Tekla Structural Designer 2018i

Reference Guides (Indian Standards)

September 2018
(6.1.06)

Table of Contents

Analysis Verification Examples

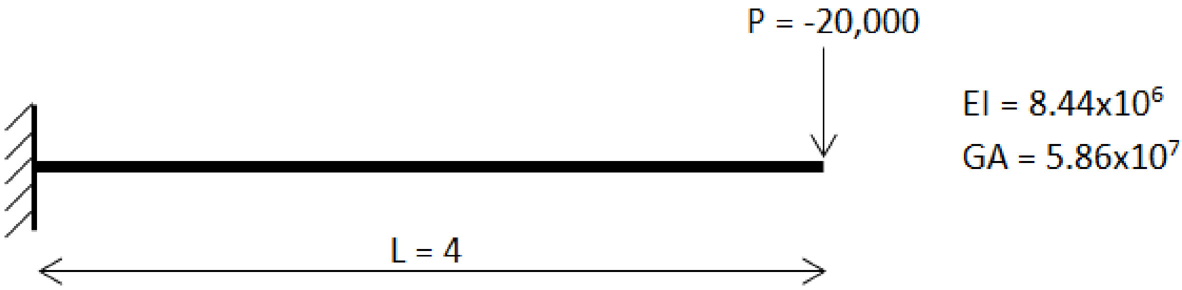
A small number of verification examples are included in this section. Our full automatic test suite for the Solver contains many hundreds of examples which are run and verified every time the Solver is enhanced.

These verification examples use SI units unless otherwise stated.

1st Order Linear - Simple Cantilever

Problem Definition

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



Assumptions

Flexural and shear deformations are included.

Key Results

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

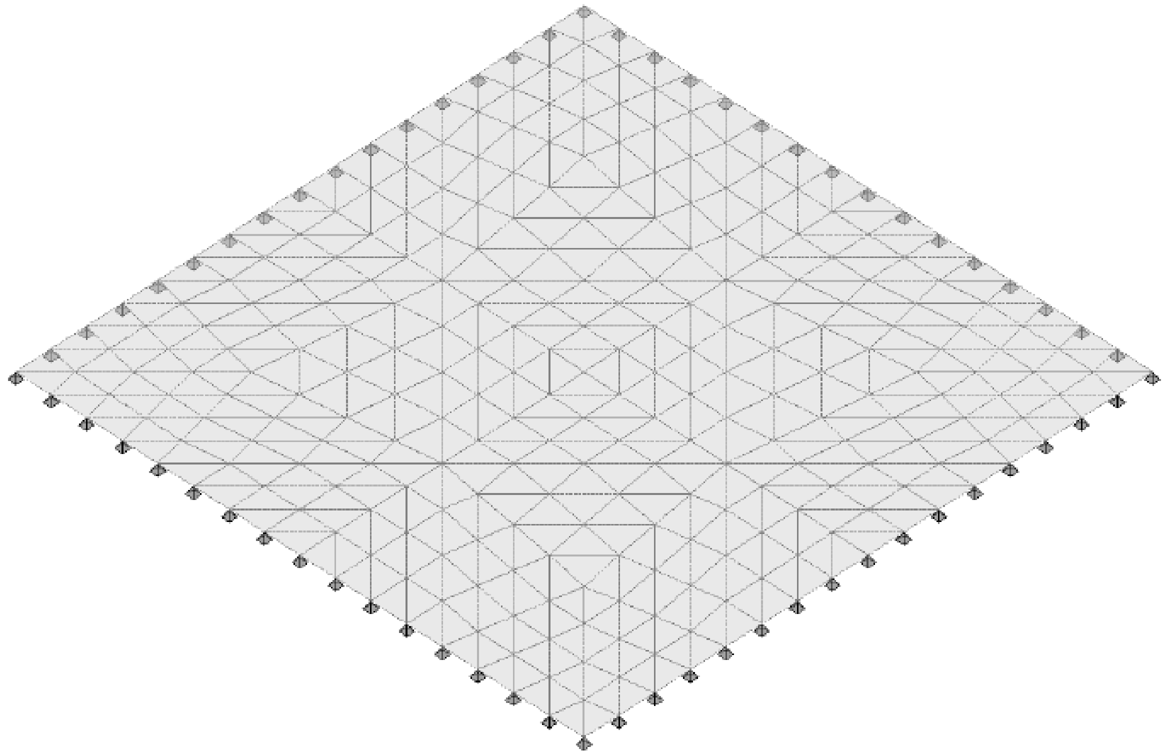
Conclusion

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

1st Order Linear - Simply Supported Square Slab

Problem Definition

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



Assumptions

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

Key Results

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

Result	Theoretical Value	Comparison 1	Solver Value	% Error

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

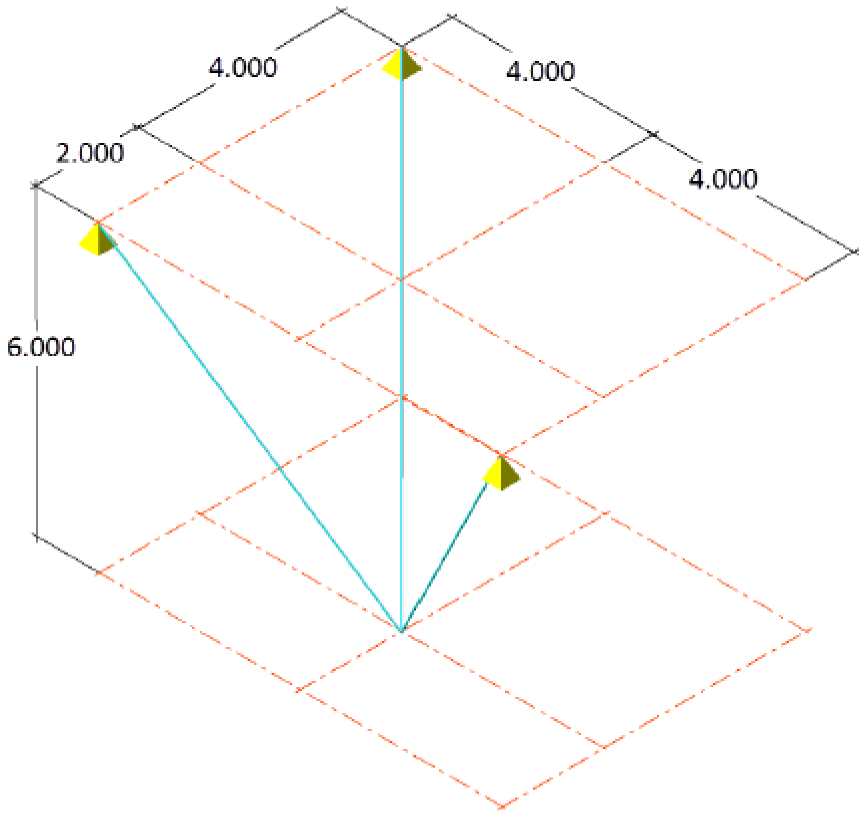
Conclusion

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

1st Order Linear - 3D truss

Problem Definition

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



Key Results

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

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

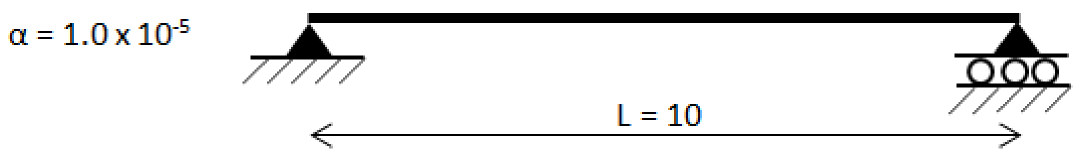
Conclusion

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

1st Order linear - Thermal Load on Simply Supported Beam

Problem Definition

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



Assumptions

The roller pin is assumed to be frictionless.

Key Results

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

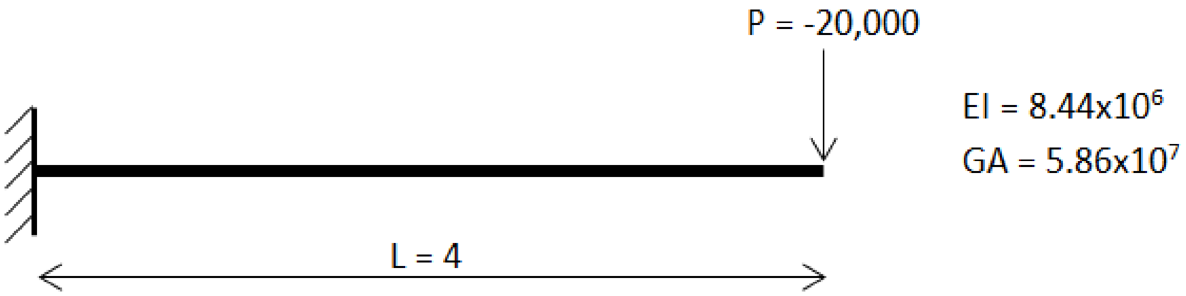
Conclusion

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

1st Order Nonlinear - Simple Cantilever

Problem Definition

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



Assumptions

Flexural and shear deformations are included.

Key Results

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

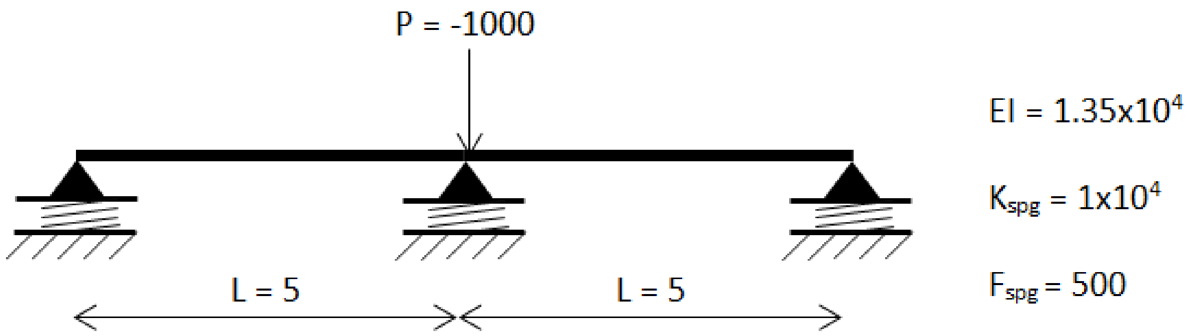
Conclusion

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

1st Order Nonlinear - Nonlinear Supports

Problem Definition

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



Assumptions

Axial and shear deformations are ignored.

Key Results

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

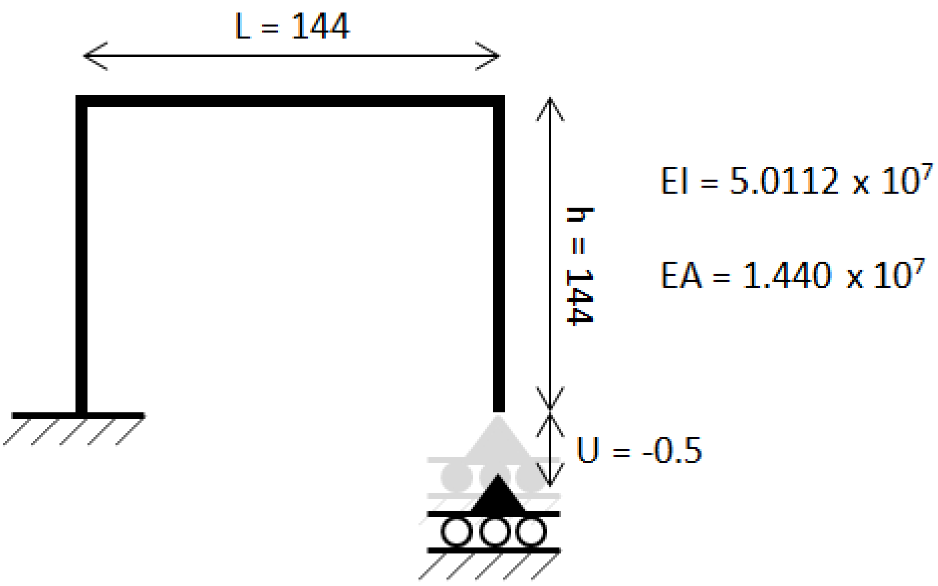
Conclusion

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

1st Order Nonlinear - Displacement Loading of a Plane Frame

Problem Definition

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



Assumptions

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

Key Results

Results were compared with two other independent analysis packages.

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

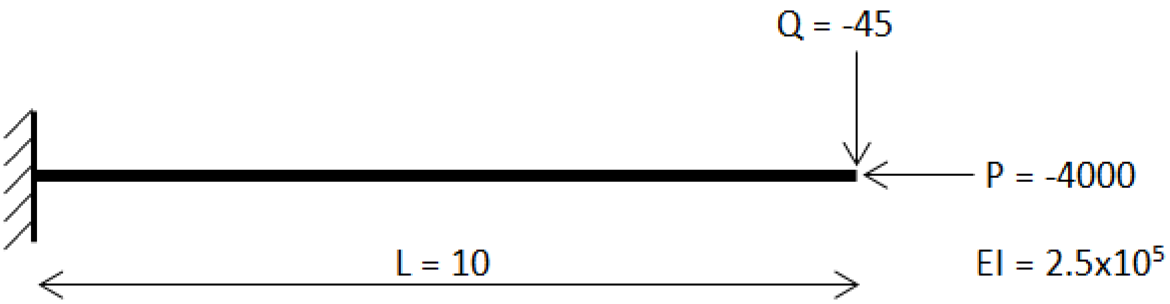
Conclusion

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

2nd Order Linear - Simple Cantilever

Problem Definition

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



Assumptions

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

Key Results

Results were compared with an independent analysis package.

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

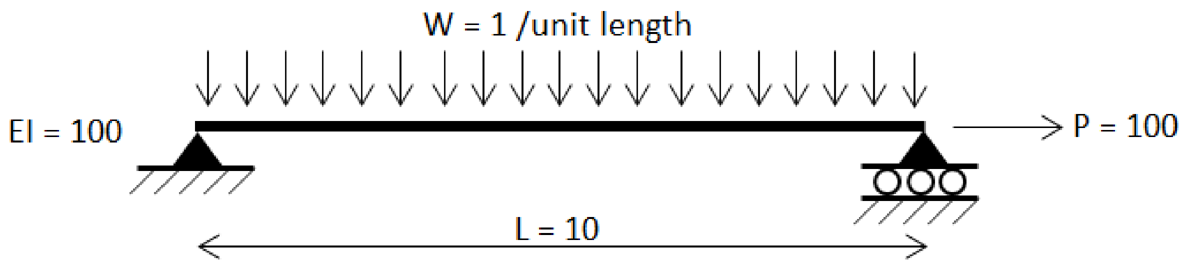
Conclusion

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

2nd Order linear - Simply Supported Beam

Problem Definition

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



Assumptions

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

Key Results

The theoretical value for deflection and moment are calculated as:

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

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

Where *U* is a variable calculated:

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

Conclusion

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

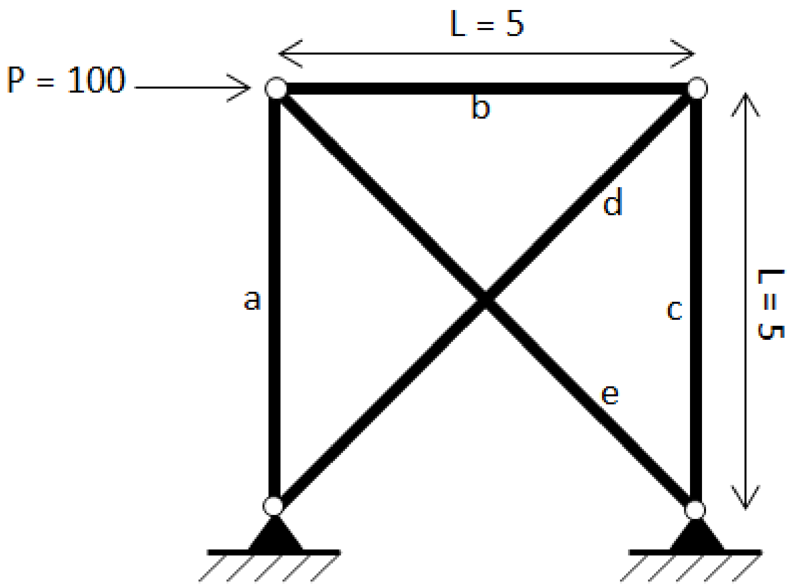
Reference

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

2nd Order Nonlinear - Tension Only Cross Brace

Problem Definition

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



Assumptions

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

Key Results

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

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

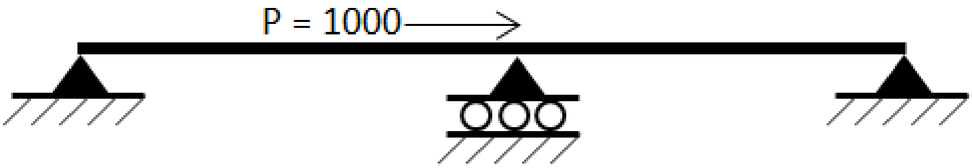
Conclusion

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

2nd Order Nonlinear - Compression Only Element

Problem Definition

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



Assumptions

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

Key Results

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

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

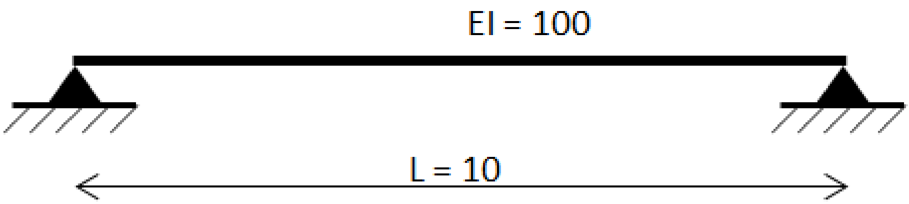
Conclusion

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

1st Order Vibration - Simply Supported Beam

Problem Definition

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



Assumptions

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

Key Results

The theoretical value for the fundamental frequency is calculated as:

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

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

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

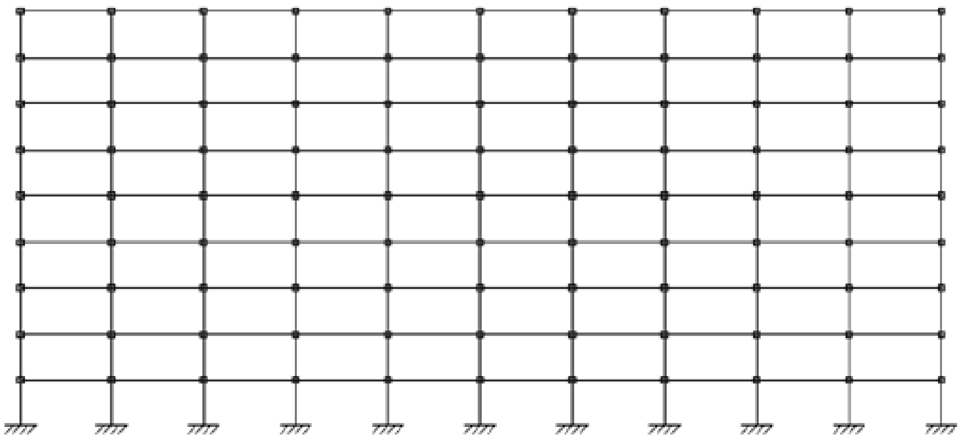
Conclusion

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

1st Order Vibration - Bathe and Wilson Eigenvalue Problem

Problem Definition

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



Assumptions

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

Key Results

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

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

Conclusion

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

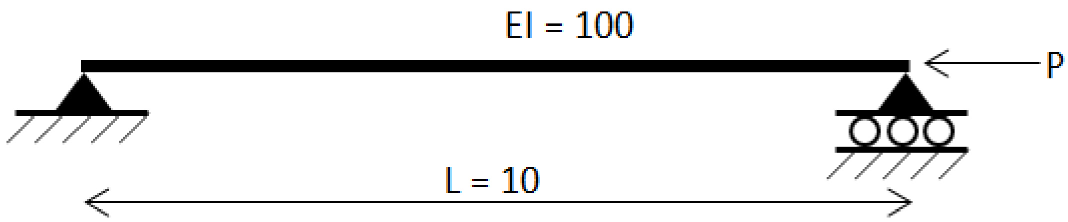
References

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

2nd Order Buckling - Euler Strut Buckling

Problem Definition

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



Assumptions

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

Key Results

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

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

With P = -1.0 the following buckling factors are obtained

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

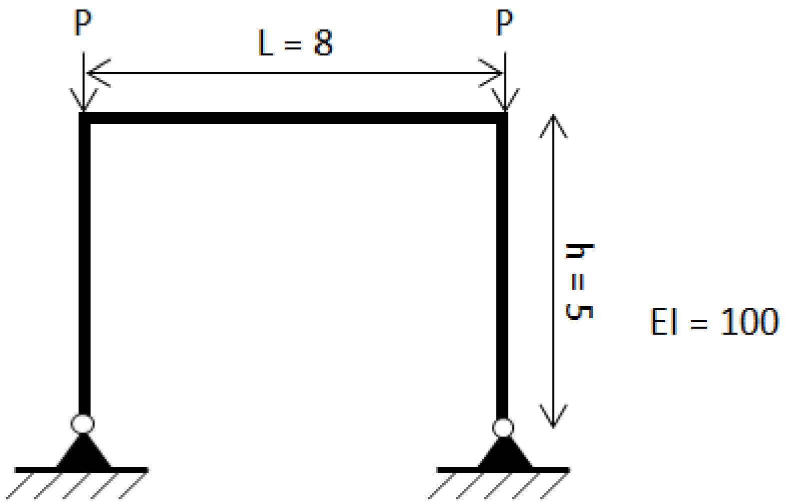
Conclusion

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

2nd Order Buckling - Plane Frame

Problem Definition

Calculate the buckling factor of the moment frame shown below.



Assumptions

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

Key Results

The theoretical buckling load is calculated by

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

where

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

Which can be solved using Newtons method and five iterations

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

Conclusion

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

References

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

Concrete Design - IS 456

Concrete Design to IS 456

This handbook describes how IS 456:2000 (Ref. 1) is applied to the design of concrete members in *Tekla Structural Designer*.

Unless explicitly noted otherwise, all clauses, figures and tables referred to are from IS 456:2000

Beam Design to IS 456

Limitations and Exclusions (Beams: IS 456)

The following general exclusions apply:

- Bundled bars.
- Design for minor axis bending and shear.
- Design for axial forces.
- Beams with a ratio of effective span/overall depth < 2.0 for simply supported beam; and effective span/overall depth < 2.5 for continuous beam are classified as deep beams and "Beyond Scope"

Materials (Beams: IS 456)

Concrete

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

Reinforcement

The reinforcement options are:

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

Slender Beams (Beams: IS 456)

The clear distance between restraints is taken as below;

For simply supported or continuous beams:[A]

Clear distance between restraints ≤ MIN [$60 * b_c$, $250 * b_c^2 / d$]

For cantilevers with lateral restraints only at support:

Clear distance between restraints ≤ MIN [$25 * b_c$ or $100 * b_c^2 / d$]

where

b_c	=	breadth of the compression face of the beam,
d	=	effective depth of beam

[A]*IS 456 : 2000 clause 23.3*

Cover to Reinforcement (Beams: IS 456)

The greatest nominal cover is selected derived from:

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

$c_{nom, \phi} \geq \phi$

where

ϕ = maximum diameter of bar in longitudinal reinforcement

The cover stated above is the cover up-to main bars and if links are present this cover is up-to the link bar.

2. Durability Requirements - The cover required to protect the reinforcement against corrosion depends on the exposure conditions. Values to be set by the user with reference to Table 16, of IS 456 : 2000 to meet the durability requirements.

3. Fire Resistance - For fire protection the values given in Table 16A of IS 456 : 2000 will ensure that fire resistance requirements are satisfied. In order to take due account of the cover required for fire resistance, a value for the nominal cover should be set by the user.

The “Durability Requirements” and “Fire Resistance” provisions are not implemented in the current version of Tekla Structural Designer.

Nominal limiting cover to reinforcement:

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

Nominal cover from top surface of beam, $c_{nom,u,top}$ Default value: 30.0mm

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

Nominal cover from side face of beam, $c_{nom,u,side}$ Default value: 30.0mm

Nominal cover from end surface of beam, $c_{nom,u,end}$ Default value: 30.0mm

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

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

$c_{nom,lim,side} = MAX[\phi_{top}, \phi_{bot}, \phi_{side}]$

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

$c_{nom,lim,top} = MAX[\phi_{top}]$

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

$c_{nom,lim,bot} = MAX[\phi_{bot}]$

Nominal cover to reinforcement from the end of a beam;

$c_{nom,lim,end} = MAX[\phi_{top}, \phi_{bot}, \phi_{side}]$

and where

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

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

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

If $c_{nom,u} < c_{nom,lim}$ a warning is displayed in the calculations.
1. IS 456 : 2000 26.4

Design Parameters for Longitudinal Bars (Beams: IS 456)

For each of these parameters, the user defined limits (specified in **Design Options > Beam > Reinforcement Settings**) are considered in addition to any IS 456 recommendations.

Minimum Distance between Bars

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

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

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

where

h_{agg} = maximum size of coarse aggregate

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

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

$s_{cl,min}$ ≥ MAX[15mm, $2h_{agg}/3$, ϕ

where

h_{agg} = nominal maximum size of coarse aggregate

ϕ = the maximum diameter of bar

Maximum Spacing of Tension Bars

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rule is applied in normal internal or external conditions of exposure. The horizontal distance between parallel reinforcement bars $s_{cl,max}$ should not be greater than the value given in Table 15 of IS 456:2000.

fy	Percentage Redistribution to or from Section Considered				
	-30	*-15*	**0**	*+15*	*+30*

	Clear Distance Between Bars				
N/mm2	*mm*	*mm*	**mm**	*mm*	*mm*
250	*215*	*260*	**300**	*300*	*300*
415	*125*	*155*	**180**	*210*	*235*
500	*105*	*130*	**150**	*175*	*195*

Redistribution is not be included in the initial release therefore only the values at 0% redistribution are currently considered from the above table. For other grades linear interpolation is applied.

Minimum Area of Reinforcement

The minimum area of **tension reinforcement**, $A_{st,min}$, is given by;[A]

$A_{st,min}$ = $(0.85*b*d)/f_y$

Where

$A_{st,min}$ = minimum area of tension reinforcement

= breadth of beam or the breadth of the web (bw) of T-beam in mm

= effective depth in mm

= characteristic strength of the reinforcement in N/mm2

[A]IS 456 : 2000 26.5.1.1

Maximum Area of Reinforcement

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

$A_{st,max}$ ≤ 0.04*b*D

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

$A_{s}'_{,max}$ ≤ 0.04*b*D

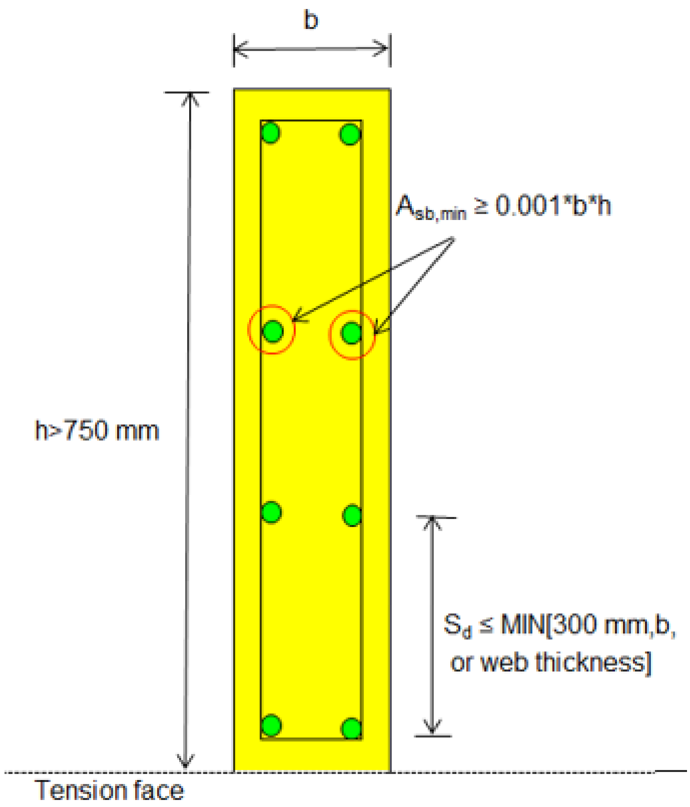
where

b	=	breadth of beam or the breadth of the web of T-beam in mm
D	=	overall depth of beam in mm

Compression reinforcement in beams shall be enclosed by stirrups for effective lateral restraint. The arrangement of stirrups shall be as specified in IS 456 : 2000 26.5.3.2

1. IS 456 : 2000 26.3.2
2. IS 456 : 2000 26.5.1.1 26.5.1.2

Side Reinforcement in Beams (Beams: IS 456)



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

Minimum total area of side face reinforcement of beam, $A_{sb,min}$

$A_{sb,min}$	=	$0.001*A_c$

Where

A_c	=	the web area of the beam
	=	b*h (For rectangular section)

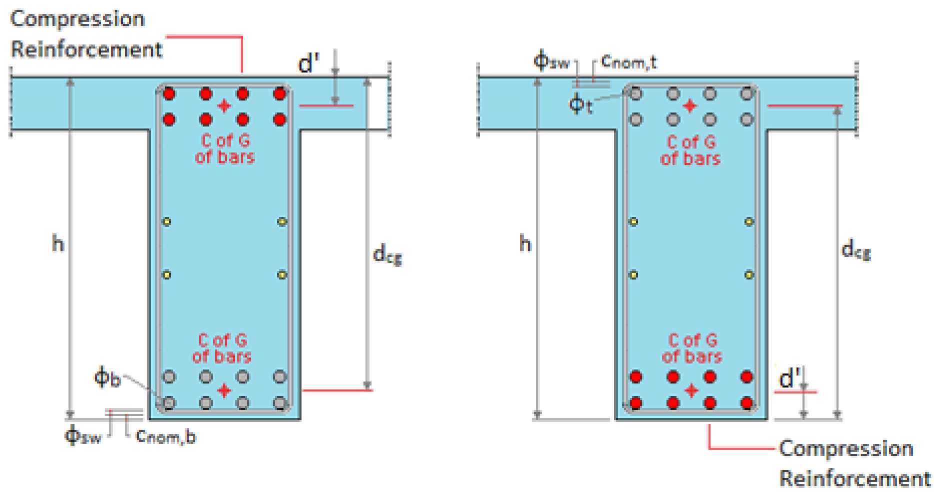
= b_v*h (For flanged section)

Asb,min shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

1. IS 456 : 2000 26.5.1.3

Effective Depth of Section (Beams: IS 456)

For the design of the longitudinal tension reinforcement, the effective depth in compression, d' is defined as the depth of compression reinforcement from highly compressed face.



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

Design for Bending (Beams: IS 456)

Redistribution of bending moments a in beams is beyond scope in the current release of Tekla Structural Designer.

Design for Bending for Rectangular Sections (Beams: IS 456)

Calculate the limiting maximum value of the neutral axis depth $x_{u,max}$ from;

$x_{u,max}$ = $(0.0035 /((0.87*f_y/E_s) + 0.0055))*d$

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

K' = (0.156)

where

$x_{u,max}$ = the limiting value of depth of the neutral axis, refer 38.1 of IS 456 : 2000

d = effective depth of the beam section

f_y = characteristic strength of reinforcement

f_{ck} = characteristic compressive strength of reinforcement

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

Depth of the neutral axis,

x_u = $0.87*f_y*A_{st}/0.36*f_{ck}*b$

Factored moment,

$M_{u,lim}$ = $0.36*b*x_{u,max}*f_{ck}*(d-0.42*x_{u,max})$

where

$M_{u,lim}$ = ultimate limiting moment capacity of the beam

IF Mu ≤ Mu,lim THEN compression reinforcement is not required.

It is desirable to design the beam as under-reinforced so that the ductility is ensured with steel stress reaching the design value;

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

A_{st} = $0.5*(f_{ck}/f_y)*[1-\surd(1-((4.6*M_u)/(f_{ck}*b*d^2)))]*b*d$

where

b = width of the beam

M_u = total factored bending moment acting on the beam

IF Mu > Mu,lim THEN compression reinforcement is required.

Design for Bending for Flanged Sections (Beams: IS 456)

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

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

where

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

In case of a flanged section, following different cases arise depending on the depth of flange D_f in relation to the depth of neutral axis x_u and in relation to the rectangular part $3x_u/7$ of rectangular-parabolic stress distribution;

Case-1: Neutral axis lying inside the flange i.e. $x_u \leq D_f$

Case-2: Neutral axis lying outside the flange or inside the web i.e. $x_u > D_f$

This case is further subdivided into two subparts depending on whether the rectangular part of the stress block ($3*x_u/7$) is less than D_f or greater than D_f as given below:

Case-2a: Rectangular part of the stress block less than D_f i.e. $3*x_u/7 < D_f$ or $x_u<7*D_f/3$

Case-2b: Rectangular part of the stress block greater than D_f i.e. $3*x_u/7 > D_f$ or $x_u>7*D_f/3$

IF Mu ≤ Mu,lim THEN compression reinforcement is not required.

For Case-1, since the concrete below the neutral axis is assumed to be cracked, the flanged beam can be considered as a rectangular beam of width $b = b_f$

For Case-2, for convenience of calculation the stress block divided into two parts, one consists of concrete in web portion of width b_w and depth x_u, and other consists of projecting flanges of width (b_f-b_w) and depth D_f

IF Mu > Mu,lim THEN compression reinforcement is required.

Design for Shear (Beams: IS 456)

Design Shear Resistance (Beams: BS 8110)

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

$V_{Rd,max}$ = $\tau_{c,max}*b*d$

where

$\tau_{c,max}$ = maximum shear stress in concrete from Table 20 of IS 456 : 2000

$V_{Ed,max}$ ≤ $V_{Rd,max}$

where

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

THEN the shear design process can proceed.

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

The design value of the shear resistance of a concrete section with no shear reinforcement, $VRd,_c$ is given by;

$V_{Rd,c}$ = τ_c*b*d

IF

$V_{Ed,max}$ < 0.5* $V_{Rd,c}$

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

ELSE IF

0.5* $V_{Rd,c}$ < $V_{Ed,max} \leq (V_{Rd,c} + (0.4*b*d))$

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

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

where

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

f_{yv} = characteristic strength of links ($f_{yv} \leq 415$ N/mm^2)

s_v = spacing of the links along the member

ELSE IF

$(V_{Rd,c} + (0.4*b*d)) < V_{Ed,max} \leq V_{Rd,max}$

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

$(A_{sv}/ s_v)_{design,shear}$ $\geq (V_{Ed,max} - V_{Rd,c}) / (0.87\ f_{yv}*d)$

Minimum Area of Shear Reinforcement (Beams: IS 456)

Minimum shear reinforcement shall be provided when, 0.5* $V_{Rd,c} < V_{Ed,max} < (V_{Rd,c} + (0.4*b*d))$;

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

where

A_{sv} = total minimum area of stirrup legs effective in shear

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

Note: As per IS 456:2000 26.5.1.6 where the maximum shear stress calculated is less than half the permissible value ($V_{Ed,max}<0.5*V_{Rd,c}$) this provision need not be complied with.

Spacing of Shear Reinforcement (Beams: IS 456)

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

$s_{v,min,u} \le s_v \le MIN(0.75*d, s_{v,max,u})$

where

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

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

Design for Torsion (Beams: IS 456)

The design shear force at a cross-section is given by;

$V_{Ed,max} = \tau_v*b*d$

where

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

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

$V_{Rd,c} = \tau_c*b*d$

where

$V_{Rd,c}$ = the design value of shear resistance of a concrete section with no shear reinforcement

Maximum combined shear strength (shear + torsion) is given by;

$V_{Rd,max} = \tau_{c,max}*b*d$

Check IF

$V_e \leq V_{Rd,max}$

where

V_e = the equivalent shear calculated

THEN the torsion design process can proceed.

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

IF

$V_e < 0.5*V_{Rd,c}$

THEN provide nominal shear reinforcement over the full length of the span.

ELSE IF

$0.5*V_{Rd,c} \leq V_e < (V_{Rd,c} + (0.4*b*d))$

THEN provide **minimum shear reinforcement** over the full length of the span as specified and **No torsion reinforcement**.

ELSE provide both longitudinal and transverse reinforcement.

Deflection Check (Beams: BS 8110)

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

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

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

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

For beams with **tension reinforcement**, the basic L/d value is multiplied by the modification factor, MF_t[B];

MF_t = $1/[0.225+(0.00322*f_s)-(0.625*\log_{10}*((b*d)/(100*A_{st})))]$

In case of tension reinforcement maximum value of MF_t is 2

where

f_s = the design service stress in the tension reinforcement

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

For beams with **compression reinforcement**, the basic L/d value is multiplied by the modification factor, MF_c;[C]

MF_c = $(1.6*p_c)/(p_c+0.275)$

where

p_c = $(100*A_{sc})/(b*d)$

[A] *IS 456 : 2000 23.2.1(e)*
[B] *SP 24-1983 (IS 456 : 1978) page 55*
[C] *SP 24-1983 (IS 456 : 1978) page 55*

Column Design to IS 456

Limitations and Exclusions (Columns: IS 456)

The following general exclusions also apply:

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

Materials (Columns: IS 456)

Concrete

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

Reinforcement

The reinforcement options are:

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

Cover to Reinforcement (Columns: IS 456)

The nominal cover is selected derived from greatest of the following:

1. Bar Size

The nominal cover to all steel should be such that the resulting cover to main bar should not be less than the size of the main bar.

where

$\varphi_{largest}$ = maximum diameter of bar in longitudinal reinforcement

IF

$\varphi_{largest} \leq 12$ mm and b ≤ 200 mm

$c_{nom,lim} \geq 25$ mm

2. Nominal maximum size of aggregate

The nominal cover should be not less than the nominal maximum size of the aggregate h_{agg} (specified in **Design Options > Column > General Parameters**).

3. A minimum value for the nominal cover

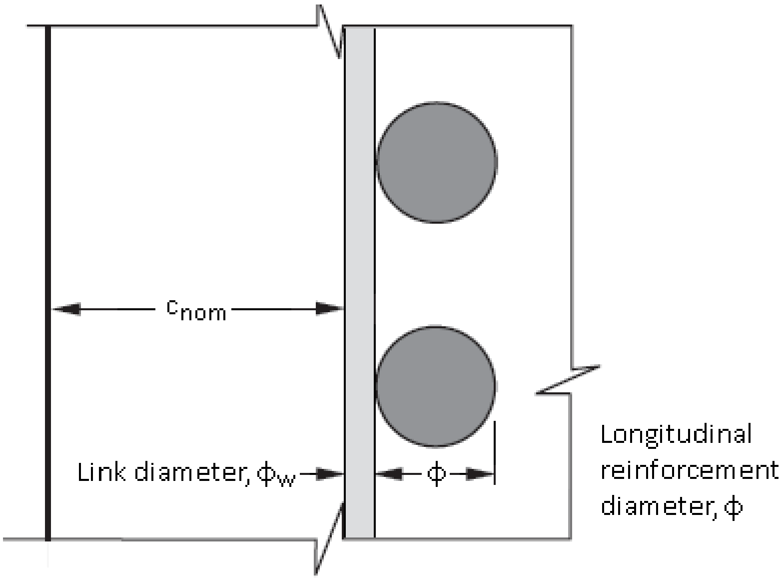
The minimum value for the nominal cover, $c_{nom,u}$, for each column is specified in the column properties.

Nominal limiting cover to reinforcement:

$c_{nom,lim}$ = MAX [$\varphi_{largest}$, h_{agg}, $c_{nom,u}$]

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

Nominal cover is the design depth of concrete cover to all steel reinforcement, including links.If links are present cover is up-to link bar. Ref: IS 456 : 2000 26.4.1.



Design Parameters for Longitudinal Bars (Columns: IS 456)

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

Minimum Longitudinal Bar Spacing

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

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

where

h_{agg} = maximum size of coarse aggregate

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

ϕ = the maximum diameter of bar

Minimum Longitudinal Total Steel Area

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

= 0.8% * column area

Maximum Longitudinal Total Steel Area

$A_{sl,max}$ = 6% * column area

1. *IS 456:2000 26.3.2*

Ultimate Axial Load Limit (Columns: IS 456)

The ultimate axial load carrying capacity P_u, for a braced or unbraced short column is obtained as below:

P_u = $0.4 * f_{ck} * A_c + 0.67 * A_{sc} * f_y$

Where

P_u = ultimate axial load on the member

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

f_{ck} = characteristic strength of concrete

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

f_y = characteristic strength of compression reinforcement

Effective Length Calculations (Columns: IS 456)

Clear Height

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

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

Effective Length

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

l_e = K * l

where

l = clear height between end restraints

K = effective length factor

The value of K may is obtained from the following equations[AA]:

a) For non sway (braced frames):

K = $[(1 + 0.145*(\beta_1 + \beta_2) - 0.265*\beta_1*\beta_2] / (2 - 0.364*(\beta_1 + \beta_2) - 0.247*\beta_1*\beta_2]$

b) Sway frames (Moment resisting frames):

K = $MIN(1.2,\{[1 - 0.2*(\beta_1 + \beta_2) - 0.12*\beta_1*\beta_2] / [1 - 0.8(\beta_1 + \beta_2) + 0.6\ \beta_1*\beta_2]\}^{0.5}$

where

β is the rotation release factor and has two values, β_1 and β_2 at the two ends (top and bottom of the columns respectively)

β_1 , β_2 = $\Sigma K_c/(\Sigma K_c+\Sigma K_b)$

ΣK_c = effective flexural stiffness of the columns

ΣK_b = effective flexural stiffness of the beams

[A] *Refer IS 800 : 2007 Annex D and Shah & Karve text book Sect. 11.2*

Determination of the non-sway or sway column[B]:

If Q ≤ 0.04

If Q > 0.04

where

Q = Stability index

[A]
[B] *Refer IS 456 : 2000 Annex E*

Column Stack Classification (Columns: IS 456)

Slenderness ratio

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

λ_x = l_{ex} / D

λ_y = l_{ey} / b

where

λ_x = slenderness ratio about major axis (x axis)

λ_y = slenderness ratio about minor axis (y axis)

D = larger dimension of the column

b = smaller dimension of the column

l_{ex} = effective length in respect of major axis (y axis)

l_{ey} = effective length in respect of minor axis (z axis)

IF λ_x and $\lambda_y < 12$

Column is considered as short

Else

Column is considered as slender

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

limiting slenderness ratio

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

For braced or unbraced columns[A]

l /b ≤ 60

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

l_0 /b ≤ 100*b / D]

Where

l = clear height between end restraints

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

[A] *IS 456 : 2000 25.3.1*
[B] *IS 456 : 2000 25.3.2*

Design Moment Calculations (Columns: IS 456)

Minimum Eccentricity

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

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

$e_{min,x}$ = MAX [l_{ux}/500 + D/30, 20mm]

$e_{min,y}$ = MAX [l_{uy}/500 + d/30, 20mm]

Where

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

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

l_{ux} = unsupported length in respect of major axis (xx-axis)

l_{uy} = unsupported length in respect of major axis (yy-axis)

[A] *IS 456 : 2000 25.4*

Short columns

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

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

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

Slender columns

Additional Moments for Slender Columns:

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

The additional moment are obtained as below:[A]

M_{ax} = kx*[((Pu*D)/2000)*(lex/D)2]

M_{ay} = $k_y*[((P_u*b)/2000)*(l_{ey}/b)^2]$

Where

M_{ax} = additional moment about major axis (x axis) as per Cl 39.7 of IS 456 : 2000

M_{ay} = additional moment about major axis (yaxis) as per Cl 39.7 of IS 456 : 2000

kx or ky = reduction factor

[A] *IS 456 : 2000 39.7.1*

Design for Combined Axial and Bending (Columns: IS 456)

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

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

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

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

where

M_{major} = Moment about the major axis

M_{minor} = Moment about the minor axis

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

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

Design for Shear (Columns: IS 456)

Design Parameters for Shear Design

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

Maximum Span Region Shear Link Spacing

$\varphi_{w,max}$ = MIN[[least lateral dimension of the column, 16 * smallest longitudinal bar diameter, user defined maximum link spacing]

Design of Column Shear Reinforcement

Column shear reinforcement design is described as below:

A) Calculate Design Shear Stress

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

$\tau_{v,i} = V_{u,i} / A_{v,i}$

Where

$V_{u,i}$ = highest shear force in the stack in this direction,

$A_{v,i}$ = shear area parallel to the axis *i*.

B) Check Maximum Shear Capacity

The maximum allowable shear stress (τ_{max}) on the section is given by,[A]

Grade of concrete	M20	M25	M30	M35	M40 and above
τ_{max} N/mm2	2.8	3.1	3.5	3.7	4.0

IF

$\tau_{v,i}$ ≤ τ_{max} (for each direction)

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

ELSE the section size must be increased.

[A] *IS 456 : 2000 Table 20*

C) Check Section Nominal Shear Capacity

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

D) Nominal Shear Capacity Not Adequate

IF

$0.5*\tau_c' \leq \tau_v \leq (\tau_c' + 0.4)$

Then provide minimum shear reinforcement as specified below,

$(A_{sv} / s_v)_{min,shear}$ ≥ $(0.4* b / (0.87*f_{yv})$

Where

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

f_{yv} = characteristic strength of links (not to be taken more than 415 N/mm^2)

s_{sv} = spacing of the links along the length of the member

Else

Provide design reinforcement as,

$(A_{sv} / s_v)_{design,shear} \geq V_{Ed,design} / (0.87 * f_{yv} * d_i)$

Where

$V_{Ed,design} = (\tau_v * b * d_i) - (\tau_c * b * d_i)$

$= (V_{Ed,max} - V_{Rd,c})$

When, $(\tau_c' + 0.4) \leq \tau_v \leq \tau_{c,max}$

The spacing s is given by[A]

$(s_{min,u} \leq s \leq MIN[0.75 * d_i, 300mm, s_{v,max,u}])$

[A] *IS 456 : 2000 26.5.1.5*

Wall Design to IS 456

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

Limitations and Exclusions (Walls: IS 456)

The following general exclusions apply:

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

Materials (Walls: IS 456)

Concrete

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

Reinforcement

The reinforcement options are:

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

Cover to Reinforcement (Walls: IS 456)

The nominal cover is selected derived from greatest of the following:

1. Bar Size

The nominal cover to all steel should be such that the resulting cover to main bar including links should not be less than the size of the main bar.

where

φ = maximum diameter of main bar

2. Nominal maximum size of aggregate

The nominal cover should be not less than the nominal maximum size of the aggregate h_{agg} (specified in **Design Options > Wall > General Parameters**).

3. A minimum value for the nominal cover

The minimum value for the nominal cover, $c_{nom,u}$, for each wall is specified in the wall properties.

Nominal limiting cover to reinforcement:

$c_{nom,lim}$ = MAX [φ, h_{agg}, $c_{nom,u}$]

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

Vertical reinforcement (Walls: IS 456)

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

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

Plain Wall Check

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

a) Check for stresses at the corner of a wall

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

b) Check for bending utilization

Check the bending utilization is less than 10%, i.e.

$A_{sc, required}$ = 0.1*0.0012* A_c = 0.00012* A_c (For $\varphi \leq 16$mm $f_y \geq 415$ N/mm^2)

$A_{sc, required}$ = 0.1*0.0015* A_c = 0.00015* A_c (For other types of bar)

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

c) Check for limiting axial load

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

$p_{uw} \leq 0.3*(t - 1.2*e_x - 2e_a)*f_{ck}$[1]

Where

t = thickness of wall

f_{ck} = characteristic strength of concrete

e_x = eccentricity of load at right angles to the plane of the wall (minimum resultant eccentricity of load at right angles to the plane of the wall = 0.05*t

e_a = additional eccentricity due to slenderness effect

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

Reinforcement area for a Plain wall

Following parameters control the vertical reinforcement area of plain wall,

Limiting minimum ratio of vertical reinforcement area to gross concrete area[2], $\rho_{v,lim,min}$

For reinforcement in one grid (for single layer reinforcement)

For $\varphi \leq 16$ mm and $f_y \geq 415$ N/mm^2

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,lim,min}*A_c = 0.0012*A_c$

For other types of bars

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,lim,min}*A_c = 0.0015*A_c$

For reinforcement in two grids, one near each face (for double layers reinforcement)

For $\varphi \leq 16$ mm and $f_y \geq 415$ N/mm^2

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,lim,min}*A_c = (2*0.0012)*A_c$

For other types of bars

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,lim,min}*A_c = (2*0.0015)*A_c$

For walls having thickness more than 200 mm, the vertical and horizontal reinforcement shall be provided in two grids.

Reinforcement area for a RC wall

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

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

Total minimum area of vertical reinforcement, $A_{sc,min} = \rho_{v,lim,min}*A_c = 0.004*A_c$

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

Where
A_c = Gross area of the concrete wall
p_u = The total design axial load on the wall due to design ultimate loads

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

To avoid practical difficulties while placing the bars, SP : 24-1983 suggests a maximum vertical steel of 2 percent of the cross-section.

Spacing of vertical loose bars

Limiting minimum clear horizontal spacing of the vertical bars, $s_{cl,v,min}$ is given by, [3]

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

Where
h_{agg} = maximum size of coarse aggregate
$s_{cl,u,v,min}$ = user specified minimum clear horizontal distance between bars
$s_{cl,v,min}$ = minimum clear horizontal distance between bars

IF

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

THEN

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

Where
ϕ = maximum diameter of vertical bar

Limiting maximum spacing of vertical bars in the wall, $s_{cl,v,max}$ = MIN [3*t,450 mm][4]

As per SP 34-1987, cl 11.2.4.1, vertical bars that are not fully restrained should not be placed further than 200 mm from a restrained bar.

Check for vertical reinforcement when wall in tension

For RC wall –

For wall thickness, t > 200 mm

The user has to put reinforcement in two grids / 'double' layers, one near each face.

It is NOT necessary to check the axial load acting on the wall is tension or compression.

For wall thickness, t < 200 mm

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

1. IS 456 : 2000 cl 32.2.5

2. IS 456 : 2000 cl 32.5

3. IS 456 : 2000 cl 26.3.2

4. IS 456 : 2000 cl 32.5

Horizontal reinforcement (Walls: IS 456)

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

Reinforcement area for Plain or RC wall

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

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

For reinforcement in one grid (for single layer reinforcement)

For $\varphi \leq 16$ mm and $f_y \geq 415$ N/mm^2

Total minimum area of horizontal reinforcement, $A_{sh,min} = \rho_{h,lim,min} * A_c = 0.0020 * A_c$

For other types of bars

Total minimum area of horizontal reinforcement, $A_{sh,min} = \rho_{h,lim,min} * A_c = 0.0025 * A_c$

For reinforcement in two grids, one near each face (for double layers reinforcement)

For $\varphi \leq 16$ mm and $f_y \geq 415$ N/mm^2

Total minimum area of horizontal reinforcement, $A_{sh,min} = \rho_{h,lim,min}*A_c = (2*0.0020)*A_c$

For other types of bars

Total minimum area of horizontal reinforcement, $A_{sh,min} = \rho_{h,lim,min}*A_c = (2*0.0025)*A_c$

Where

A_c= Gross area of the concrete wall

IF

$A_{sc} > 0.01*A_c$

THEN

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

Where
A_{sc}= Vertical reinforcement

Total maximum area of horizontal reinforcement, $A_{sh,max} = \rho_{h,lim,max}*A_c = 0.04*A_c$

Diameter of horizontal bar

Minimum diameter of horizontal bar, $\phi_{h,min}$ = MAX [$0.25*\phi_v$, $\phi_{h,min,u}$, 6 mm][1]

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

Spacing of horizontal loose bars

Limiting minimum clear vertical spacing of the horizontal bars, $s_{cl,h,min}$ is given by, [2]

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

Where
h_{agg} = maximum size of coarse aggregate
$s_{cl,u,h,min}$ = user specified minimum clear vertical distance between bars
$s_{cl,h,min}$ = minimum clear vertical distance between bars

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

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

Where
ϕ = diameter of horizontal bar

Limiting maximum horizontal spacing, $s_{cl,h,max}$ = MIN [3*t , 450mm]

Reinforcement area for a Plain wall

For a plain wall,

$0 \le \rho_v < 0.004$

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

1. IS 456 : 2000 cl 26.5.3.2

2. IS 456 : 2000 cl 26.3.2

Link/Confinement Reinforcement (Walls: IS 456)

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

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

Reinforcement area

There is no code requirement to satisfy.

Diameter of confinement reinforcement

Minimum diameter of link, $\phi_{w,min}$ = MAX [$0.25*\phi_v$, 6 mm , $\phi_{w,min,u}$][1]

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

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

Spacing of confinement reinforcement

The spacing of links, s_v in horizontal direction is given by;[2]

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

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

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

$s_{v,min,u} \le s_v \le$ MIN [$15*\phi_v$, $s_{v,max,u}$,300 mm]

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

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

1. IS 456 : 2000 cl 26.5.3.2 4c

2. SP 34-1987 cl 11.2.4.1

Ultimate Axial Load Limit (Walls: IS 456)

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

Effective Length and Slenderness Calculations (Walls: IS 456)

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

Design Moment Calculations (Walls: IS 456)

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

Design for Combined Axial and Bending (Walls: IS 456)

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

Design for Shear (Walls: IS 456)

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

Chapter

Slab Design to IS 456

Limitations and Exclusions (Slabs: IS 456)

The following general exclusions apply:

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

- Lightweight concrete

Materials (Slabs: IS 456)

Concrete

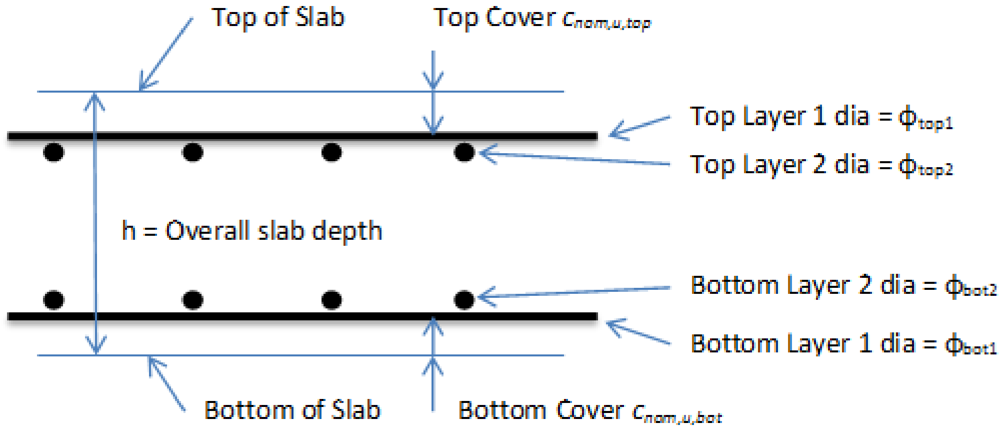
Only normal weight is included in the current release. (Lightweight concrete is excluded).

Reinforcement

The reinforcement options are:

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

Reinforcement Parameters (Slabs: IS 456)



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

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

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

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

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

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

h = overall slab depth

b = unit width

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

Cover to Reinforcement (Slabs: IS 456)

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

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

This value is then checked against the nominal limiting cover, $c_{nom,lim}$ which depends on the diameter of the reinforcement.

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

Limiting Reinforcement Parameters (Slabs: IS 456)

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

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

Minimum and Maximum Loose Bar Diameter (Slabs: IS 456)

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

Minimum Loose Bar Diameter

For “flat slab”:

φ_{min} = 8mm (default)

For “beam and slab”:

φ_{min} = 8mm (default)

Maximum Loose Bar Diameter

For “flat slab”:

φ_{min} = 16mm (default)

For “beam and slab”:

φ_{min} = 16mm (default)

Minimum Clear Spacing (Slabs: IS 456)

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

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

where

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

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

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

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

If

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

Then

$s_{cl,min} > \varphi$

Where

φ = maximum bar diameter

Basic Cross Section Design (Slabs: IS 456)

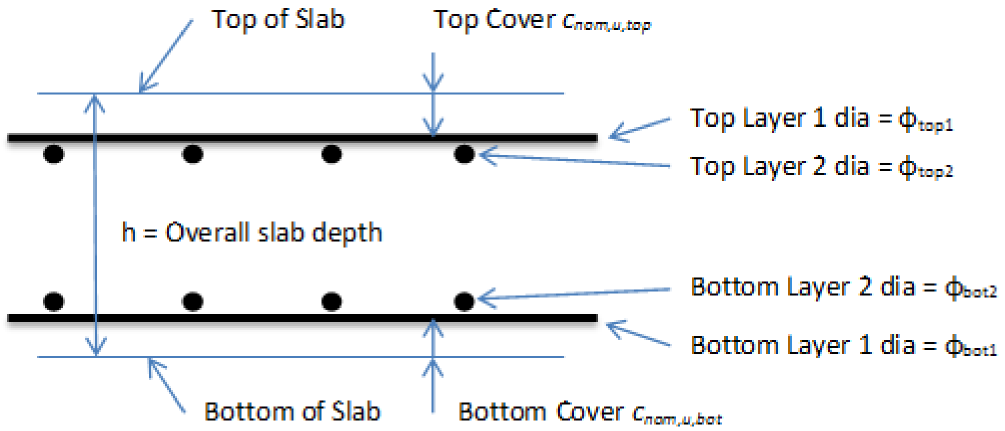
Regardless of whether design is being carried out for a slab panel or a patch, a unit width of slab is always designed for a known design force.

h = overall slab depth

b = unit width

The unit width of slab is 1m, and so the design cross section is a rectangular section where b = 1000mm

Matching Design Moments to Reinforcement Layers (Slabs: IS 456)



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

- Mdx-top - used to determine the reinforcement requirement of the x-direction bars in the top of the slab.
- Mdy-top - used to determine the reinforcement requirement of the y-direction bars in the top of the slab
- Mdx-bot - is used to determine the reinforcement requirement of the x-direction bars in the bottom of the slab.
- Mdy-bot - is used to determine the reinforcement requirement of the y-direction bars in the bottom of the slab.

For each set of design bending moments, the effective depths d and d_2 are established - taking account of the direction of the outer bar layer (as specified in the Reinforcement properties section of the slab panel properties).

Design for Bending (Slabs: IS 456)

For the design moment under consideration the appropriate effective depths d and d_2 are calculated. The design is then basically the same as employed for the design of rectangular beams.
see: Design for Bending *(Beams: IS 456)*

Deflection Check (Slabs: IS 456)

The span-effective depth check only applies to “Beam and Slab” panels. The basic principle is the same as used for beams.
see: Deflection Check *(Beams: IS 456)*

Chapter

References

1. **Bureau of Indian Standards**. *IS 456 : 2000. Plain and Reinforced Concrete - Code of Practice.* **BIS 2000**.