Tekla Structural Designer 2017

Reference Guide – Australia Standards

March 2017
(5.0.0.7)

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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 storey 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

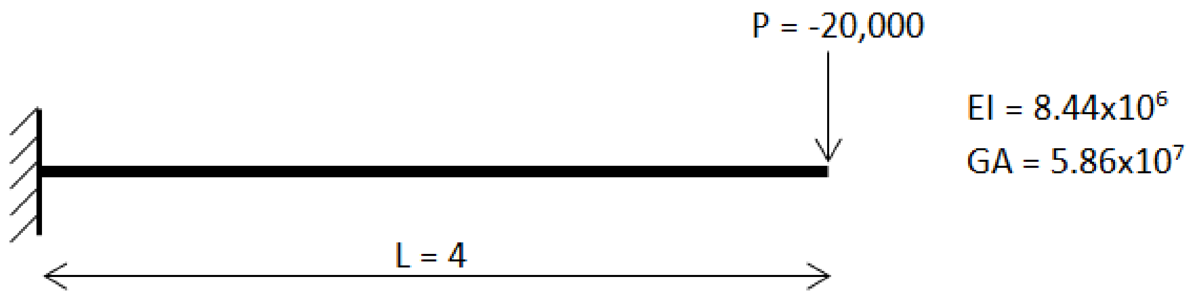
A small number of verification examples are included in this section. Our full automatic test suite for the Solver contains many hundreds of examples which are run and verified every time the Solver is enhanced.

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%

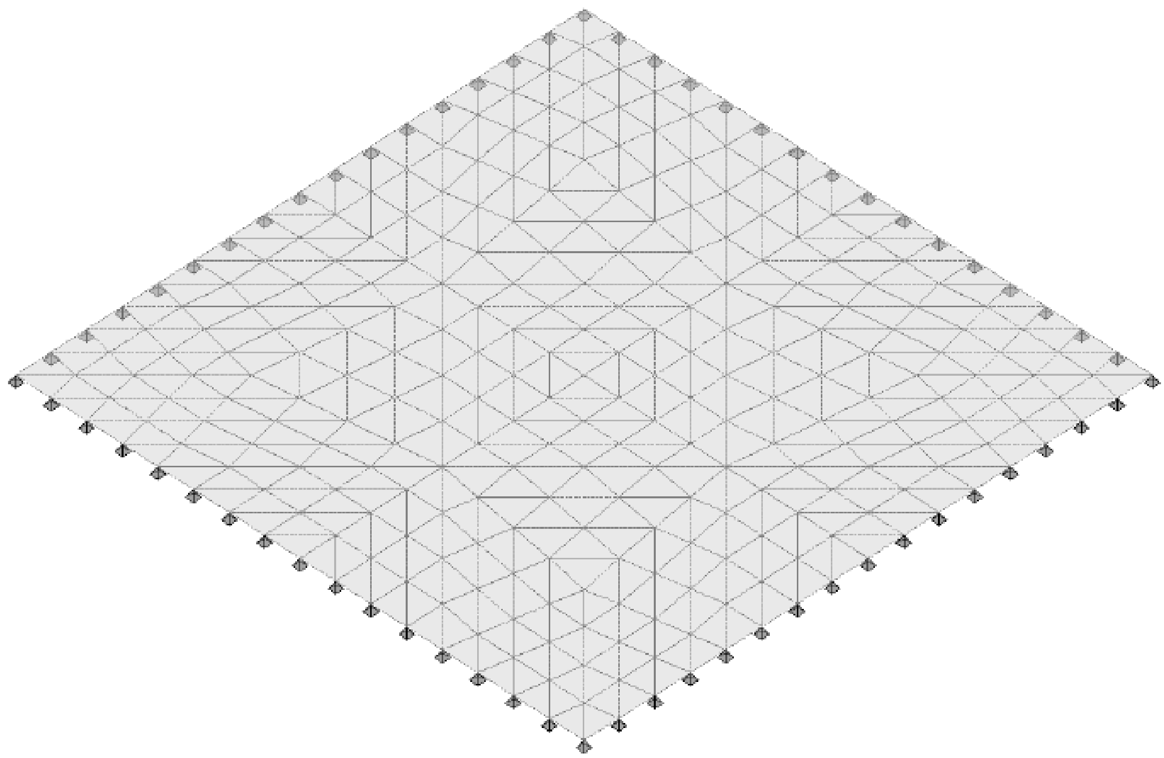
Conclusion

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 $E=2x10^{11}$, $G=7.7x10^{10}$ and $\rho=7849$.



Assumptions

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

Key Results

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

Result	Theoretical Value	Comparison 1	Solver Value	% Error
Mid-span deflection	7.002×10^{-3}	6.990×10^{-3}	7.031×10^{-3}	0.43%
Mid Span Moment	23616	23708	23649	0.14%

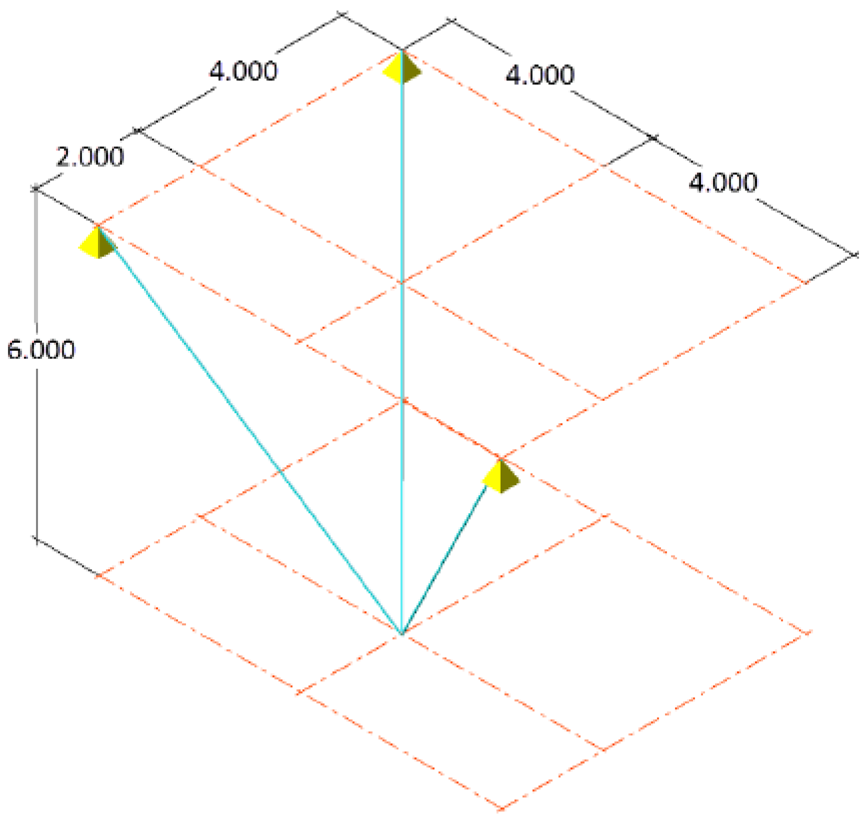
Conclusion

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

1st Order Linear - 3D truss

Problem Definition

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



Key Results

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

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

Conclusion

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

1st Order linear - Thermal Load on Simply Supported Beam

Problem Definition

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

Assumptions

The roller pin is assumed to be frictionless.

Key Results

Result	Theoretical	Theoretical	Solver	%

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

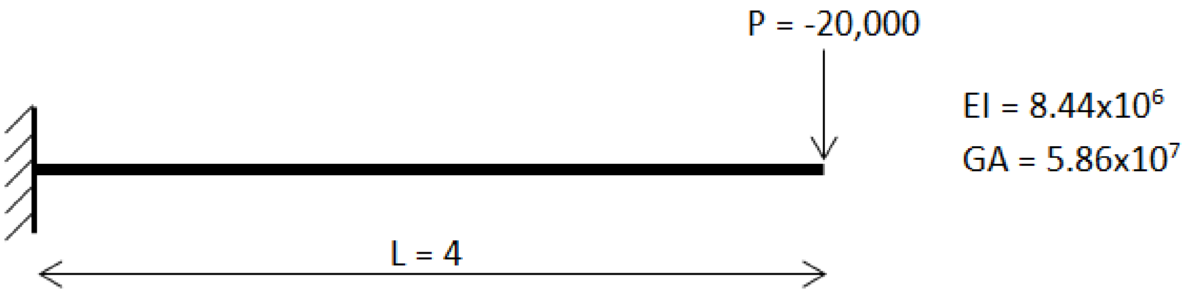
Conclusion

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

1st Order Nonlinear - Simple Cantilever

Problem Definition

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



Assumptions

Flexural and shear deformations are included.

Key Results

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

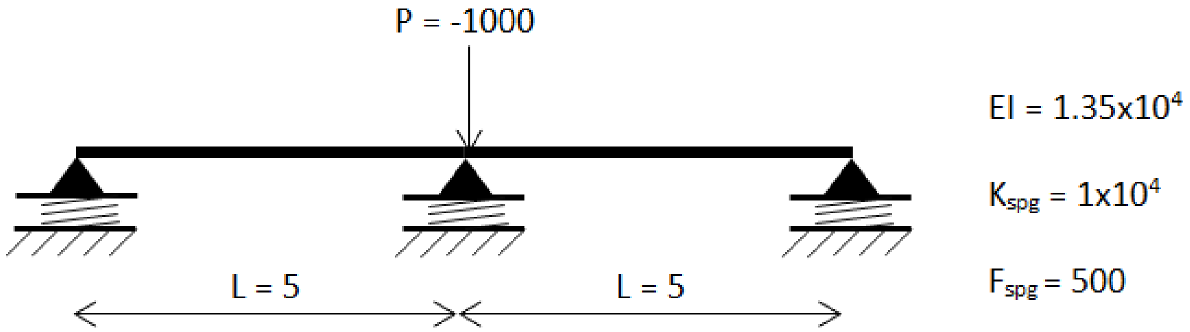
Conclusion

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

1st Order Nonlinear - Nonlinear Supports

Problem Definition

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



Assumptions

Axial and shear deformations are ignored.

Key Results

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

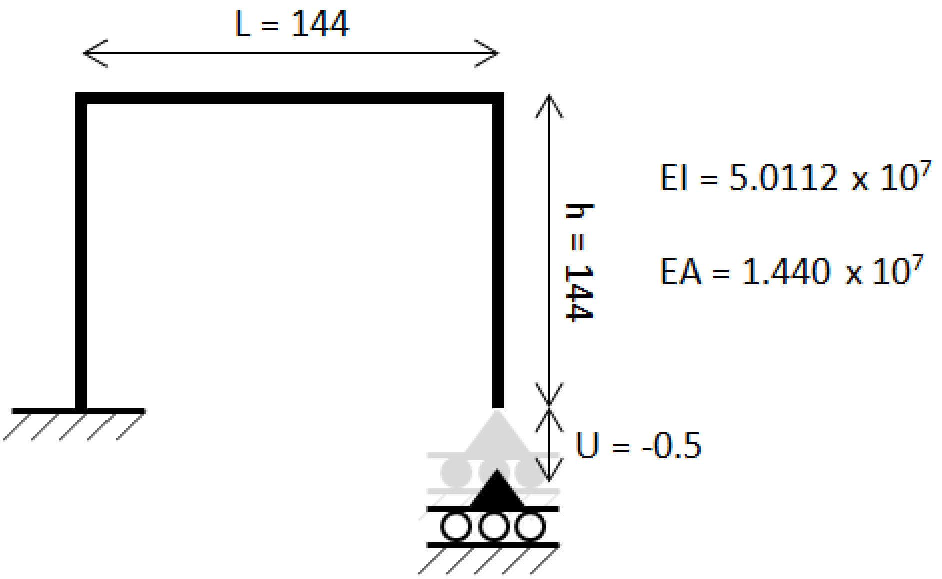
Conclusion

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

1st Order Nonlinear - Displacement Loading of a Plane Frame

Problem Definition

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



Assumptions

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

Key Results

Results were compared with two other independent analysis packages.

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

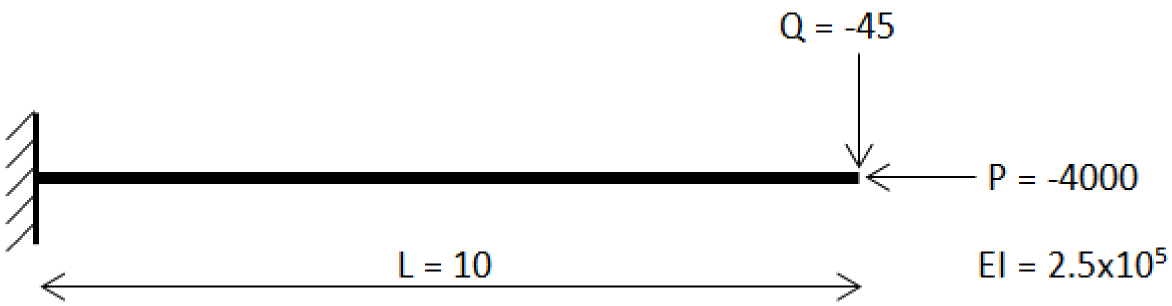
Conclusion

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

2nd Order Linear - Simple Cantilever

Problem Definition

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



Assumptions

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

Key Results

Results were compared with an independent analysis package.

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

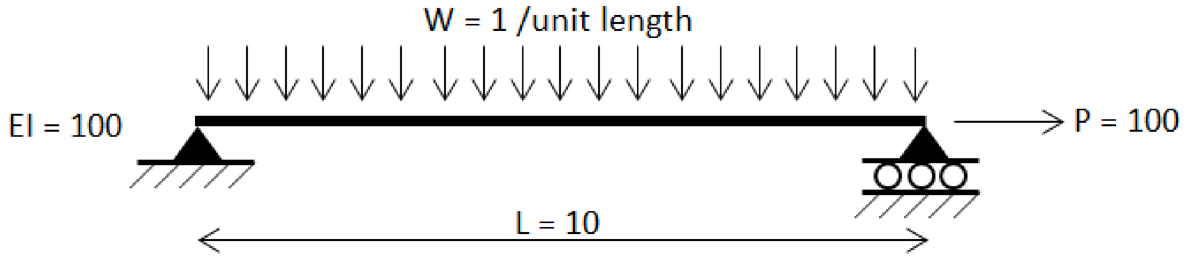
Conclusion

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

2nd Order linear - Simply Supported Beam

Problem Definition

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



Assumptions

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

Key Results

The theoretical value for deflection and moment are calculated as:

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

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

Where *U* is a variable calculated:

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

Conclusion

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

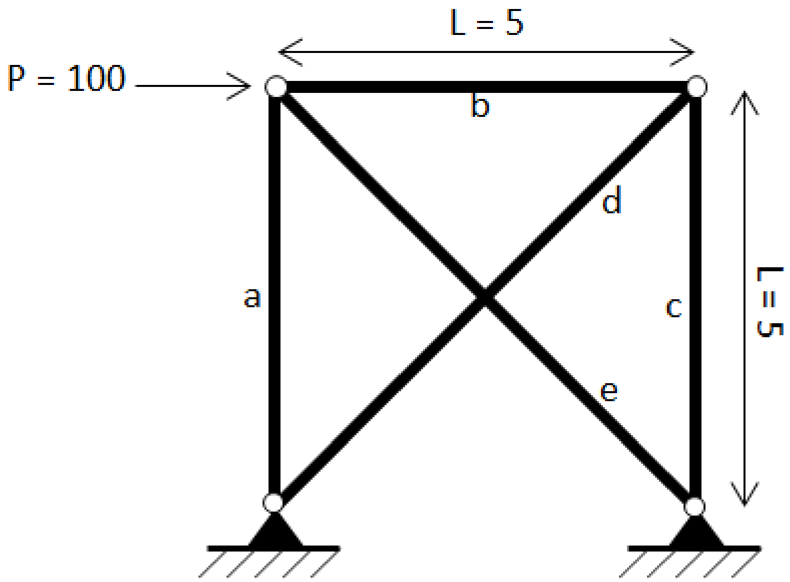
Reference

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

2nd Order Nonlinear - Tension Only Cross Brace

Problem Definition

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



Assumptions

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

Key Results

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

Result	Theoretical Formula	Theoretical Value	Solver Value	% Error
a	0	0	0	0

b	-P	-100	-100	0
c	-P	-100	-100	0
d	$P\sqrt{2}$	141.42	141.42	0
e	0	0	0	0

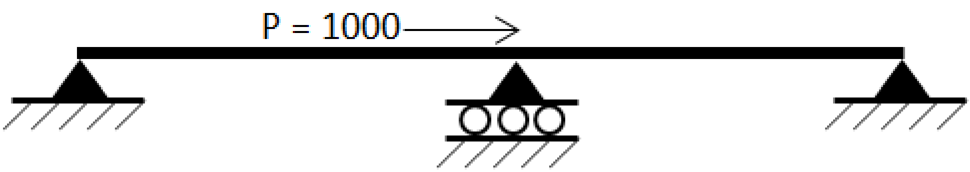
Conclusion

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

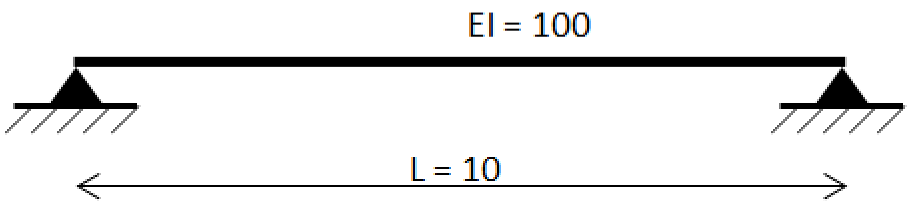
Conclusion

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%

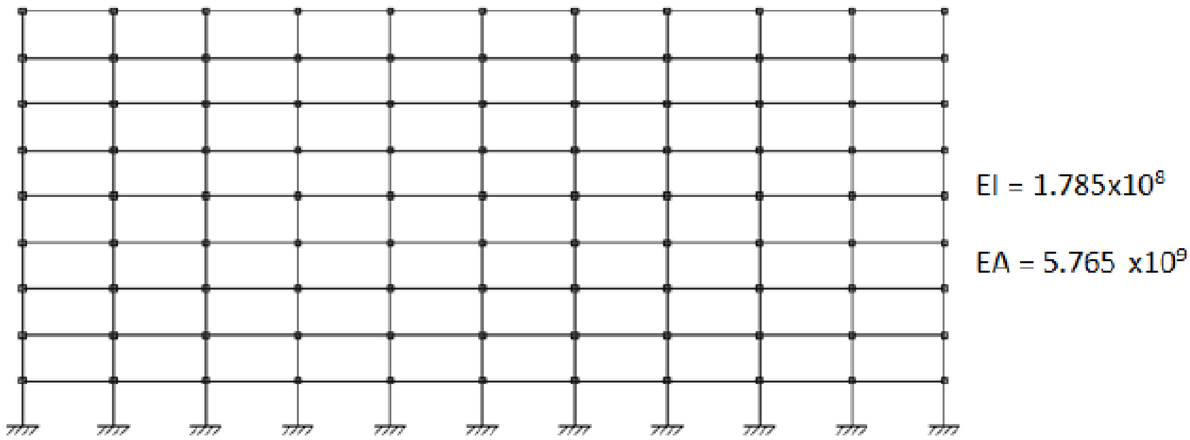
Conclusion

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.

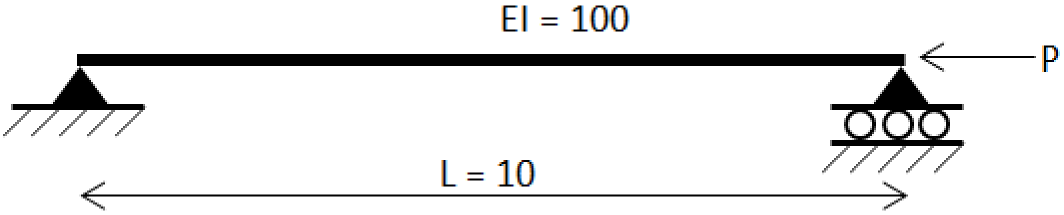
References

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%

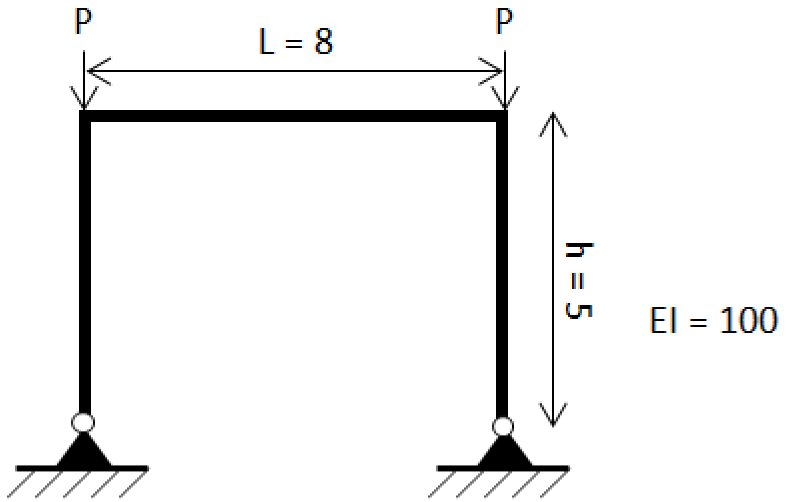
Conclusion

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.

Australian Standards Loading to AS/NZS 1170.0 and AS 1170.1

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

Load Cases

Loadcase Types

The following load case types can be created:

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

temperature	N/A	N/A	N/A	N/A
settlement	N/A	N/A	N/A	N/A

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.

Imposed and Roof Imposed Loads

Imposed Load Reductions

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

Wind Loads

The AS 1170.2 Wind Wizard

The Wind Wizard is not included in this release.

Simple Wind Loading

Simple wind loads can be applied via element or structure loads.

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...**

Manually Defined Combinations

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

Notional Horizontal Forces (NHFs)

NHF's are automatically derived from the load cases within the current combination, their magnitude being calculated as 0.2% of the factored vertical load that passes through any beam/column intersection in the structure.

The values of the NHFs may vary for each load combination.

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

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

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

The Combinations Generator

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

Combination Generator - Combinations

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

The following basic load combinations are created:-

- 1.35 (Permanent)
- 1.2 (Permanent) + 1.5 (Imposed)
- 1.2 (Permanent) + 1.5 (ψ_l * Long-term Imposed)
- 1.2 (Permanent) + 1.0 (Wind) + 1.0 (ψ_c * Imposed)
- 0.9 (Permanent) + 1.0 (Wind)

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

The combination names are generated automatically.

Combination Generator - Service

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

The following basic load combinations are created:-

- 1.0 (Permanent)
- 1.0 (ψ_s * Imposed)
- 1.0 (ψ_l * Imposed)
- 1.0 (Wind)

Combination Generator - NHF

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

Any combination with wind in is automatically greyed.

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

Combination Classes

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

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

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

Steel Design to AS 4100

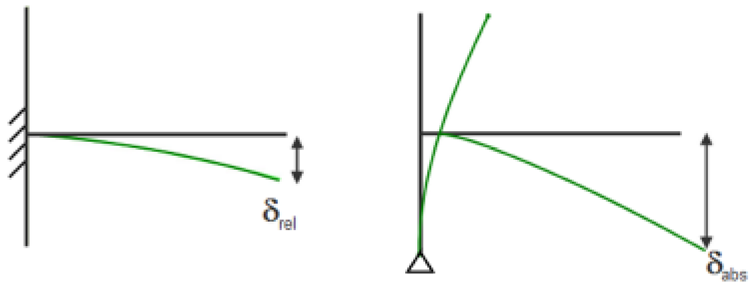
Tekla Structural Designer designs steel members and composite members to a range of international codes. This reference guide specifically describes the design methods applied when the steel design and composite design resistance codes are set as AS 4100 and AS 2327.1 respectively.

Unless explicitly noted otherwise, all clauses, figures and tables referred to are from AS 4100-1998/Amdt 1-2012 (Ref. 1); apart from the Composite Beam section, within which references are to AS 2327.1-2003 (Ref. 2) unless otherwise stated.

Basic Principles

Deflection checks

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



Relative Deflection   Absolute Deflection

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

Steel Beam Design to AS 4100

Design Method

Unless explicitly stated all calculations are in accordance with the relevant sections of AS 4100 (Ref. 1). You may find the Commentary (Ref. 3) to the Standard published by Standards Australia International useful.

Ultimate Limit State (Strength)

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

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

Classification

General

The classification of the cross section is in accordance with AS 4100.

Beams can be classified for flexure about either principal axis as:

- Compact
- Non-compact
- Slender

Slender sections about either axis will not be designed in *Tekla Structural Designer*.

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

Shear Capacity

The shear check is performed according to AS 4100 Clause 5.11.

For rolled and welded I- and H-sections, and for Channel sections, an approximately uniform shear stress distribution is assumed when calculating major axis shear capacity (to Clause 5.11.2). For these same sections a non-uniform shear stress distribution is assumed when calculating minor axis shear capacity (to Clause 5.11.3), with a shear stress ratio $f^*_{vm} / f^*_{va} = 1.5$

For hollow sections a non-uniform shear stress distribution is assumed when calculating both major and minor axis shear capacity (to Clause 5.11.3). For a CHS section the nominal shear yield capacity is taken per Clause 5.11.4, while RHS and SHS sections assume a shear stress ratio $f^*_{vm} / f^*_{va} = 3 * (2 * b + d) / [2 * (3 * b + d)]$ for major axis shear capacity and $f^*_{vm} / f^*_{va} = 3 * (2 * d + b) / [2 * (3 * d + b)]$ for minor axis shear capacity.

Shear buckling

For rolled and welded I- and H-sections, and for Channel sections, about the major axis, and also for RHS and SHS sections about both axes, when the shear panel depth to thickness ratio exceeds $82/\sqrt{(f_y/250)}$ then the shear buckling capacity will be calculated per Clause 5.11.5.1 assuming an unstiffened shear panel.

Note that for rolled and welded I- and H-sections, and for Channel sections, f_y will be taken as the yield strength of the web based on t_w.

Moment Capacity (Section)

The (section) moment capacity check is performed according to AS 4100 Clause 5.1 for the moment about the x-x axis (Mx) and about the y-y axis (My), at the point under consideration.

For (member) moment capacity refer to the section Lateral Torsional Buckling Resistance (Member Moment Capacity).

Note that for all section types, the effective section modulus about the major axis (Z_{ex}) will be based on the minimum slenderness ratio considering both flange and web. Internally *Tekla Structural Designer* will calculate the following:

- flange slenderness ratio, $z_f = (\lambda_{ey} - \lambda_{ef}) / (\lambda_{eyf} - \lambda_{epf})$
- web slenderness ratio, $z_w = (\lambda_{eyw} - \lambda_{ew}) / (\lambda_{eyw} - \lambda_{epw})$

For sections which have flexure major class either Compact or Non-compact, the effective section modulus about the major axis (Z_{ex}) will then be calculated by:

- $Z_{ex} = Z_x + [MIN(z_f, z_w, 1.0) * (Z_c - Z_x)]$ where $Z_c = MIN(S_x, 1.5 * Z_x)$

Note that for Channel sections under minor axis bending:

- if there is single curvature with the flange tips in compression then Z_{ey} will be based on Z_{eyR}

- if there is single curvature with the web in compression then Z_{ey} will be based on Z_{eyL}
- if there is double curvature then Z_{ey} will be based on the minimum of Z_{eyR} and Z_{eyL}

Combined Bending & Shear Capacity (Section)

The combined bending & shear capacity check is performed according to AS 4100 Clause 5.12.3, assuming bending is resisted by the whole of the cross-section, for the coincident major shear and moment about the x-x axis (Mx) and minor shear and moment about the y-y axis (My), at the point under consideration.

Note that if the (section) moment capacity is found to be less than the design moment then the combined bending & shear check will automatically be set as Fail.

Axial Capacity (Section)

The (section) axial capacity check is performed according to AS 4100 Clause 6.1 for axial compression, or Clause 7.1 for axial tension, using the gross cross-section area for A_n in both cases.

Note that member (axial compression) capacity is a buckling check and as such is considered under the heading Compression Buckling

Ultimate Limit State (Buckling)

Lateral Torsional Buckling Resistance (Member Moment Capacity)

For beams with major axis bending, a Lateral Torsional Buckling (LTB) check is required, except in the following circumstances:

- when the segment critical flange is continuously restrained for LTB, or
- when bending exists about the minor axis only, or
- when the section is a CHS, or
- when the segment length satisfies the relevant limit given in Clause 5.3.2.4 of AS 4100

In the latter case, when calculating the limiting LTB length, the ratio β_m will be taken as -0.8 if the segment has major axis bending induced by transverse load within its length, and the ratio of end moments otherwise.

The LTB resistance (member moment capacity) check is performed according to AS 4100 Clause 5.6

Note that the moment modification factor α_m will be calculated from the equation given in AS 4100 Clause 5.6.1.1 (a) (iii) except for cantilevers, where α_m will be 0.25 if the free end moment is greater than the ignore forces major moment, and 1.0 otherwise.

The twist restraint factor k_t will be determined by consideration of the LTB cross-section restraints at either end of the segment, per Table 5.6.3(1) of AS 4100

The load height factor k_l will default to 1.4 for a non-cantilever and 2.0 for a cantilever.

The lateral rotation factor k_r will default to 1.0.

Compression Buckling Resistance (Member Capacity under Axial Compression)

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

The compression buckling resistance (member capacity under axial compression) check is performed according to AS 4100 Clause 6.3

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

Combined Actions Resistance

Important Note

Clause 8.2 of AS 4100 defines the design bending moments to be used in the combined actions checks as either amplified moments from a first order linear elastic analysis or the moments resulting directly from a second order elastic analysis. *Tekla Structural Designer* will not provide amplified moments from a first order linear elastic analysis and you are expected to switch to second order analysis to complete the design for combined actions.

Combined Actions Resistance - Section Capacity

The combined actions section capacity check is performed according to AS 4100 Clause 8.3

The higher tier equations will be used automatically if the conditions for their use are met.

Note that if the design axial force exceeds the design axial section capacity then the check will automatically be set as Fail.

In the section capacity check, the design forces are those which are coincident at any one point along the member.

Combined Actions Resistance - Member Capacity

The combined actions member capacity check is performed according to AS 4100 Clause 8.4

The higher tier equations will be used automatically if the conditions for their use are met.

In the higher tier equation for M_i, the ratio β_m will be based on the relevant strut length; if the strut length has bending induced by transverse load within its length then β_m will be taken as -1.0, and the ratio of end moments otherwise.

In the higher tier equation for M_{ox}, the ratio β_m will be based on the LTB segment length, and taken as the ratio of end moments.

Note that if the design axial force exceeds the design axial member capacity then the check will automatically be set as Fail.

In the member capacity check, the design forces are the maxima in the design length being considered, where the design lengths are based on the major and minor strut lengths within a loop of LTB lengths.

Therefore, since any one design length will comprise both major and minor strut lengths, the design axial force for each design length will be taken as the maximum axial compression or axial tension force from the major and minor strut lengths considered together.

Since both axial compression and axial tension are to be considered, but make use of different equations, then in cases where both axial forces exist within a design length the compression equations and tension equations will both be evaluated and the worst case of the two will be reported.

Note that in bi-axial bending cases, zero axial force will be treated as compression.

Web Openings

The checks for beams with web openings are not included in this release.

Serviceability limit state

Beams are assessed for deflection. Only the total load deflection is active by default, with a span/over value assigned of 250 per Table B1 of AS 4100.

Composite Beam Design to AS 2327.1

The design of composite beams is not included in this release.

Steel Column Design to AS 4100

Design method

Unless explicitly stated all calculations are in accordance with the relevant sections of AS 4100 (Ref. 1). You may find the Commentary (Ref. 3) to the Standard published by Standards Australia International useful.

Ultimate Limit State (Strength)

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

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

Hollow sections

The checks for CHS, RHS and SHS relate to 'hot-finished hollow sections' only - 'cold-formed hollow sections' are not included in this release.

Classification

The flexural classification of the cross section is in accordance with AS 4100

Columns can be classified for flexure about either principal axis as:

- Compact
- Non-compact
- Slender

Slender sections about either axis will not be designed in *Tekla Structural Designer*.

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

Shear Capacity

The shear check is performed according to AS 4100 Clause 5.11.

For rolled and welded I- and H-sections, and for Channel sections, an approximately uniform shear stress distribution is assumed when calculating major axis shear capacity (to Clause 5.11.2). For these same sections a non-uniform shear stress distribution is assumed when calculating minor axis shear capacity (to Clause 5.11.3), with a shear stress ratio $f^*_{vm} / f^*_{va} = 1.5$

For hollow sections a non-uniform shear stress distribution is assumed when calculating both major and minor axis shear capacity (to Clause 5.11.3). For a CHS section the nominal shear yield capacity is taken per Clause 5.11.4, while RHS and SHS sections assume a shear stress ratio $f^*_{vm} / f^*_{va} = 3 * (2 * b + d) / [2 * (3 * b + d)]$ for major axis shear capacity and $f^*_{vm} / f^*_{va} = 3 * (2 * d + b) / [2 * (3 * d + b)]$ for minor axis shear capacity.

Shear buckling

For rolled and welded I- and H-sections, and for Channel sections, about the major axis, and also for RHS and SHS sections about both axes, when the shear panel depth to thickness ratio exceeds $82/\sqrt{(f_y/250)}$ then the shear buckling capacity will be calculated per Clause 5.11.5.1 assuming an unstiffened shear panel.

Note that for rolled and welded I- and H-sections, and for Channel sections, f_y will be taken as the yield strength of the web based on t_w.

Moment Capacity (Section)

The (section) moment capacity check is performed according to AS 4100 Clause 5.1 for the moment about the x-x axis (Mx) and about the y-y axis (My), at the point under consideration.

For (member) moment capacity refer to the section Lateral Torsional Buckling Resistance (Member Moment Capacity).

Note that for all section types, the effective section modulus about the major axis (Z_{ex}) will be based on the minimum slenderness ratio considering both flange and web. Internally *Tekla Structural Designer* will calculate the following:

- flange slenderness ratio, $z_f = (\lambda_{ey} - \lambda_{ef}) / (\lambda_{eyf} - \lambda_{epf})$
- web slenderness ratio, $z_w = (\lambda_{eyw} - \lambda_{ew}) / (\lambda_{eyw} - \lambda_{epw})$

For sections which have flexure major class either Compact or Non-compact, the effective section modulus about the major axis (Z_{ex}) will then be calculated by:

- $Z_{ex} = Z_x + [MIN(z_f, z_w, 1.0) * (Z_c - Z_x)]$ where $Z_c = MIN(S_x, 1.5 * Z_x)$

Note that for Channel sections under minor axis bending:

- if there is single curvature with the flange tips in compression then Z_{ey} will be based on Z_{eyR}
- if there is single curvature with the web in compression then Z_{ey} will be based on Z_{eyL}
- if there is double curvature then Z_{ey} will be based on the minimum of Z_{eyR} and Z_{eyL}

Eccentricity Moments

Eccentricity moment will be added algebraically to the coincident real moment (at top or bottom of column stack) only if the resulting 'combined' moment has a larger absolute magnitude than the absolute real moment alone.

The resulting 'combined' design moment (major and/or minor) will be that used in moment capacity, combined bending & shear, LTB, and combined actions checks.

Combined Bending & Shear Capacity (Section)

The combined bending & shear capacity check is performed according to AS 4100 Clause 5.12.3, assuming bending is resisted by the whole of the cross-section, for the coincident major shear and moment about the x-x axis (Mx) and minor shear and moment about the y-y axis (My), at the point under consideration. The design moments may include eccentricity moments - see Moment Capacity (Section): Eccentricity Moments.

Note that if the (section) moment capacity is found to be less than the design moment then the combined bending & shear check will automatically be set as Fail.

Axial Capacity (Section)

The (section) axial capacity check is performed according to AS 4100 Clause 6.1 for axial compression, or Clause 7.1 for axial tension, using the gross cross-section area for A_n in both cases.

Note that member (axial compression) capacity is a buckling check and as such is considered under the heading .

Ultimate Limit State (Buckling)

Lateral Torsional Buckling Resistance (Member Moment Capacity)

For beams with major axis bending, a Lateral Torsional Buckling (LTB) check is required, except in the following circumstances:

- when the segment critical flange is continuously restrained for LTB, or
- when bending exists about the minor axis only, or
- when the section is a CHS, or
- when the segment length satisfies the relevant limit given in Clause 5.3.2.4 of AS 4100

In the latter case, when calculating the limiting LTB length, the ratio β_m will be taken as -0.8 if the segment has major axis bending induced by transverse load within its length, and the ratio of end moments otherwise.

The LTB resistance (member moment capacity) check is performed according to AS 4100 Clause 5.6

Note that the moment modification factor α_m will be calculated from the equation given in AS 4100 Clause 5.6.1.1 (a) (iii) except for cantilevers, where α_m will be 0.25 if the free end moment is greater than the ignore forces major moment, and 1.0 otherwise.

The design moment may include eccentricity moment - see Moment Capacity (Section): Eccentricity Moments - but note in particular that the ratio β_m and the moment modification factor α_m will be based on real moments only.

The twist restraint factor k_t will be determined by consideration of the LTB cross-section restraints at either end of the segment, per Table 5.6.3(1) of AS 4100

The load height factor k_l will default to 1.4 for a non-cantilever and 2.0 for a cantilever.

The lateral rotation factor k_r will default to 1.0.

Compression Buckling Resistance (Member Capacity under Axial Compression)

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

The compression buckling resistance (member capacity under axial compression) check is performed according to AS 4100 Clause 6.3

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

Combined Actions Resistance

Important Note

Clause 8.2 of AS 4100 defines the design bending moments to be used in the combined actions checks as either amplified moments from a first order linear elastic analysis or the moments resulting directly from a second order elastic analysis. *Tekla Structural Designer* will not provide amplified moments from a first order linear elastic analysis and you are expected to switch to second order analysis to complete the design for combined actions.

Combined Actions Resistance - Section Capacity

The combined actions section capacity check is performed according to AS 4100 Clause 8.3

The higher tier equations will be used automatically if the conditions for their use are met.

Note that if the design axial force exceeds the design axial section capacity then the check will automatically be set as Fail.

In the section capacity check, the design forces are those which are coincident at any one point along the member.

Combined Actions Resistance - Member Capacity

The combined actions member capacity check is performed according to AS 4100 Clause 8.4

The higher tier equations will be used automatically if the conditions for their use are met.

In the higher tier equation for M_i, the ratio β_m will be based on the relevant strut length; if the strut length has bending induced by transverse load within its length then β_m will be taken as -1.0, and the ratio of end moments otherwise.

In the higher tier equation for M_{ox}, the ratio β_m will be based on the LTB segment length, and taken as the ratio of end moments, using real moments only.

In the member capacity check, the design forces are the maxima in the design length being considered, where the design lengths are based on the major and minor strut lengths within a loop of LTB lengths.

Therefore, since any one design length will comprise both major and minor strut lengths, the design axial force for each design length will be taken as the maximum axial compression or axial tension force from the major and minor strut lengths considered together.

Note that if the design axial force exceeds the design axial member capacity then the check will automatically be set as Fail.

Since both axial compression and axial tension are to be considered, but make use of different equations, then in cases where both axial forces exist within a design length the compression equations and tension equations will both be evaluated and the worst case of the two will be reported.

Note that in bi-axial bending cases, zero axial force will be treated as compression.

Serviceability limit state

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

- Sway X (mm) and λ_{critx}
- Sway Y (mm) and λ_{crity}
- Twist i.e. Sway X-Y (non-dimensional ratio)

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

A sway assessment is only performed for the column if the Lambda Crit Check box is checked on the Column Properties dialog.

If very short columns exist in the building model these can distort the overall sway classification for the building. For this reason you may apply engineering judgement to uncheck the Lambda Crit Check box for those columns for which a sway assessment would be inappropriate

Steel Brace Design to AS 4100

Design Method

Unless explicitly stated all brace calculations are in accordance with the relevant sections of AS 4100 (Ref. 1).

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

Hollow sections

The checks for CHS, RHS and SHS relate to 'hot-finished hollow sections' only - 'cold-formed hollow sections' are not included in this release.

Classification

No classification is required for braces.

Axial Capacity (Section)

The (section) axial capacity check is performed according to AS 4100 Clause 6.1 for axial compression, or Clause 7.1 for axial tension, using the gross cross-section area for A_n in both cases.

Note that member (axial compression) capacity is a buckling check and as such is considered under the heading Compression Buckling.

Compression Buckling Resistance (Member Capacity under Axial Compression)

The compression buckling resistance (member capacity under axial compression) check is performed according to AS 4100 Clause 6.3

The default effective length factor in each axis is 1.0

References

1. **Standards Australia International**. AS 4100-1998/Amdt 1-2012: Steel structures. **SAI 2012.**
2. **Standards Australia International.** AS 2327.1-2003: Composite structures. Part 1: Simply supported beams. **SAI 2003.**
3. **Standards Australia International.** AS 4100 Supp1-1999: Steel structures – Commentary. (Supplement to AS 4100-1998). **SAI 1999.**
4. **Standards Australia International/Standards New Zealand.** AS/NZS 1170.0:2002 (Including Amendments Nos.1,2,4 and 5). Structural design actions. Part 0: General principles. **SAI/NZS 2011.**
5. **Standards Australia International/Standards New Zealand.** AS/NZS 1170.1:2002 (Including Amendments Nos.1 and 2). Structural design actions. Part 1: Permanent, imposed and other actions. **SAI/NZS 2009.**
6. **Standards Australia International/Standards New Zealand.** AS/NZS 1170.2:2011 (Including Amendments Nos.1 and 2). Structural design actions. Part 2: Wind actions. **SAI/NZS 2012.**

Site Specific Spectra for ELF and RSA

In addition to code spectra, TSD also allows you to define your own site specific spectra for ELF and RSA. This can be required for locations which use another country's loading and design codes where the code spectra are not relevant and so the local site spectra need to be defined for ELF/RSA analysis.

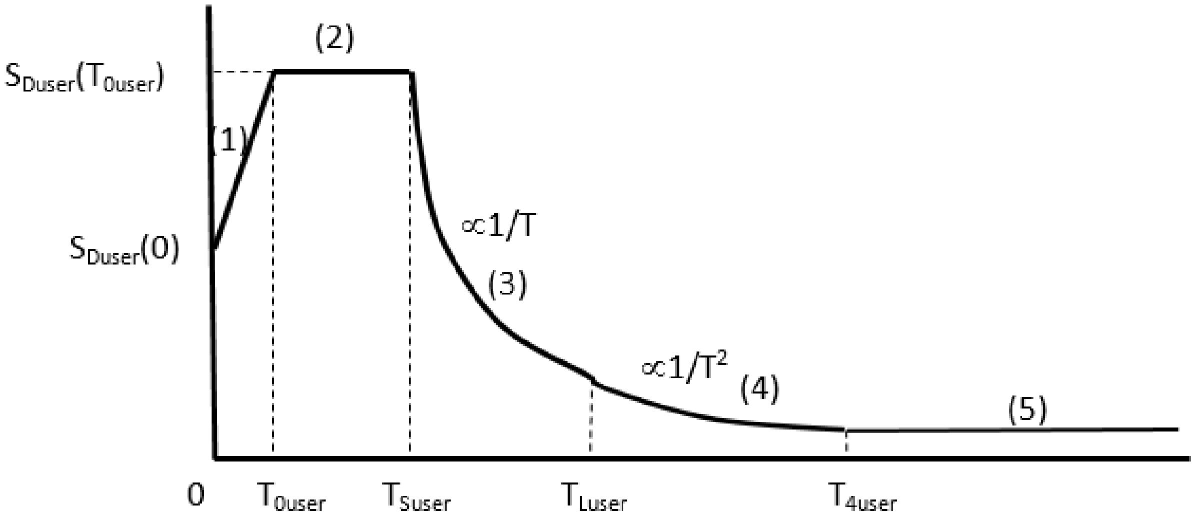
The option to set a site specific spectrum and to define it is part of the seismic wizard - ‘similar’ requirements are applicable to all head codes, (i.e. you are required to specify significant periods and acceleration).

Limitations

No vertical spectrum is considered in the current release.

Eurocodes require the vertical seismic action to be considered when a_{vg} is relatively high,
i.e. a_{vg} > 0.25 m/s^2 for:
- Horizontal members > 20m
- Cantilevers > 5m
- Beam supporting columns

United States (ACI/AISC) Headcode



Parameters

- $S_{Duser}(0)$ – units g
- $S_{Duser}(T_{0user})$ – units g
- T_{0user} – units sec
- T_{Suser} – units sec
- T_{Luser} – units sec
- T_{4user} – units sec, default = 4s
- Input limits
  - $S_{Duser}(0) > 0$
  - $S_{Duser}(T_{0user}) > 0$ if $T_{0user} > 0$
  - $0s \leq T_{0user} < T_{Suser} < T_{Luser} \leq T_{4user}$

Curve Equations

Design Response Spectrum curves for $S_a(g)$ are defined by (note same curve for Dir1 and Dir2)

Line (1) – straight line

Eqn $S_a(g)(T) = S_{Duser}(0) + ((S_{Duser}(T_{0user}) - S_{Duser}(0)) \times T / T_{0user})$

limits $0 \leq T \leq T_{0user}$

Line (2) – straight line

Eqn $S_a(g)(T) = S_{Duser}(T_{0user})$

limits $T_{0user} \leq T \leq T_{Suser}$

Line (3) – curve

Eqn $S_a(g)(T) = S_{Duser}(T_{0user}) \times T_{Suser} / T$

limits $T_{Suser} \leq T \leq T_{Luser}$

Line (4) – curve

Eqn $S_a(g)(T) = S_{Duser}(T_{0user}) \times T_{Suser} \times T_{Luser} / T^2$

limits $T_{Luser} \leq T \leq T_{4user}$

Line (5) – straight line continued from (4)

Adjusted Design Response Spectrum curves for $S_a(g)/(R/I_e)$ are defined by (note different curve for Dir1 and Dir2)

Line (1) – straight line

Eqn $S_D(T) = [S_{Duser}(0) + ((S_{Duser}(T_{0user}) – S_{Duser}(0)) \times T / T_{0user})] /(R/I_e)$

limits $0 \leq T \leq T_{0user}$

Line (2) – straight line

Eqn $S_D(T) = S_{Duser}(T_{0user}) / (R/I_e)$

limits $T_{0user} \leq T \leq T_{Suser}$

Line (3) – curve

Eqn $S_D(T) = S_{Duser}(T_{0user}) \times T_{Suser} / T / (R/I_e)$

limits $T_{Suser} \leq T \leq T_{Luser}$

Line (4) – curve

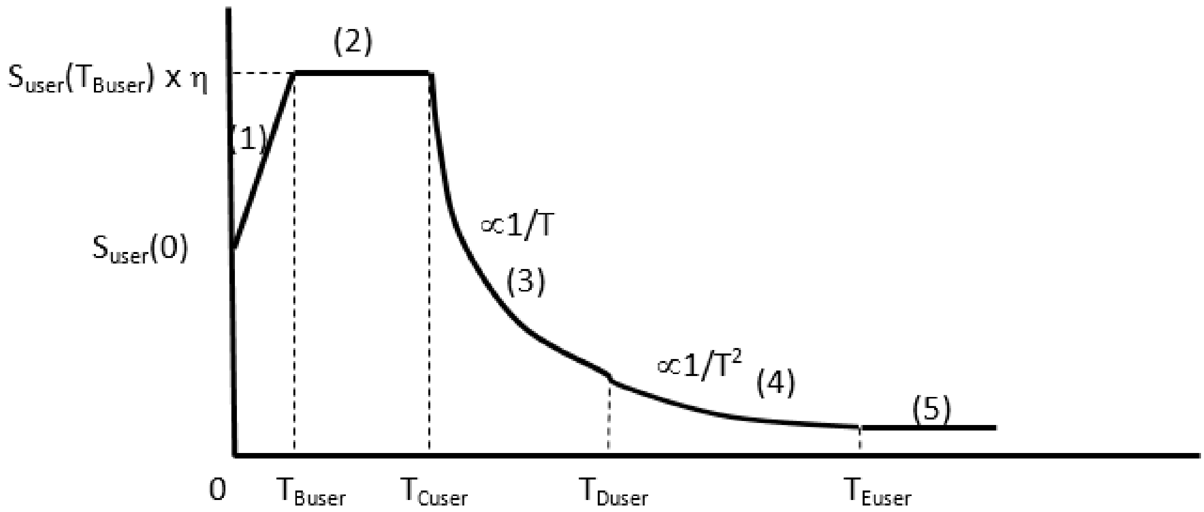
Eqn $S_D(T) = S_{Duser}(T_{0user}) \times T_{Suser} \times T_{Luser} / T^2 / (R/I_e)$

limits $T_{Luser} \leq T \leq T_{4user}$

Line (5) – straight line continued from (4)

If T_{0user} = 0, then no line (1) exists
If T_{Luser} = T_{4user} then no line (4) exists

Europe (Eurocode), United Kingdom (Eurocode), Singapore (Eurocode) Headcodes



Parameters

- $S_{user}(0)$ – units g
- $S_{user}(T_{Buser})$ – units g
- T_{Buser} – units sec
- T_{Cuser} – units sec
- T_{Duser} – units sec
- T_{Euser} – units sec, default = 4s
- Input limits
  - $S_{user}(0) > 0$
  - $S_{user}(T_{Buser}) > 0$
  - $0s \leq T_{Buser} < T_{Cuser} < T_{Duser} \leq T_{Euser}$

Curve Equations

Elastic Response Spectrum curves for S_e/a_g are defined by (note same curve for both Dir1 and Dir2))

Line (1) – straight line

Eqn $S_e/a_g(T) = S_{user}(0) + ((S_{user}(T_{Buser}) \times \eta - S_{user}(0)) \times T / T_{Buser})$

limits $0 \leq T \leq T_{Buser}$

Line (2) – straight line

Eqn $S_e/a_g(T) = S_{user}(T_{Buser}) \times \eta$

limits $T_{Buser} \leq T \leq T_{Cuser}$

Line (3) – curve

Eqn $S_e/a_g(T) = S_{user}(T_{Buser}) \times \eta \times T_{Cuser} / T$

limits $T_{Cuser} \le T \le T_{Duser}$

Line (4) – curve

Eqn $S_e/a_g(T) = S_{user}(T_{Buser}) \times \eta \times T_{Cuser} \times T_{Duser} / T^2$

limits $T_{Duser} \le T \le T_{Euser}$

Line (5) – straight line continued from (4)

Design Spectrum for Elastic Analysis curves for S_d/a_g are defined by (Note two curves, one for Dir1 and one for Dir2)

Line (1) – straight line

Eqn $S_d/a_g(T) = [\ 2/3 \times S_{user}(0) + ((S_{user}(T_{Buser}) / q - 2/3 \times S_{user}(0)) \times T / T_{Buser})]$

limits $0 \le T \le T_{Buser}$

Line (2) – straight line

Eqn $S_d/a_g(T) = S_{user}(T_{Buser}) / q$

limits $T_{Buser} \le T \le T_{Cuser}$

Line (3) – curve

Eqn $S_d/a_g(T) = S_{user}(T_{Buser}) \times T_{Cuser} / T / q$

limits $T_{Cuser} \le T \le T_{Duser}$

Line (4) – curve

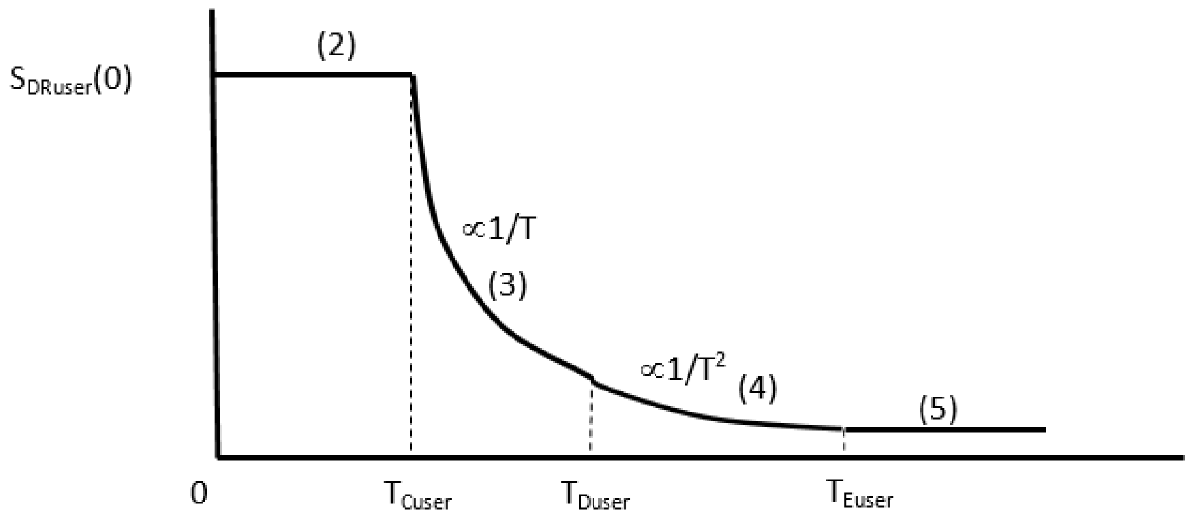
Eqn $S_d/a_g(T) = S_{user}(T_{Buser}) \times T_{Cuser} \times T_{Duser} / T^2 / q$

limits $T_{Duser} \le T \le T_{Euser}$

Line (5) – straight line continued from (4)

If T_{Buser} = 0, then no line (1) exists
If T_{Duser} = T_{Euser} then no line (4) exists

Malaysia (Eurocode) Headcode



Parameters

- $S_{DRuser}(0)$ – units g
- $S_{DRuser}(T_{Buser})$ – units g, (if T_{Buser} = 0 set = $S_{DRuser}(0)$ and read-only)
- T_{Buser} – units sec, default = 0s
- T_{Cuser} – units sec
- T_{Duser} – units sec
- T_{Euser} – units sec, default = 4s
- Input limits
  - $S_{DRuser}(0) > 0$
  - $S_{DRuser}(T_{Buser}) > 0$
  - $0s \leq T_{Buser} < T_{Cuser} < T_{Duser} \leq T_{Euser}$

Curve Equations

Elastic Response Spectrum curves for S_e/a_g are defined by (note same curve for both Dir1 and Dir2)

Line (1) – straight line

Eqn $S_e/a_g(T) = S_{DRuser}(0) + ((S_{DRuser}(T_{Buser}) - S_{DRuser}(0)) \times T / T_{Buser})$

limits $0 \leq T \leq T_{Buser}$

Line (2) – straight line

Eqn $S_e/a_g(T) = S_{DRuser}(T_{Buser})$

limits $T_{Buser} \leq T \leq T_{Cuser}$

Line (3) – curve

Eqn $S_e/a_g(T) = S_{DRuser}(T_{Buser}) \times T_{Cuser} / T$

limits $T_{Cuser} \leq T \leq T_{Duser}$

Line (4) – curve

Eqn $S_e/a_g(T) = S_{DRuser}(T_{Buser}) \times T_{Cuser} \times T_{Duser} / T^2$

limits $T_{Duser} \leq T \leq T_{Euser}$

Line (5) – straight line continued from (4)

Design Spectrum for Elastic Analysis curves for S_d/a_g are defined by (Note two curves, one for Dir1 and one for Dir2)

Line (1) – straight line

Eqn $S_d/a_g(T) = [S_{DRuser}(0) + ((S_{DRuser}(T_{Buser}) - S_{DRuser}(0)) \times T / T_{Buser})] / q$

limits $0 \leq T \leq T_{Buser}$

Line (2) – straight line

Eqn $S_d/a_g(T) = S_{DRuser}(T_{Buser}) / q$

limits $T_{Buser} \leq T \leq T_{Cuser}$

Line (3) – curve

Eqn $S_d/a_g(T) = S_{DRuser}(T_{Buser}) \times T_{Cuser} / T / q$

limits $T_{Cuser} \leq T \leq T_{Duser}$

Line (4) – curve

Eqn $S_d/a_g(T) = S_{DRuser}(T_{Buser}) \times T_{Cuser} \times T_{Duser} / T^2 / q$

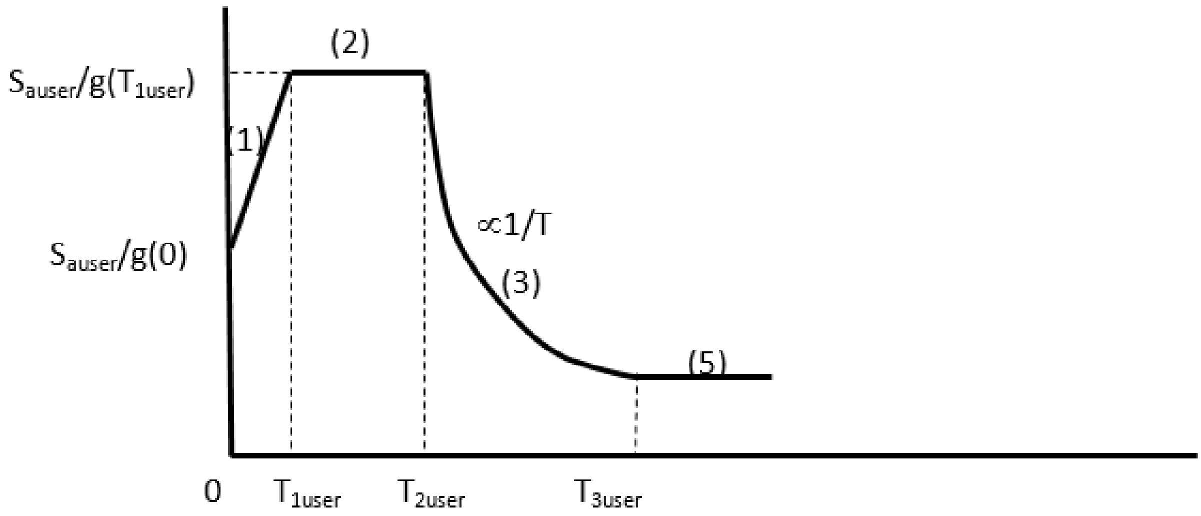
limits $T_{Duser} \leq T \leq T_{Euser}$

Line (5) – straight line continued from (4)

If T_{Buser} = 0, then no line (1) exists (as in the example above)
If $T_{Duser} = T_{Euser}$ then no line (4) exists

The above by default does not quite equate to Line (4) in the Malaysian NA because there is an extra term $(2 \times \pi)^2 / T^2 \times \gamma I \times m_r \times (T - T_D)$ that tweaks the $1/T^2$ curve slightly. In Sarawak, this term is non-existent, in Peninsular Malaysia it does not exist for Flexible soils. In the TSD User defined spectrum it is also non-existent.

India (IS) Headcode



Note In this example $T_{3user} = T_{4user}$ so there is no line (4)

Parameters

- Damping $factor_{user}$
- $S_{auser}/g(0)$ – units g
- $S_{auser}/g(T_{1user})$ – units g
- T_{1user} – units sec
- T_{2user} – units sec
- T_{3user} – units sec, default = 4s
- T_{4user} – units sec, default = 4s
- Input limits
  - $S_{auser}/g(0) > 0$
  - $S_{auser}/g(T_{1user}) > 0$
  - $0s \leq T_{1user} < T_{2user} < T_{3user} \leq T_{4user}$

Curve Equations

Average Response Spectrum curves for S_a/g are defined by (note same curve for both Dir1 and Dir2)

Line (1) – straight line

Eqn $S_a(g)(T)$ = Damping factor$_{user}$ x [$S_{auser}/g(0)$ + (($S_{auser}/g(T_{1user})$ – $S_{auser/g}(0)$) x T / T_{1user})]

limits $0 \leq T \leq T_{1user}$

Line (2) – straight line

Eqn $S_a(g)(T)$ = Damping factor$_{user}$ x $S_{auser}/g(T_{1user})$

limits $T_{1user} \leq T \leq T_{2user}$

Line (3) – curve

Eqn $S_a(g)(T)$ = Damping factor$_{user}$ x $S_{auser}/g(T_{1user})$ x T_{2user} / T

limits $T_{2user} \leq T \leq T_{3user}$

Line (4) – curve

Eqn $S_a(g)(T)$ = Damping factor$_{user}$ x $S_{auser}/g(T_{1user})$ x T_{2user} x T_{3user} / T^2

limits $T_{3user} \leq T \leq T_{4user}$

Line (5) – straight line continued from (4)

Design Response Spectrum curves for $A_h(g)$ (=$S_a/g/((2xR)/(ZxI))$) are defined by (Note two curves, one for Dir1 and one for Dir2)

Line (1) – straight line

Eqn $A_h(g)(T)$ = Damping factor$_{user}$ x [$S_{auser}/g(0)$ + ((S_{auser} /g(T_0) – S_{auser} /g(0)) x T / T_{1user})] /((2xR)/(ZxI))

But not less than Z/2

limits $0 \leq T \leq T_{1user}$

Line (2) – straight line

Eqn $A_h(g)(T)$ = Damping factor$_{user}$ x S_{auser} /g(T_{1user}) / ((2xR)/(ZxI))

limits $T_{2user} \leq T \leq T_{3user}$

Line (3) – curve

Eqn $A_h(g)(T)$ = Damping factor$_{user}$ x S_{auser} /g(T_{1user}) x T_{2Suser} / T / ((2xR)/(ZxI))

limits $T_{2user} \leq T \leq T_{3user}$

Line (4) – curve

Eqn $S_a(g)(T) = \text{Damping factor}_{user} \times S_{auser}/g(T_{1user}) \times T_{2user} \times T_{3user} / T^2 / ((2xR)/(ZxI))$

limits $T_{3user} \leq T \leq T_{4user}$

Line (5) – straight line continued from (4)

If T_{1user} = 0, then no line (1) exists
If $T_{3user} = T_{4user}$ then no line (4) exists

Default Spectra

The following details the initial values of the parameters for spectra – this differs by head code:-

ASCE - Horizontal Design Spectrum

11.4.5 (1)	$0 \leq T \leq T_0$	$S_{DS} \times (0.4 + 0.6 \times T / T_0) \times (1/(R/I_e)$	Start point/linear
11.4.5 (2)	$T_{0 \leq T \leq Ts}$	$S_{DS}/(R/I)$	Constant
11.4.5 (3)	$T_{s \leq T \leq TL}$	$(S_{D1}/T)/(R/I)$	Constant/T
11.4.5 (4)	$T_{L \leq T \leq 4s}$	$(S_{D1} \times T_L / T^2)/(R/I)$	Constant/T^2

EC - Horizontal Design Spectrum (UK, Singapore and Europe)

3.13	$0 \leq T \leq T_B$	$a_g \times S \times [\ 2/3 + T/T_B \times (\ 2.5 / q - 2/3\ )]$	Start point/linear
3.14	$T_B \leq T \leq T_C$	$a_g \times S \times 2.5 / q$	Constant
3.15	$T_C \leq T \leq T_D$	$a_g \times S \times T_C/T \times 2.5 / q$	Constant/T
3.16	$T_D \leq T \leq 4s$	$a_g \times S \times 2.5 / q \times (T_C \times T_D) / T\text{^}2$	Constant/T^2

EC - Horizontal Design Spectrum (Malaysia)

For **Rock soil** sites

$0 \leq T \leq T_C$	$((2 \times \pi)^2 \times \gamma I \times S_{DR}(1.25) / (T_C \times T_D)) / q$	Constant
$T_C \leq T \leq T_D$	$((2 \times \pi)^2 \times \gamma I \times S_{DR}(1.25) / (T \times T_D)) / q$	Constant/T
$T_D \leq T \leq 4s$	$((2 \times \pi)^2 / T^2 \times [\lambda I \times S_{DR}(1.25) + \gamma I \times m_r \times (T - T_D))] / q$	Constant/T^2

For **Stiff Soil** sites

$0 \le T \le T_C$	$((2 \times \pi)^2 \times \gamma I \times S_{DR}(1.25) \times 1.5 / (T_C \times T_D)) / q$	Constant
$T_C \le T \le T_D$	$((2 \times \pi)^2 \times \gamma I \times S_{DR}(1.25) \times 1.5 / (T \times T_D)) / q$	Constant/T
$T_D \le T \le 4s$	$((2 \times \pi)^2 / T^2 \times [\lambda I \times S_{DR}(1.25) \times 1.5 + \gamma I \times m_r \times (T - T_D))] / q$	Constant/T^2

For **Flexible Soil** sites

$0 \le T \le T_C$	$((2 \times \pi)^2 \times \gamma I \times S_{DR}(1.5T_S) \times 3.6 / (T_C \times T_D)) / q$	Constant
$T_C \le T \le T_D$	$((2 \times \pi)^2 \times \gamma I \times S_{DR}(1.5T_S) \times 3.6 / (T \times T_D)) / q$	Constant/T
$T_D \le T \le 4s$	$((2 \times \pi)^2 / T^2 \times [\lambda I \times S_{DR}(1.5T_S) \times 3.6 + \gamma I \times m_r \times (T - T_D))] / q$	Constant/T^2

IS - Horizontal Design Spectrum

For **rocky, or hard soil** sites

$0.00 \le T \le 0.10$	damping factor x (1 + 15 T)	Start point/linear
$0.10 \le T \le 0.40$	damping factor x (2.50)	Constant
$0.40 \le T \le 4.00$	damping factor x (1.00/T)	Constant/T

For **medium soil** sites

$0.00 \le T \le 0.10$	damping factor x (1 + 15 T)	Start point/linear
$0.10 \le T \le 0.55$	damping factor x (2.50)	Constant
$0.55 \le T \le 4.00$	damping factor x (1.36/T)	Constant/T

For **soft soil** sites

$0.00 \le T \le 0.10$	damping factor x (1 + 15 T)	Start point/linear
$0.10 \le T \le 0.67$	damping factor x (2.50)	Constant
$0.67 \le T \le 4.00$	damping factor x (1.67/T)	Constant/T