+d) + (D+d) \times \text{Sin}(\text{angle}-90)-(T+d) \times \text{Cos}(\text{Angle}-90))^2 + ((D+d) \times \text{Cos}(90-\text{Angle}) - (T+d))^2)}$ (approx)

Note $d = d_{\text{drop}}$ if a drop is present

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



$$u_0 = B_{top} + B_{bottom} + 2 \times D / \sin(\text{Angle})$$

$$\text{Bounding rectangle } D_{bound} = D$$

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

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

$$a_n = \max(D, B_{top}, B_{bottom})$$

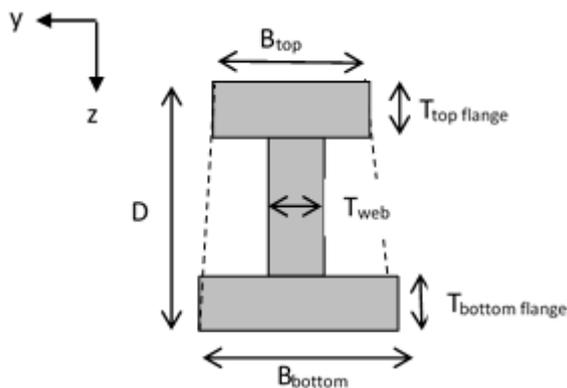
$$b_n = \min(D, B_{top}, B_{bottom})$$

$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_o = (B_{top} + d) + (B_{bottom} + d) + 2 \times (D + d) / \sin(\text{Angle}) \text{ (approx)}$$

Note $d = d_{drop}$ if a drop is present

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



$$u_0 = B_{bottom} + B_{top} + 2 \times \sqrt{((B_{bottom} - B_{top})/2)^2 + D^2} \text{ (approx)}$$

$$\text{Bounding rectangle } D_{bound} = D$$

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

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

$$a_n = \max(D, B)$$

$$b_n = \min(D, B)$$

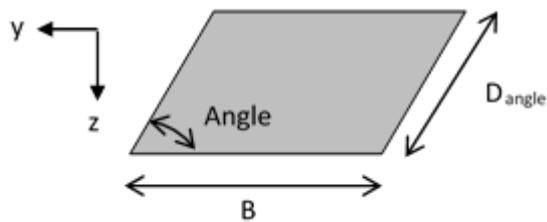
$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_o = (B_{bottom} + d) + (B_{top} + d) + 2 \times \sqrt{((B_{bottom} - B_{top})/2)^2 + (D + d)^2} \text{ (approx)}$$

Note $d = d_{drop}$ if a drop is present

Parallelogram (D_{angle} , B , angle)

Reference Guides (ACI AISC)



$$u_0 = 2 \times (B + D_{\text{angle}})$$

$$\text{Bounding rectangle } D_{\text{bound}} = D \times \sin(\text{Angle})$$

$$\text{Bounding rectangle } B_{\text{bound}} = B + D_{\text{angle}} \times \cos(\text{Angle})$$

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

$$a_n = \max(B, D \times \sin(\text{Angle}))$$

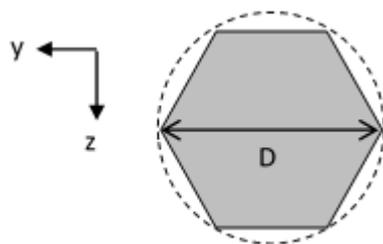
$$b_n = \min(B, D \times \sin(\text{Angle}))$$

$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_0 = 2 \times ((B + d) + (D_{\text{Angle}} + d)) \text{ (approx)}$$

Note $d = d_{\text{drop}}$ if a drop is present

Polygon (D, n) - $n > 4$



$$u_0 = 2 \times n \times D/2 \times \sin(180/n)$$

$$\text{Bounding circle } D_{\text{bound}} = n \times D/\pi \times \sin(180/n) \text{ (equivalent perimeter)}$$

$$\text{Bounding circle perimeter } u_{0\text{bound}} = \pi \times D_{\text{bound}}$$

$$a_n = \max(D, B)$$

$$b_n = \min(D, B)$$

$$\beta = a_n / b_n$$

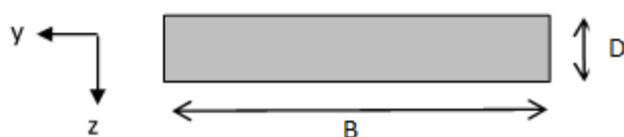
$$\text{Critical perimeter } b_0 = 2 \times (D_{\text{bound}} + B_{\text{bound}} + 2 \times d)$$

Note $d = d_{\text{drop}}$ if a drop is present

Loaded perimeter for Walls

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

Rectangular (D and B)



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

$$\text{Bounding rectangle } D_{\text{bound}} = D$$

$$\text{Bounding rectangle } B_{\text{bound}} = B$$

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

$$a_n = \max(D, B)$$

$$b_n = \min(D, B)$$

$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_0 = 2 \times (D + B + 2 \times d)$$

Note d is for the slab

Loaded perimeter for Point Loads

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

$$u_0 = 2 \times (D_{\text{load}} + B_{\text{load}})$$

$$\text{Bounding rectangle } D_{\text{bound}} = D_{\text{load}}$$

$$\text{Bounding rectangle } B_{\text{bound}} = B_{\text{load}}$$

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

$$a_n = \max(D_{\text{load}}, B_{\text{load}})$$

$$b_n = \min(D_{\text{load}}, B_{\text{load}})$$

$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_0 = 2 \times (D_{\text{load}} + B_{\text{load}} + 2 \times d)$$

Note d is for the slab

Additional Loaded perimeter drops

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

$$\text{Loaded perimeter } u_{0\text{drop}} = 2 \times B_{\text{drop}} \times D_{\text{drop}}$$

$$a_n = \max(D_{\text{drop}}, B_{\text{drop}})$$

$$b_n = \min(D_{\text{drop}}, B_{\text{drop}})$$

$$\beta = a_n / b_n$$

$$\text{Critical perimeter } b_0 = 2 \times (D_{\text{drop}} + B_{\text{drop}} + 2 \times d)$$

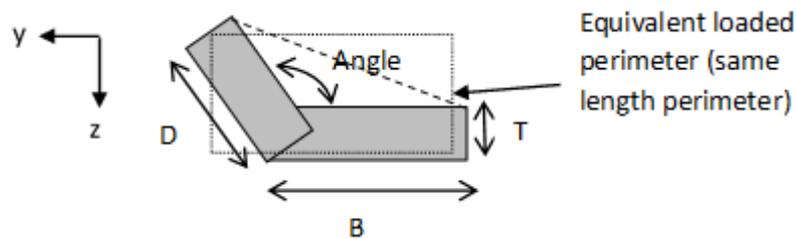
Note d is for the slab around the drop

The equivalent loaded perimeter

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

- $D_{\text{equiv}} = D_{\text{bound}} \times u_0 / u_{0\text{bound}}$

- $B_{equiv} = B_{bound} \times u_0 / u_{0bound}$



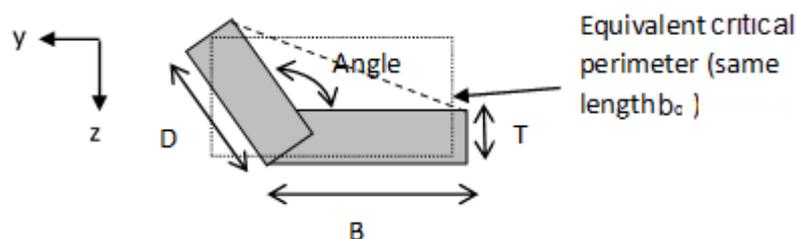
The equivalent perimeter is used in three situations

- adjustment of the loaded perimeter length/shape u_0 for edge and corner columns/walls
- the rectangle from which the 1st to nth critical perimeters are determined
- Reduction in V_{Ed}

The equivalent critical perimeter

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

- $d_1 = (b_o / 2 - B_{equiv} - D_{equiv}) / 2$
- $D_{equiv} = D_{bound} \times u_0 / u_{0bound}$
- $B_{equiv} = B_{bound} \times u_0 / u_{0bound}$



Columns and Walls not 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.

Length of the critical perimeter b_o

Critical perimeter without drops

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

- For all internal column/wall shapes and point loads, b_o is as given in the section:
- For all corner column/wall shapes and point loads

$$b_o = A + B + C$$

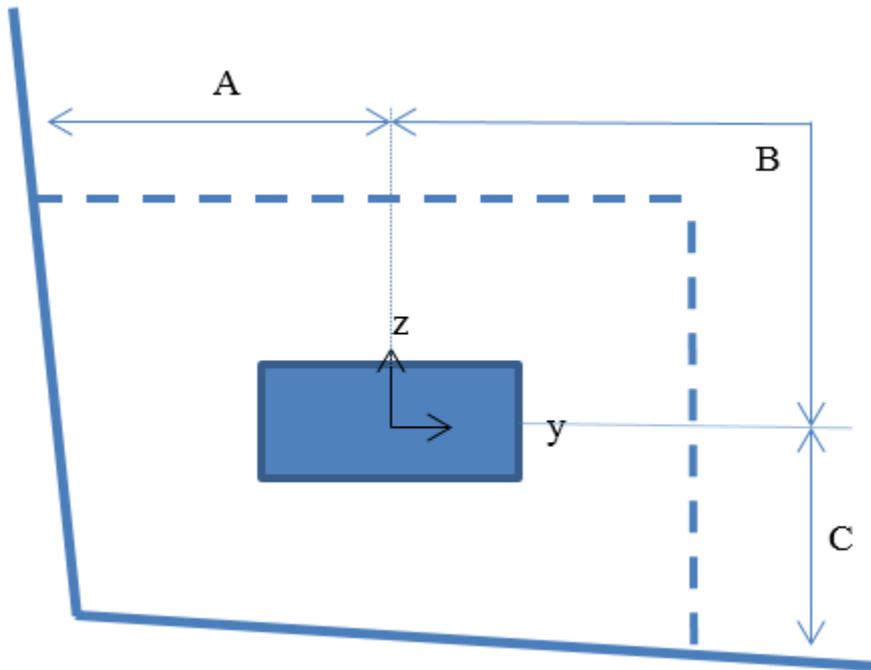
where

for a rectangle

$A = \text{dist from centroid to edge along local } y$

$B = c_1/2 + d + c_2/2$

$C = \text{dist from centroid to edge along local } z$



- For all edge column/wall shapes and point loads
 $b_o = A + B + C$

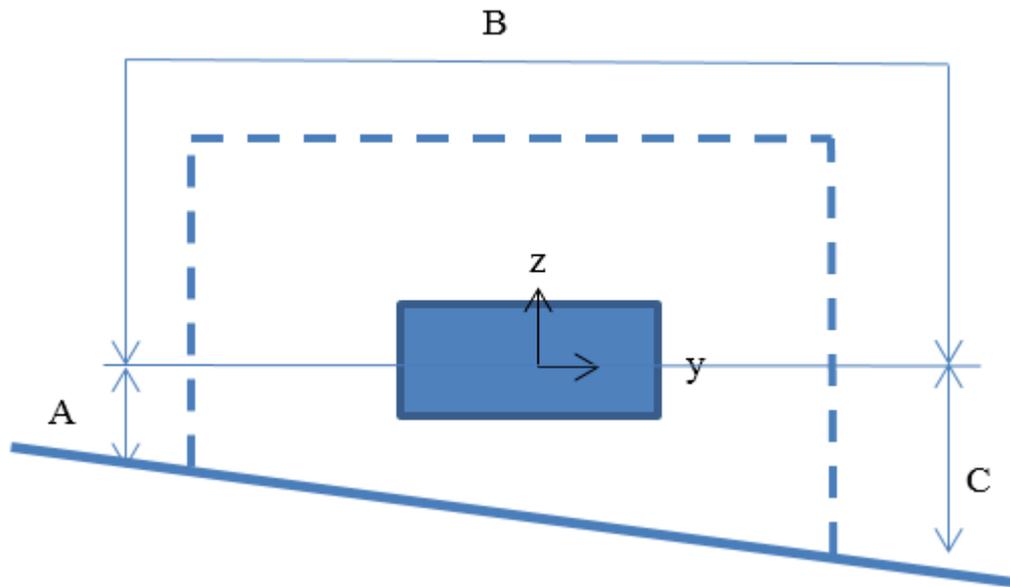
where

For a rectangle

$A = \text{dist from centroid to edge along local } y \text{ or local } z$

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

$C = \text{dist from centroid to edge along local } y \text{ or local } z$



If a critical perimeter passes across a slab edge then only the perimeter length in the slab is counted in b_o .

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

Overlapping Control Perimeters

The calculations are beyond scope in the following situations:

- If two 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.

Modification of control perimeters to take account of slab openings

If any openings have been defined in the slab and if the nearest opening edge is not greater than $10 \times h$ (h = slab thickness) from the face of the column then the length of the loaded perimeter at the column face, b_o and out from there to b_{on} need to be reduced to take account of the presence of the opening(s) as indicated in fig 6.14.



When a perimeter length has been reduced to cater for openings - as the exact position of the opening in relation to the reinforcement strips is not known, the calculations conservatively ignore any patch reinforcement in the punching checks - only the slab reinforcement is used.

User Modification of control perimeters

If you want to consider the effect of openings, but do not want to place them in the slab, this can be done by defining the following properties:

- u_0 - user reduction

- u_1 - user reduction

When applied, the length of the respective shear perimeters are reduced by the specified amount.

References

1. **American Concrete Institute.** Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. **ACI 2008.**
2. **American Concrete Institute.** Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary. **ACI 2011.**
3. **American Concrete Institute.** Metric Building Code Requirements for Structural Concrete (ACI 318M-11) and Commentary. **ACI 2014.**

Concrete Design - BS 8110

Beam Design to BS 8110

Design for Bending

Design for Bending for Flanged Sections

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

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

where

$h_{f,\text{side}i}$ = the depth of the slab on side 'i' of the beam

Calculate the value of K from Δ ;

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

Calculate the lever arm, z from;

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

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 $a \leq h_f$, 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 $b_w = b_{\text{eff}}$.

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.

Reference Guides (ACI AISC)

Calculate the value of β_f as below;

$$\beta_f = \{0.45 \cdot (h_f/d) \cdot (1 - (b_w/b_{eff})) \cdot (1 - (h_f/2 \cdot d))\} + \{0.15 \cdot (b_w/b_{eff})\}^B$$

Calculate the moment of resistance of the section as below;

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

Calculate the value of K

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

Calculate the limiting value of K, known as K'

$$K' = 0.156$$

IF

$$M \leq M_R, h_f < 0.45 \cdot d \text{ and } K \leq K'$$

THEN

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

ELSE

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

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

Where

$$\gamma_m = \text{partial safety factor for concrete in flexure} = 1.5^D$$

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;

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

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

$$K' = (0.156)$$

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

Calculate the lever arm, z from;

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

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

$$f_s = \begin{cases} \text{compression stress in steel} \\ 0.87 * f_y & \text{if } d'/d \leq 0.5 * \{1 - (f_y/800)\} \\ E_s * \epsilon_c * \{1 - (2d'/d)\} & \text{if } d'/d > 0.5 * \{1 - (f_y/800)\} \end{cases}$$

Calculate the lever arm, z from;

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

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

Steel Design to AISC 360 ASD and LRFD

Tekla Structural Designer designs steel and composite members to a range of international codes. This reference guide specifically describes the design methods applied when the AISC 360 ASD or AISC 360 LRFD resistance codes are selected.

General

Seismic Design

All 'Gravity Only Design' members are designed as per the normal AISC Specification rules for the seismic load combinations.

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05) ([Ref. 8](#)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems. These rules are applied as follows:

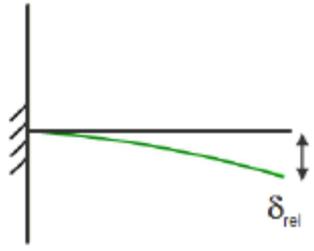
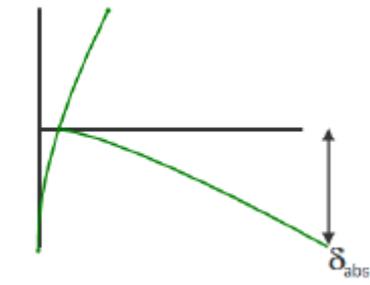
- If SDC = A - no additional requirements
- If SDC = D, E or F, apply rules for AISC 341

For each of X and Y directions:

- If SDC = B or C and $R \leq 3$ - no additional requirements
- If SDC = B or C and $R > 3$, apply rules for AISC 341

Deflection checks

Tekla Structural Designer calculates both **relative** and **absolute** deflections. Relative deflections measure the internal displacement occurring within the length of the member and take no account of the support settlements or rotations, whereas absolute deflections are concerned with deflection of the structure as a whole. The absolute deflections are the ones displayed in the structure deflection graphics. The difference between **relative** and **absolute** deflections is illustrated in the cantilever beam example below.

	
Relative Deflection	Absolute Deflection

Relative deflections are given in the member analysis results graphics and are the ones used in the member design.

Steel Grade

The steel grade can be chosen from the standard range for the USA or from an international range. User defined grades can also be added.



The upper limit for the steel grade is defined in the AISC Specification as 75 ksi - you should not attempt to add a grade higher than this. See 11.2 (360-05) or 11.3 (360-10).

The elastic modulus of steel for use in design is defined in the AISC Specification as $E = 29,000$ ksi

Steel Beam Design to AISC 360

Design method

Either a load and resistance factor design (LRFD) or an allowable strength design (ASD) can be performed to determine the adequacy of the section for each condition.

The design method employed is consistent with the design parameters specified in the relevant chapters of the AISC Specification and associated 'Commentary', unless specifically noted otherwise. As both the 2005 ([Ref. 1](#)) and 2010 ([Ref. 2](#)) versions are supported, where clauses are specific to a particular version these are indicated as (360-05) or (360-10) as appropriate.

A basic knowledge of the design methods for beams in accordance with the specification is assumed.

Section Classification

Cross-section classification is determined using Table B4.1 (360-05), or Tables B4.1a+B4.1b (360-10).

At every cross section there are two classifications for each element in the section (flange or web) - one for axial compression and one for bending (flexure).

If axial compression does not exist (0kip or tension), the axial classification is not applicable. If bending is not present about both axes then the flexure classification is not applicable.

For axial compression the web and flanges are classified as either Compact or Slender and the worst of the two is the resultant axial classification.

For bending both the web and flange are classified as Compact, Non compact or Slender and the worst of the two is the resultant flexural classification.

The classification of the section must normally be Compact or Non compact, however sections which are classified as Slender will be allowed if they are subject to axial load only.

D2. Axial Tension

If axial tension exists, tensile yielding and rupture checks are performed at the point of maximum tension in accordance with Eqns D2.1 and D2.2.



In the rupture check the net area A_e is assumed to equal the gross area A_g .

A warning is issued if the slenderness ratio L/r exceeds 300.

E. Axial Compression

If axial compression exists, the member is assessed for Flexural Buckling and for Torsional and Flexural Torsional buckling. The compressive strength is determined in accordance with Eqns E3.1 and E4.1. For double angles these equations are subject to the modifications of Section E6.

The member length or member sub lengths between braces are checked for:

- Flexural buckling about major axis - for each unbraced length between adjacent points of major axis lateral brace and or torsional brace.
- Flexural buckling about minor axis - for each unbraced length between adjacent points of minor axis lateral brace and or torsional brace.

- Torsional and flexural torsional buckling - for each unbraced length between adjacent points of torsional brace (this check is not applied to hollow sections.)

For any unbraced length, the required compressive force P_r is taken as the maximum compressive force in the relevant length.

A warning is issued if the slenderness ratio KL/r exceeds 200.

G2. Shear Strength

Shear checks are performed at the point of maximum shear in accordance with Section G2.

F2. Flexure

The member is assessed for Flexure in accordance with Section F2.

The following checks are potentially required:

About the x axis - within the LTB braced length

- Yielding
- Compression flange local buckling
- Web local buckling
- Local buckling
- Lateral Torsional Buckling (only required for I and C sections)

About the y axis in the LTB braced length

- Yielding
- Compression flange local buckling
- Web local buckling
- Local buckling

You can switch off the lateral torsional buckling checks for any unbraced length by indicating the length is continuously braced. If you use this option you must be able to provide justification that the unbraced length is adequately braced against lateral torsional buckling.

When the checks are required *Tekla Structural Designer* assumes a top flange (but not bottom flange) brace is provided at the position of each incoming beam. You can add or remove these braces if they don't reflect the actual brace provided by the incoming section. Each unbraced length which is not defined as being continuously braced is then checked in accordance with Section F2.

H1. Combined Forces

Members subject to axial tension or compression and flexure about one or both axes are assessed in accordance with Section H1.

Seismic Design Rules

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05)([Ref. 8](#)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems.

See “Assumptions/Limitations of the Seismic Provisions” for a list of the assumptions and limitations that apply with respect to the application of these rules to *Tekla Structural Designer* models.

The rules applied depend upon the seismic load resisting system as defined in the AISC Seismic Provisions and are listed below:

For a moment resisting frame

9. Special Moment Frame (SMF)

- 9.4a. Classification
- 9.8. Max spacing of bracing

10. Intermediate Moment Frame (IMF)

- 10.4a. Classification
- 10.8. Max spacing of bracing

11. Ordinary Moment Frame (OMF)

- 11.4. Classification

Moment resisting frame with a truss component

12. Special Truss Moment Frame (STMF) ¹- Beyond Scope

For a braced frame

13. Special Concentrically Braced Frames (SCBF)

- 13.2d. Classification
- 13.4a.(2). Max lat brace spacing
- 13.4a. V and inverted V type

14. Ordinary Concentrically Braced Frames (OCBF)

- 14.2. Classification
- 14.3. Beams with V and inverted V type
- Beams (not columns with no K braces)
 - 14.2.(2). Max lat brace spacing

15. Eccentrically Braced Frames (EBF) - Beyond Scope

Buckling resistant braced frame

16. Buckling Restrained Braced Frames (BRBF) - Beyond Scope

Frames containing composite beams

Composite Special Concentrically Braced Frames (C-SCBF) - Beyond Scope

Composite Ordinary Braced Frames (C-OBF) - Beyond Scope

Composite Eccentrically Braced Frames (C-EBF) - Beyond Scope

[1.](#) Beyond Scope of the current version of Tekla Structural Designer

Composite Beam Design to AISC 360

Design method

Either a load and resistance factor design (LRFD) or an allowable strength design (ASD) can be performed to determine the adequacy of the section for each condition.

The design method employed is consistent with the design parameters for simple composite beams as specified in Chapter I of the AISC Specification and associated 'Commentary', unless specifically noted otherwise. As both the 2005 ([Ref. 1](#)) and 2010 ([Ref. 2](#)) versions are supported, where clauses are specific to a particular version these are indicated as (360-05) or (360-10) as appropriate.

A basic knowledge of the design methods for composite beams in accordance with the specification is assumed.

Construction stage

At construction stage the beam is acting alone before composite action is achieved and is unshored.

When you design or check a beam for construction stage loading the following checks are carried out in accordance with the relevant chapters of the AISC Specification, consistent with the approach (i.e. LRFD or ASD) used at the composite stage.

Section classification

Cross-section classification is determined using Table B4.1 (360-05), or Tables B4.1a+B4.1b (360-10) and must be Compact or Non compact. Sections which are classified as Slender are beyond the scope of *Tekla Structural Designer*.

Shear strength - I3.1b (360-05), I4.2 (360-10)

Shear checks are performed at the point of maximum shear based upon the properties of the steel section alone in accordance with Section G2.

Strength during construction - I3.1c (360-05), I3.1b (360-10)**Flexure**

Checks are performed at the point of maximum moment along the beam based upon the properties of the steel section alone in accordance with Section F2.

Lateral torsional buckling checks

When the forms are attached to the top flange then full lateral restraint can be assumed, irrespective of the angle of the deck. In this case you should indicate the beam is continuously braced.

In other cases any incoming beams will be automatically identified.

Each sub-length which is not defined as being continuously braced is checked in accordance with Section F2.

Deflection checks

Relative deflections are used in the composite beam design. (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 live 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, (as per CC.1.1 of ASCE 7-05([Ref. 6](#)) or ASCE 7-10([Ref. 7](#))).

Composite stage

Tekla Structural Designer performs all checks for the composite stage condition in accordance with Section I3 unless specifically noted otherwise.

Equivalent steel section

An equivalent steel section is determined for use in the composite stage calculations by removing the fillet while maintaining the full area of the section. This approach reduces

the number of change points in the calculations while maintaining optimum section properties.

Shear strength - I3.1b (360-05), I4.2 (360-10)

Shear checks are performed at the point of maximum shear in accordance with Section G2 for the maximum required shear strength, V_r , at the composite stage. The shear check is performed on the bare beam alone at the composite stage ignoring any contribution from the concrete slab.

Strength of composite beams with shear connectors - I3.2

Section classification

For section classification purposes the true section is used. *Tekla Structural Designer* classifies the section in accordance with Section I3.2a. Only the web of the section is classified - the bottom flange is in tension and so cannot buckle locally and it is assumed that the top flange is sufficiently braced by the composite slab.

The classification of the web must be compact so that plastic stress blocks can be used.

Flexure

Checks are performed at the point of maximum moment and the position of application of each point load as well as all other points of interest along the beam. Flexure is calculated in accordance with Section I3.2 (360-05/-10). Since the flexural strength at all point loads is checked then this will inherently satisfy Section I3.2d (6) (360-05) or Section I8.2c (360-10) which require that "the number of shear connectors placed between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum required flexural strength at the concentrated load point".

During the selection process, in auto design mode point loads are taken to be 'significant' if they provide more than 10% of the total shear on the beam. For the final configuration and for check mode all point loads are checked for flexure.

Shear connectors

Tekla Structural Designer checks shear connectors to Section I1-3 (360-05), or Section I8 (360-10).

The nominal strength of headed stud shear connectors in a solid slab or a composite slab is determined in accordance with Section I3.2d (360-05), or Section I3.2d with shear connector strength from I8.2a (360-10).

Ribs perpendicular

The reduction factor R_p is taken as,

$$R_p = 0.6 \quad \text{for any number of studs and } e_{\text{mid-ht}} < 2 \text{ in}$$

$$= 0.75 \quad \text{for any number of studs and } e_{\text{mid-ht}} \geq 2 \text{ in}$$

In *Tekla Structural Designer* you are therefore not required to input the actual value of $e_{\text{mid-ht}}$ instead you simply indicate if it is less than 2 in.

Ribs parallel

$$R_p = 0.75 \quad \text{in all cases}$$

Ribs at other angles

Where the ribs are at an angle θ_r to the beam there is no guidance in the AISC Specification. The approach adopted by *Tekla Structural Designer* is to apply a geometric adjustment of the reduction factors R_g and R_p which for the purposes of this adjustment are combined into one 'k' factor. The combined reduction factor is calculated for perpendicular and parallel separately and then adjusted as shown below.

$$k_s = k_1 * \sin^2\theta_r + k_2 * \cos^2\theta_r$$

Where:

$$k_s = \text{the adjusted value of the combined reduction factor } R_g * R_p$$

$$k_1 = \text{the value of the combined reduction factor } R_g * R_p \text{ for ribs perpendicular}$$

$$k_2 = \text{the value of the combined reduction factor } R_g * R_p \text{ for ribs parallel}$$

Degree of shear connection

For efficient design the number of studs should be minimized. If the number provided has an overall capacity greater than the capacity of the concrete flange or steel beam (whichever is the lesser) then this is full shear connection. Anything less than this, is partial shear connection. There are, however, limits on the amount of partial interaction that are recommended by the AISC Specification – see note '3' (p.16.1-311 of the 2005 Commentary, or p.16.1-356 of the 2010 Commentary).

For all beams, the number of connectors required for full shear connection is,

$$N_s = (\min(T_s, (C_{c1} + C_{c2}))) / Q_n \text{ rounded up to the next group size above}$$

Where:

$$T_s = \text{the tensile yield strength of the steel section}$$

$$C_{c1} = \text{the strength of the concrete flange above the ribs}$$

$$C_{c2} = \text{strength of the concrete in the ribs (zero for perpendicular decks)}$$

$$Q_n = \text{the nominal strength of an individual shear connector}$$

The degree of partial shear connection is given by,

$$I_{nt} = N_a * Q_n / (\min((C_{c1} + C_{c2}), T_s))$$

Where:

N_a = the number of shear connectors provided from the nearer point of support to the position under consideration

The degree of partial shear connection is checked at the point of maximum bending moment or the position of a point load if at that position the maximum utilization ratio occurs.

To determine the status of the check *Tekla Structural Designer* applies the following rules:

- If the partial interaction ratio at the position of maximum moment is less than the absolute minimum interaction ratio (default 25%), then this generates a FAIL status,
- If the partial interaction ratio at the position of maximum utilization ratio when this is at a different position to the maximum moment, is less than the absolute minimum interaction ratio, then this generates a WARNING status,
- If the partial interaction ratio at the position of maximum moment, or maximum utilization ratio if this is different, is greater than the absolute minimum interaction ratio, then this generates a PASS status,
- If the partial interaction ratio at any point load position that is not the maximum utilization ratio is less than the absolute minimum interaction ratio, then this does not affect the status in any way.
- If the partial interaction ratio at any position is less than the advisory minimum interaction ratio (default 50%) then this is given for information only and does not affect the status in any way.

Dimensional requirements

The dimensional limits given below are either recommendations or code limits:

- the nominal rib height of the profiled deck, h_r should be not greater than 3 in
- the mean width of the ribs of the profiled sheet, w_r should be not less than 2 in (for re-entrant decks the 'mean' is taken as the minimum opening at the top of the rib)
- the nominal diameter of stud connectors, d_{sc} should be not greater than $\frac{3}{4}$ in
- the height of the stud after welding, H_s should be at least $1\frac{1}{2}$ in greater than the nominal rib height of the profiled deck – see Section I3.2c(b) (360-05), or Section I3.2c(2) (360-10).
- the total depth of the composite slab, d_{cs} should not be less than $3\frac{3}{4}$ in
- the thickness of concrete above the main flat surface of the top of the ribs of the sheeting, $d_{cs} - h_r$ should not be less than 2 in
- concrete cover, $d_{cs} - H_s$ over the connector should not be less than $\frac{1}{2}$ in – see Section I3.2c(b) (360-05), or Section I3.2c(2) (360-10).
- the longitudinal spacing should not exceed the lesser of 36 in or $8 * \text{the slab depth, } d_{cs}$ (see Section 6.2.6.2 of Structural Steel Designer's Handbook. Second Edition([Ref. 4](#)))

- where studs are spaced at greater than 18 in centers puddle welds or other appropriate means are required to ensure anchorage of deck – see Section I3.2c (360-05), or Section I3.2c(4) (360-10).
- the clear distance between the edge of a connector and the edge of the steel beam flange should be not less than $\frac{3}{4}$ in (as universal good practice).
- Section I8.2d of the AISC Specification (360-10) requires that the minimum edge distance from the center of an anchor to a free edge in the direction of the shear force shall be 8 in for normalweight concrete and 10 in for lightweight concrete. This requirement will apply only in a limited number of configurations and therefore is not checked in Fastrak.
- the spacing of connectors in the direction of shear i.e. along the beam should be not less than, $6 * \text{the stud diameter}$
- the spacing of connectors transverse to direction of shear i.e. across the beam should be not less than $4 * \text{the stud diameter}$ except for the condition given in the next item
- where rows of studs are staggered, the minimum transverse spacing between longitudinal lines of studs should be not less than $3 * \text{the stud diameter}$ with the amount of stagger such that the diagonal distance between studs on adjacent longitudinal lines is not less than $4 * \text{the stud diameter}$
- the stud connector diameter should not exceed 2.5 times the flange thickness unless located directly over the web.



You should confirm that the chosen configuration of decking and studs meet those dimensional requirements that you deem appropriate.

Serviceability Limit State (SLS)

Section properties (SLS)

In the calculation of the gross moment of inertia of the composite section the steel deck is ignored as is any concrete in tension. The concrete is converted into an equivalent steel section using an effective modular ratio based on the proportions of long and short term loads which are relevant to the particular calculation. Two alternative approaches are given - see p.16.1-308 in the 2005 Commentary, or p.16.1-353 in the 2010 Commentary for obtaining these properties.

One (the 'traditional method') calculates the gross uncracked inertia of the transformed section but uses 75% of the resulting value in the determination of deflections. The other uses a given formula to determine a 'lower-bound' inertia. While studies have shown that the simple application of a reduction factor (0.75) is more onerous than the lower-bound solution, the simpler 'traditional method' is the approach adopted in *Tekla Structural Designer*.

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 Dry	bare beam
Dead	composite properties calculated using the modular ratio for long term loads*
Live, Roof Live	composite properties calculated using the effective modular ratio** appropriate to the long term load percentage for each load.
Wind, Snow, Earthquake	composite properties calculated using the modular ratio for short term loads
Total loads	these are calculated from the individual loadcase loads as detailed above.

*The long term modulus is taken as the short term value divided by a factor (for shrinkage and creep), entered in the Slab properties.

$$n_s = \text{the short term modular ratio} \\ = E_s/E_c$$

$$n_L = \text{the long term modular ratio} \\ = (E_s/E_c) * k_n$$

**The effective modular ratio, n_E is based on the percentage of load which is considered long term. These calculations are repeated for each individual load in a loadcase.

The effective modular ratio is given by,

$$n_E = n_s + \rho_L * (n_L - n_s)$$

$$\rho_L = \text{the proportion of the load which is long term}$$

The calculated Slab Dry, Live and Total load deflections (where necessary adjusted for the effect of partial interaction) are checked against the limits you specify.



All the beam deflections calculated above are 'relative' deflections. For an illustration of the difference between relative and absolute deflection see .

Stress checks (SLS)

The Commentary (Section I3.1, paragraph 2 of the 2005 version, Section I3.2, of the 2010 version) suggests that where deflection controls the size of the beam then either it should be ensured that the beam is elastic at serviceability loading or that the inelastic deformations are taken into account. *Tekla Structural Designer* adopts the former approach. This is confirmed by checking that yield in the beam and crushing in the concrete do not occur at serviceability loading i.e. a service stress check. If they are found to fail, suggesting inelasticity at serviceability loading, then a warning will appear on the deflections page and the service stress results are available to view.

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. The partial safety factors for loads are taken as those provided by you for the service condition on the Design Combinations page. The stress checks assume that full interaction exists between the steel and the concrete at serviceability state.

Natural frequency checks (SLS)

The calculation of the natural frequency of a composite beam can be complex and is dependent upon the support conditions, the load profile and the properties of the composite section. In reality the vibration of a composite beam is never in isolation – the whole floor system (including the slabs and other adjacent beams) will vibrate in various modes and at various frequencies.

A simple (design model) approach is taken based on uniform loading and pin supports. This fairly simple calculation is provided to the designer for information only. The calculation can be too coarse particularly for long span beams and does not consider the response side of the behavior i.e. the reaction of the building occupants to any particular limiting value for the floor system under consideration. In such cases the designer will have the option to perform a Floor Vibration Analysis within the *Tekla Structural Designer* application.

Simplified approach

The natural frequency is determined from,

$$NF = 0.18 * \sqrt{(g/\Delta_{NF})}$$

Where:

Δ_{NF} = the maximum static instantaneous deflection (in inches) that would occur under the effects of Slab Dry loading, and the proportion of dead loads and live loads specified by the user (as specified on the Natural Frequency page of the Design Wizard). It is based upon the composite inertia but not modified for the effects of partial interaction.

g = the acceleration due to gravity (386.4 in/s²)

This is not given in the AISC Specification but is taken from Chapter 3 of Steel Design Guide Series 11. Floor Vibrations due to Human Activity.[\(Ref. 5\)](#) It's formulation is derived from the first mode of vibration of a simply supported beam subject to a udl.

Steel Column Design to AISC 360

Design Method

Either a load and resistance factor design (LRFD) or an allowable strength design (ASD) can be performed to determine the adequacy of the section for each condition.

The design method employed is consistent with the design parameters specified in the relevant chapters of the AISC Specification and associated 'Commentary', unless specifically noted otherwise. As both the 2005 [\(Ref. 1\)](#) and 2010 [\(Ref. 2\)](#) versions are supported, where clauses are specific to a particular version these are indicated as (360-05) or (360-10) as appropriate.

A basic knowledge of the design methods for columns in accordance with the specification is assumed.

Section classification

Cross-section classification is determined using Table B4.1 (360-05), or Tables B4.1a+B4.1b (360-10).

At every cross section there are two classifications for each element in the section (flange or web) - one for axial compression and one for bending (flexure).

If axial compression does not exist (0kip or tension), the axial classification is NA. If bending is not present about both axes then the flexure classification is NA.

For axial compression the web and flanges are classified as either Compact or Slender and the worst of the two is the resultant axial classification.

For bending both the web and flange are classified as Compact, Non compact or Slender and the worst of the two is the resultant flexural classification.

The classification of the section must normally be Compact or Non compact, however sections which are classified as Slender will be allowed if they are subject to axial load only.

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

D2. Axial Tension

If axial tension exists, tensile yielding and rupture checks are performed at the point of maximum tension in accordance with Eqns D2.1 and D2.2.



In the rupture check the net area A_e is assumed to equal the gross area A_g .

A warning is also issued if the slenderness ratio L/r exceeds 300.

E. Axial Compression

If axial compression exists, the member is assessed for Flexural Buckling and for Torsional and Flexural Torsional buckling. The compressive strength is determined in accordance with Eqns E3.1 and E4.1. For double angles these equations are subject to the modifications of Section E6.

The member length or member sub lengths between braces are checked for:

- Flexural buckling about major axis - for each sub-length between adjacent points of major axis lateral bracing and or torsional bracing.
- Flexural buckling about minor axis - for each sub-length between adjacent points of minor axis lateral bracing and or torsional bracing.
- Torsional and flexural torsional buckling - for each sub-length between adjacent points of torsional bracing (this check is not applied to hollow sections.)

For any sub-length, the required compressive force P_r is taken as the maximum compressive force in the relevant sub-length.

A warning is also issued if the slenderness ratio KL/r exceeds 200.

G2. Shear Strength

Shear checks are performed 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 in accordance with Section G2.

F2. Flexure

The member is assessed for Flexure in accordance with Section F2. The following checks are potentially required:

About the x axis - within the LTB sub-length

- Yielding
- Compression flange local buckling
- Web local buckling
- Local buckling
- Lateral Torsional Buckling (only required for I and C sections)

About the y axis in the LTB sub-length

- Yielding
- Compression flange local buckling
- Web local buckling
- Local buckling

The lateral torsional buckling checks can be switched off for any sub-length by indicating the length is continuously braced. If you use this option you must be able to provide justification that the sub-length is adequately braced against lateral torsional buckling.

When the checks are required you can set the effective length of each sub-beam (the portion of the beam between one brace 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.

H1. Combined Forces

Members subject to axial tension or compression and flexure about one or both axes are assessed in accordance with Section H1.

Seismic Design Rules

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05) ([Ref. 8](#)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems.

See "Assumptions/Limitations of the Seismic Provisions" for a list of the assumptions and limitations that apply with respect to the application of these rules to *Tekla Structural Designer* models.

The rules applied depend upon the seismic load resisting system as defined in the AISC Seismic Provisions and are listed below:

For a moment resisting frame

9. Special Moment Frame (SMF)

- 9.4a. Classification
- 9.4. Column strength check
- 9.6. Column/beam moment ratio

10. Intermediate Moment Frame (IMF)

- 10.4a. Classification
- 10.4. Column strength

11. Ordinary Moment Frame (OMF)

- 11.4. Classification
- 11.4. Column strength check

Moment resisting frame with a truss component

12. Special Truss Moment Frame (STMF) ¹- Beyond Scope

For a braced frame

13. Special Concentrically Braced Frames (SCBF)

- 13.2d. Classification
- 13.2b. Column strength check

14. Ordinary Concentrically Braced Frames (OCBF)

- 14.2. Classification
- 14.2. Column strength check

15. Eccentrically Braced Frames (EBF) - Beyond Scope

Buckling resistant braced frame

16. Buckling Restrained Braced Frames (BRBF) - Beyond Scope

Frames containing composite beams

Composite Special Concentrically Braced Frames (C-SCBF) - Beyond Scope

Composite Ordinary Braced Frames (C-OBF) - Beyond Scope

Composite Eccentrically Braced Frames (C-EBF) - Beyond Scope

[1.](#) Beyond Scope of the current version of Tekla Structural Designer

Steel Brace Design to AISC 360

Design Method

Tekla Structural Designer allows you to analyze and design a member with pinned end connections for axial compression, tension and seismic design forces.

Either a load and resistance factor design (LRFD) or an allowable strength design (ASD) can be performed to determine the adequacy of the section for each condition.

The design method employed is consistent with the design parameters specified in the relevant chapters of the AISC Specification and associated 'Commentary', unless specifically noted otherwise. As both the 2005 ([Ref. 1](#)) and 2010 ([Ref. 2](#)) versions are supported, where clauses are specific to a particular version these are indicated as (360-05) or (360-10) as appropriate.

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

Section Classification

Cross-section classification is determined using Table B4.1 (360-05), or Tables B4.1a+B4.1b (360-10).

D2. Axial Tension

If axial tension exists, tensile yielding and rupture checks are performed at the point of maximum tension in accordance with Eqns D2.1 and D2.2.

A warning is also issued if the slenderness ratio L/r exceeds 300.

E. Axial Compression

If axial compression exists, the member is assessed for Flexural Buckling and for Torsional and Flexural Torsional buckling. The compressive strength is determined in accordance with Eqns E3.1 and E4.1. For double angles these equations are subject to the modifications of Section E6.

The member length or member sub lengths between braces are checked for:

- Flexural buckling about major axis - for each braced length between adjacent points of major axis lateral bracing and or torsional bracing.
- Flexural buckling about minor axis - for each braced length between adjacent points of minor axis lateral bracing and or torsional bracing.
- Torsional and flexural torsional buckling - for each braced length between adjacent points of torsional bracing (this check is not applied to hollow sections.)

For any braced length, the required compressive force P_r is taken as the maximum compressive force in the relevant length.

A warning is also issued if the slenderness ratio KL/r exceeds 200.

Seismic Design Rules

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05) ([Ref. 8](#)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems.

See "Assumptions/Limitations of the Seismic Provisions" for a list of the assumptions and limitations that apply with respect to the application of these rules to *Tekla Structural Designer* models.

The rules applied depend upon the seismic load resisting system as defined in the AISC Seismic Provisions and are listed below:

For a braced frame

13. Special Concentrically Braced Frames (SCBF)
 - 13.2d. Classification

- 13.2b. brace required strength
- 13.2a. brace slenderness limit
- 13.2e. built up members - double angles

14. Ordinary Concentrically Braced Frames (OCBF)

- 14.2. Classification
- 14.2. Bracing members, V or A braces

15. Eccentrically Braced Frames (EBF) - Beyond Scope

Buckling resistant braced frame

16. Buckling Restrained Braced Frames (BRBF) - Beyond Scope

Frames containing composite beams

Composite Special Concentrically Braced Frames (C-SCBF) - Beyond Scope

Composite Ordinary Braced Frames (C-OBF) - Beyond Scope

Composite Eccentrically Braced Frames (C-EBF) - Beyond Scope

Truss Member Design to AISC 360

Design Method

Unless explicitly stated all truss calculations will adopt either a load and resistance factor design (LRFD) or an allowable strength design (ASD) as consistent with the design parameters as specified in the AISC Specification and associated Commentary.

Design Checks

Truss Members can either be defined manually, or the process can be automated using the **Truss Wizard**. Irrespective of the method used the resulting Truss Members will be one of four types:

- Internal
- Side
- Bottom
- Top

Depending on the type, different design procedures are adopted.

Internal and Side Truss Members

The design checks for internal and side truss members are the same as those for braces. With the exception that seismic forces are not designed for. See: 'Theory and Assumptions' in the Braces Chapter.

Top and Bottom Truss Members

The design checks for top and bottom truss members are the same as those for beams. With the exception that seismic forces are not designed for. See: 'Theory and Assumptions' in the General Beams Chapter

References

- 1. American Institute of Steel Construction.** ANSI/AISC 360-05 Specification for structural steel buildings. **AISC, 2005.**
- 2. American Institute of Steel Construction.** ANSI/AISC 360-10 Specification for structural steel buildings. **AISC, 2010.**
- 3. American Concrete Institute.** Building Code Requirements for Structural Concrete and Commentary. ACI 318-08. **ACI, 2008.**
- 4. Brockenbrough, R. L. & Merritt, F. S.** Structural Steel Designer's Handbook. Second Edition. **McGraw-Hill 1994 USA.**
- 5. American Institute of Steel Construction.** Steel Design Guide Series 11. Floor Vibrations due to Human Activity. **AISC, 1997.**
- 6. American Society of Civil Engineers.** Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-05. **ASCE, 2006.**
- 7. American Society of Civil Engineers.** Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-10. **ASCE, 2010.**
- 8. American Institute of Steel Construction.** ANSI/AISC 341-05 Seismic Provisions for Structural Steel Buildings. **AISC, 2006.**

Assumptions & Limitations of the Seismic Provisions (AISC 341-05)

This section describes the assumptions and limitations that apply with respect to the application of the AISC Seismic Provisions (AISC 341-05) ([Ref. 8](#)) within *Tekla Structural Designer*.

9. Special Moment Frames (SMF)

9.2. Beam-to-Column connections

Beam to column connections used in the SLRS are assumed to satisfy the requirements of 9.2a, b, c & d

9.3. Panel zone of Beam-to-Column Connections

Panel zone of beam to column connections assumed to satisfy the requirements of 9.3a, b, & c

9.5. Continuity Plates

Continuity plates are assumed to comply with 9.5

9.6. Column-Beam Moment Ratio

All General Columns in SMFs - The exceptions in 9.6 are ignored and the check is performed for all SMFs



As part of the check, the shear at the plastic hinge in each SMF beam connected to the SMF column at each level is calculated and used to calculate M_p _beam. This shear component is only correct following second-order analysis. If a first-order analysis is run instead, the value of shear will not be correct.

9.7. Lateral Bracing at Beam-Column Connections

Beam-column connections are always assumed braced as per 9.7a

9.8. Lateral Bracing of Beams

All General Beams in SMFs - The position of lateral braces will not be checked for the location of points of concentrated force or positions of plastic hinge

Lateral braces will not be designed to meet the additional criteria

9.9. Column Splices

Column splices are assumed to comply with 9.9. Column splices in columns not part of the SLRS are assumed to comply with 8.4b.

8.5. Column bases

Column bases are assumed to comply with 8.5a, b & c

10. Intermediate Moment Frame (IMF)

10.2. Beam-to-Column Connections

Beam-to-column connections used in the SLRS are assumed to satisfy the requirements of 10.2a, b, c & d

10.3. Panel Zone of Beam-to-Column Connection

No additional requirements

10.5. Continuity Plates

Continuity plates are assumed to comply with 10.5

10.6. Column-Beam Moment Ratio

No additional requirements

10.7. Lateral Bracing at Beam-to-Column connections

No additional requirements

10.8. Lateral Bracing of Beams - max spacing

The position of lateral braces will not be checked for the location of points of concentrated force or positions of plastic hinge.

Lateral braces will not be designed to meet the additional criteria.

10.9. Column Splices

Column splices are assumed to comply with 10.9. Column splices in columns not part of the SLRS are assumed to comply with 8.4b.

8.5. Column Bases

Column bases are assumed to comply with 8.5a, b & c

11. Ordinary Moment Frame (OMF)

11.2. Beam-to-Column Connections

Beam-to-column connections used in the SLRS are assumed to satisfy the requirements of 11.2a, b, c & d

11.3. Panel Zone of Beam-to-Column Connection

No additional requirements

11.5. Continuity Plates

Continuity plates are assumed to comply with 11.5

11.6. Column-Beam Moment Ratio

No additional requirements.

11.7. Lateral Bracing at Beam-to-Column connections

No additional requirements

11.8. Lateral Bracing of Beams

No additional requirements

11.9. Column Splices

Column splices are assumed to comply with 11.9. Column splices in columns not part of the SLRS are assumed to comply with 8.4b.

8.5. Column Bases

Column bases are assumed to comply with 8.5a, b & c

13. Special Concentrically Braced Frames (SCBF)

13.2. Members

Lateral force distribution - It is assumed that the force resisted by tension braces is between 30% and 70% of the total horizontal force along the line of braces as per 13.2c

Columns and braces - all General Columns and braces in SCBFs - Only single angle slenderness between stitches is checked by the software, it is assumed that the other stitching requirements are met in accordance with 13.2e

13.3. Required Strength of Bracing Connections

Bracing connections and where applicable beam- to-column connections used in the SLRS are assumed to satisfy the requirements of 13.3a, b, & c. The beam end reactions in the software take no account of these forces.

13.4. Special bracing configuration requirements

13.4a. V and inverted V type

- Beams
 - it is assumed that the beams are continuous between columns
- max lat brace spacing $L_b \leq L_{pd}$ (see below)
- it is assumed that the user will apply the relevant lateral restraint at point of braces to comply with 13.4a
- lateral braces will not be designed to meet the additional criteria - it is assumed that the user will check this independently.

13.4b. K braces are not permitted.

13.5. Column Slices

Column splices are assumed to comply with 13.5. Column splices in columns not part of the SLRS are assumed to comply with 8.4b.

13.6. Protected Zone

The protected zone is assumed to comply with 13.6.

8.5. Column Bases

Column bases are assumed to comply with 8.5a, b & c

14. Ordinary Concentrically Braced Frames (OCBF)

14.2. brace members

K braces are not permitted as they are beyond current scope

14.3. Special bracing configuration requirements

Beams with V type and inverted V

- Beams
 - it is assumed that the beams are continuous between columns
- max lat brace spacing $L_b \leq L_{pd}$ (see below)
- it is assumed that the user will apply the relevant lateral restraint at point of braces to comply with 13.4a

- lateral braces will not be designed to meet the additional criteria - it is assumed that the user will check this independently.

14.4. Required strength of bracing connections

Bracing connections used in the SLRS are assumed to satisfy the requirements of 14.4

14.5. OCBF above seismic isolation systems

These are currently beyond the scope of the program.

8.4. Column splices

Column splices are assumed to comply with 8.4a. Column splices in columns not part of the SLRS are assumed to comply with 8.4b.

8.5. Column bases

Column bases are assumed to comply with 8.5a, b & c

