Tekla Structural Designer 2021

Design Codes Reference: US codes

May 2021

Contents

1 US codes

1.1 Loading (ASCE7)

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

The following topics are covered:

Loadcases (ASCE7)

Loadcase types (ASCE7)

The following loadcase types can be created:

Type	Calculated automatically	Include in the combination generator	Live load reductions	Pattern load
self weight (beams, columns and walls)	yes/no	yes/no	N/A	N/A
slab wet	yes/no	N/A	N/A	N/A
slab dry	yes/no	yes/no	N/A	N/A
dead	N/A	yes/no	N/A	N/A
live	N/A	yes/no	yes/no	yes/no
roof live	N/A	yes/no	yes/no	N/A
wind	N/A	yes/no	N/A	N/A
snow	N/A	yes/no	N/A	N/A
snow drift	N/A	yes/no	N/A	N/A
temperature	N/A	N/A	N/A	N/A
settlement	N/A	N/A	N/A	N/A
seismic	N/A	yes	N/A	N/A

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

Self weight (ASCE7)

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

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

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

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

Live and roof live loads (ASCE7)

Live load reductions

Reductions can be applied to roof live and live loads to take account of the unlikelihood of the whole building being loaded with its full design live load. The reduction is calculated based on total floor area supported by the design member. Roof live and live load types each have their own reductions applied in accordance with either Section 4.8 and 4.9 of ASCE 7-05, or Section 4.7 and 4.8 of ASCE 7-10, -16 as appropriate.

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

- Horizontal steel beams with vertical webs (major axis horizontal) which are set to be “gravity only” pin ended only
- Columns of any material
- Concrete walls, mid-pier or meshed

Live load reduction factor

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

R	=	$(0.25 + 15 / Sqrt(K_{LL}*A_T))$ - where R <= 1.0	US-units
R	=	$(0.25 + 4.57 / Sqrt(K_{LL}*A_T))$	metric-units

K_{LL} comes from Table 4-2 in ASCE7-05/ASCE7-10, or Table 4.7-1 in ASCE7-16. Essentially:

Interior and exterior cols (no cantilever slabs)	K_{LL}= 4
Edge and interior beams (no cantilever slabs)	K_{LL}= 2
Interior beams (with cantilever slabs)	K_{LL}= 2
Cantilever beams	K_{LL}= 1
Edge cols (with cantilever slabs)	K_{LL}= 3
Corner cols (with cantilever slabs)	K_{LL}= 2
Edge beams (with cantilever slabs)	K_{LL}= 1
For all beams and column stacks supporting one floor	$R \geq 0.5$
For all column stacks supporting two or more floors	$R \geq 0.4$

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

Roof live load reduction factor

The roof live load reduction factor is calculated as follows:

R	=	$R_1 * R_2$	
where			
R_1	=	$1.2 - 0.001 * A_T$, where $1.0 \geq R_1 \geq 0.6$	US-units
	=	$1.2 - 0.011 * A_T$	metric-units
R_2	=	1.0 (conservatively assumes roofs < 18 degs)	

Wind loads (ASCE7)

The ASCE7 Wind wizard...

NOTE The **Wind Wizard** is fully described in the Wind Modeling Engineer’s Handbook.

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

- Directional Procedure Part 1 - Rigid Buildings of All Heights (Chapter 27)

- Envelope Procedure Part 1 - Low-Rise Buildings (Chapter 28)

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

In order to run the **Wind Wizard** the following assumptions/limitations exist:

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

Simple wind loading

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

Patterning of live loads (ASCE7)

ASCE7 pattern loading for LRFD combinations is as follows:

Code class	Load combination	Loaded spans	Unloaded spans
LRFD	1.2D + 1.6L + 0.5Lr	1.2D + 1.6L + 0.5Lr	1.2D + 0.5Lr

Combinations (ASCE7)

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

Application of notional loads in combinations (ASCE7)

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

- NL Dir1+
- NL Dir1-
- NL Dir2+

- NL Dir2-

When you run the the Combinations Generator you are required to select the NL directions to add, and the factors to be applied as part of the process. Alternatively, you can set up the combinations manually and apply notional loads and factors to each as required.

Combination generator (ASCE7)

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

NOTE Temperature and settlement loadcase types are not included in the **Generate...** command - these need to be added manually.

Combination generator - Combinations

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

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

Combination generator - Service

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

Combination generator - NL

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

Any combination with wind in is automatically greyed.

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

Combination classes (ASCE7)

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

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

Then (where applicable) you indicate whether they are to be checked for strength or service conditions, or both. You also have the option to make any of the combinations inactive.

Construction stage combination (ASCE7)

A Construction Stage load combination is only required for the purpose of designing any composite beams within the model. It is distinguished from other combinations by setting its "Class" to Construction Stage. Typically this combination would include a loadcase of type "Slab Wet", (not "Slab Dry"), other loadcases being included in the combination as required.

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

Gravity combination (ASCE7)

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

They are used in the Gravity Sizing processes as follows:

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

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

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

Lateral combinations (ASCE7)

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

They are used during the Full Design processes as follows:

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

Seismic combinations (ACSE7)

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

Modal mass combinations (ASCE7)

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

NOTE It is always assumed that all loads in the loadcases in the combination are converted to mass for modal analysis. You are permitted to add lumped mass directly to the model.

1.2 Steel design to AISC 360 ASD and LRFD

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

Click the links below to find out more:

- General (page 14)
- Steel beam design (page 16)
- Composite beam design (page 27)
- Steel column design (page 35)
- Column base plate design (page 39)
- Steel brace design (page 45)
- Truss member design (page 47)
- Steel single, double angle and tee section design (page 48)

General

Click the links below to find out more:

- Seismic design (AISC 360) (page 15)
- Deflection checks (AISC 360) (page 15)
- Steel grade (AISC 360) (page 15)

Seismic design (AISC 360)

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

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05) (Ref. 9), (AISC 341-10) (Ref. 10) or (AISC 341-16) (Ref. 11). These additional design rules ONLY apply to members in Seismic Load Resisting Systems. These rules are applied as follows:

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

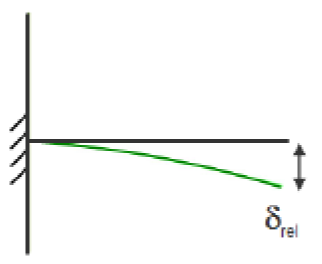
For each of X and Y directions:

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

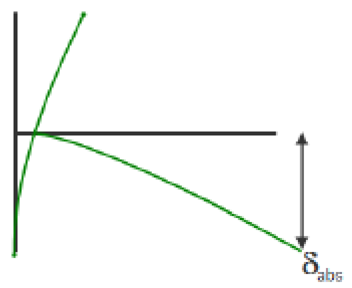
Deflection checks (AISC 360)

Relative and Absolute Deflections

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



Relative Deflection



Absolute Deflection

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

Steel grade (AISC 360)

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

WARNING For composite beams, the upper limit for the steel grade is defined in the AISC Specification as 75 ksi (525 MPa) - see I1.2 (360-05) or I1.3 (360-10, 360-16). If you add a grade higher than this and apply to a composite beam all the design checks will be flagged as beyond scope.

WARNING For non-composite beams, the upper limit for the steel grade is defined in the AISC 360 Commentary A3.1a as 100 ksi (690 MPa). If you add a grade higher than this and apply to a non-composite member (rolled or built-up) all the design checks will be flagged as beyond scope.

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

Steel beam design to AISC 360

Design method (Beams: AISC 360)

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

The design method employed is consistent with the design parameters specified in the relevant chapters of the AISC Specification and associated

'Commentary' (Ref. 2 (page 55)), unless specifically noted otherwise. As the 2005, 2010 and 2016 versions are all supported, where clauses are specific to a particular version these are indicated as (360-05), (360-10), or (360-16) as appropriate.

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

Steel beam limitations and assumptions (Beams: AISC 360)

The following general limitations apply:

- Continuous beams (more than one span) must be co-linear in the plane of the web within a small tolerance (sloping in elevation is allowed),
- Rolled doubly symmetric prismatic sections, doubly symmetric hollow sections, channel sections are fully designed, plated beams are also fully designed
- Single angles, double angles and tees are designed, but additional limitations apply, (see Angle and tee limitations)
- Design of beams with web openings is beyond scope.

The following additional limitations apply for plated beams:

- Double and single symmetric I-sections allowed
- Single or multi-span allowed, including cantilever spans
- Design for axial force (tension or compression) or flexure (major or minor) or any combination of these
- Non-composite only
- Flanges and web all have same grade steel
- No design of curved beams (plan or elevation)
- No auto design
- No torsion design
- No seismic design

The following assumptions apply:

- All supports are considered to provide torsional restraint, that is lateral restraint to both flanges. This cannot be changed. It is assumed that a beam that is continuous through the web of a supporting beam or column together with its substantial moment resisting end plate connections is able to provide such restraint.
- If, at the support, the beam over sails the supporting beam or column then the detail is assumed to be such that the bottom flange of the beam is well connected to the supporting member and, as a minimum, has torsional stiffeners provided at the support.

- In the Tekla Structural Designer model, when not at supports, coincident restraints to both flanges are assumed when one or more members frame into the web of the beam at a particular position and the cardinal point of the centre-line model of the beam lies in the web. Otherwise, only a top flange or bottom flange restraint is assumed. Should you judge the actual restraint provided by the in-coming members to be different from to what has been assumed, you have the flexibility to edit the restraints as required.
- Intermediate lateral restraints to the top or bottom flange are assumed to be capable of transferring the restraining forces back to an appropriate system of bracing or suitably rigid part of the structure.
- It is assumed that you will make a rational and "correct" choice for the effective lengths between restraints for compression buckling. **The default value for the effective length factor of 1.0 may be neither correct nor safe.**

Section classification (Beams: AISC 360)

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

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

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

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

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

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

Classification for plated beams

Since built-up (plated) beams allow for asymmetric sections, the general approach in flexure classification for all built-up beams is:

- under major bending the compression flange is classified (both flanges are classified if double curvature exists, and the worst case is reported)
- under minor bending both flanges are classified
- under biaxial bending, major and minor bending are considered independently and the worst case is reported

D2. Axial tension (Beams: AISC 360)

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

NOTE In the rupture check the net area *Ae* is assumed to equal the gross area *Ag*.

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

E. Axial compression (Beams: AISC 360)

If axial compression exists, the member is assessed for Flexural Buckling and for Torsional and Flexural Torsional buckling.

The compressive strength is determined in accordance with Equations E3.1 and E4.1. For double angles these equations are subject to the modifications of Section E6.

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

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

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

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

G2. Shear strength (Beams: AISC 360)

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

Plated beams only

Since built-up (plated) beams allow for asymmetric sections, under minor shear the web shear coefficient, C_v is calculated for each flange separately and Equation G2-1 taken as:

$V_n = 0.6 * F_y * (A_{w,top} * C_{v,top} + A_{w,btm} * C_{v,btm})$

F2. Flexure (Beams: AISC 360)

The member is assessed for Flexure in accordance with Section F2 to F10 (as appropriate).

The following checks are potentially required:

About the x axis - within the LTB braced length

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

About the y axis in the LTB braced length

- Yielding
- Flange local buckling
- Web local buckling
- Local buckling

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

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

Plated beams only

The following additional checks apply about the x axis:

- Compression flange yielding
- Tension flange yielding

The approach to evaluating the web plastification factors, R_{pc} and R_{pt} in Section F4.2 and F4.4 of 360-10, has been adopted for 360-05 also i.e. the ratio I_{yc}/I_y is considered as well as h_c/t_w but note the following:

- under 360-05, I_{yc} is taken as the minimum inertia about the y axis of top and bottom flange (regardless which flange is in compression)

- under 360-10, I_{yc} is taken as the inertia about the y axis of the compression flange being considered

For flange local buckling about the y axis Equation F6-2 is used for both double and single symmetric sections, but in the latter case the more slender of the two flanges is assessed i.e. the higher λ value will be used.

H1. Combined forces (Beams: AISC 360)

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

Plated beams only

For built-up (plated) beams a Proportioning Limit check applies. In AISC 360-05 and 360-10 this is detailed within the chapter on design for flexure (section F13.2). Load combinations which result in major axis bending on a built-up (plated) beam cause this check to be made. Any load combination which fails the Proportioning Limit check is considered as Beyond Scope for Combined Forces (regardless of whether any axial force is compressive or tensile).

DG9. Torsion (Beams: AISC 360)

Torsion design is carried out according to AISC design guide 9 (DG9), AISC 360-05 and AISC 360-10 for single span, pin ended steel beams with open and closed section types as follows:

Open sections (I- symmetric rolled)

- A torsion design and an angle rotation check can be carried out for applied torsion forces only

Closed sections (HSS only)

- •An angle of rotation check can be carried out for applied forces only

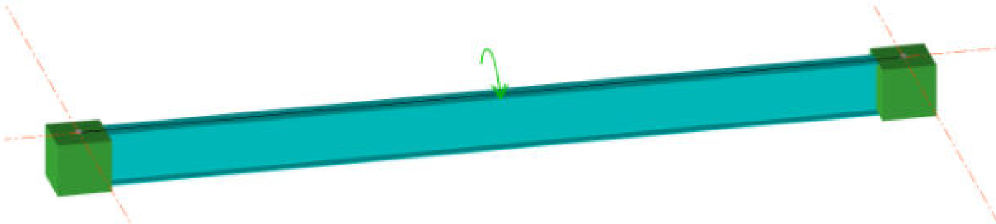
Torsion design - loading (Beams: AISC 360)

For design of open sections (i.e. rolled I sections in the current release) torsion design is carried out for “applied torsion loading” only and in accordance with those cases in Appendix B of DG9 with torsion fixed and warping free member ends (i.e. cases 3, 4 and 5 of DG9, with some extension for partial UDL and VDL).

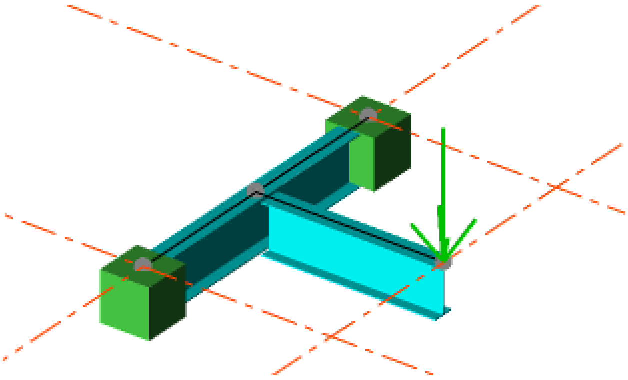
Applied torsion loading

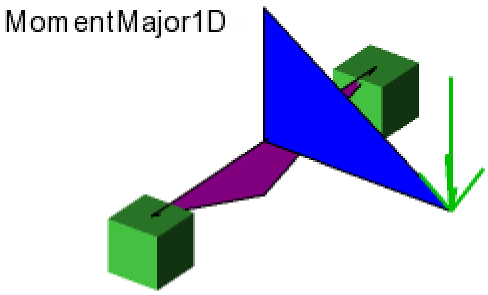
Tekla Structural Designer defines “applied torsion loading” as:

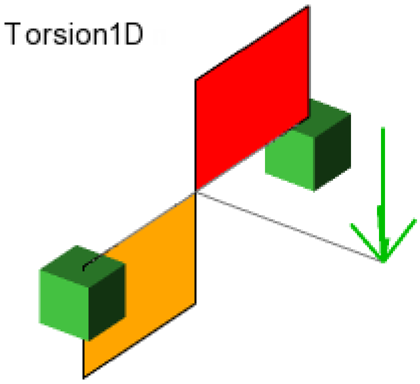
- A force that is manually applied by the User using the Member Loading in the Load ribbon, (as shown below)



- Or a force that is induced from a moment connection between primary and secondary beams, or a cantilever beam (as shown below, with bending and torsion moment diagrams).







Angle of rotation check (Beams: AISC 360)

I symmetric & HSS section

A torsion rotation check is optionally carried out based on the applied torsion loading only.

The check is applied by selecting “Apply rotation limit” (located in the steel beam properties under the Torsion heading). The default limit is also set in the steel beam properties as 2° but can be adjusted to suit.

Torsion design general checks (Beams: AISC 360)

1. Auto design is not carried out in the current release, only check design (and check design is only carried out if the “check for torsion” flag is set to on in the Edit dialog or Properties Window)
2. Lateral restraint amplification factor (in accordance with section 4.7.3 of DG9):
    a. It is assumed lateral displacement and twist are not restrained at any load point. Therefore, in accordance with section 4.7.3 of DG9, σ_{by} and σ_w will always be amplified in the presence of torsion.
    b. To avoid a negative value, Tekla Structural Designer applies a lower limit of 0.001ksi OR N/mm^2 to the denominator of the amplification factor,

       $(\phi F_{cr}^{e} - \sigma_{bx})$
    c. $F_{cr}^{e} = F_{cr,bx}$
    d. Amplification factor = 1.0 when $\sigma_{bx} = 0.0$
3. Both major and minor axis shear buckling are checked if loaded in the relevant axis. A warning is issued if the buckling limit defined in AISC Sect G2 is exceeded. Torsion design will, however, be continued - the engineer is expected to deem if the shear buckling condition is safe.
4. Torsion shear stresses:
    a. A cross-section check is carried out at points of interest taken from the load analysis diagram.
5. Combined forces and torsion:
    a. HSS - sections
        1. A cross-section check is carried out at points of interest taken from the load analysis diagrams as well as 10th positions along the member. In cases where the final utilization ratio approaches 1.0 we strongly recommend the engineer considers other locations, where a more critical location than that chosen in Tekla Structural Designer may exist.
    b. I symmetric

1. We take the most critical axial stress value across all axial strut lengths to determine $F_{cr,a}$
2. We take the most critical major bending stress value across all LTB lengths to determine $F_{cr,bx}$
3. In ASD design checks, the value used for $F_{cr,a}$ and $F_{cr,bx}$ is F_{cr} / 1.67 since F_a and F_b used in DG9 relate to the 1989 ASD Specification where this factor was effectively already taken into account.

Web openings (Beams: AISC 360)

Circular openings as an equivalent rectangle

Each circular opening is replaced by equivalent rectangular opening, the dimensions of this equivalent rectangle for use in all subsequent calculations are:

- d_o' = 0.9 * opening diameter
- l_o = 0.45 * opening diameter

Web opening design checks

Common design checks for both composite and non-composite beams

The following design checks are carried out at each opening for both composite and steel beams:

- **Section and opening dimension limit check** including the spacing of multiple openings if applicable.
- **Classification check**. Non-compact sections are beyond scope.
- **Moment-shear interaction check**. First the maximum pure flexural and shear strength is calculated following the guidelines of the Design Guide for the currently selected edition of the headcode. Then the direct formulas (3-5a and 3-5b) are used to calculate design shear and bending strength.
- **Deflection calculation**. As deflection calculations are headcode independent, for simplicity a single approach is used irrespective of the headcode selected.

Additional design checks for non-composite beams or composite beams at construction stage

The following additional design checks are carried out at each opening only for non-composite beams or composite beams at construction stage.

- **Lateral torsional buckling**. The 'standard' lateral torsional buckling check is run but the torsional constant is multiplied by a reduction factor according to the design guide. Strength over the openings should not be the governing UR.

- **Buckling of tee-shaped compression zone**. The tee which is in compression is investigated as an axially loaded column following the procedures of selected headcode. For unreinforced members this is not required when the aspect ratio of the tee is less than or equal to 4. For reinforced openings, this check is only required for large openings in regions of high moment.

Additional design checks for composite beams at composite stage

The following additional design checks are carried out at each opening only for composite beams at composite stage.

- **Slab reinforcement check**. The check of minimum transverse and longitudinal slab reinforcement ratio to prevent cracking of the slab in the vicinity of the web opening.
- **Number of shear connectors above the opening**. To limit the effect of bridging a minimum of two studs per foot is applied to the total number of studs. If this criterion is already satisfied by normal stud requirements, additional studs are not needed. A warning is shown when this criteria is not met.

Deflections

The simplified rules in DG2 are for limited cases and therefore have not been implemented. Instead Tekla Structural Designer uses a first principles approach as per Eurocodes.

The deflection of a beam with web openings will be greater than that of the same beam without openings. This is due to two effects:

- the reduction in the beam inertia at the positions of openings due to primary bending of the beam,
- the local deformations at the openings due to Vierendeel effects. This has two components - that due to shear deformation and that due to local bending of the upper and lower tee sections at the opening.

The primary bending deflection is established by 'discretising' the member and using a numerical integration technique based on 'Engineer's Bending Theory' - M/I = E/R = σ/y. In this way the discrete elements that incorporate all or part of an opening will contribute more to the total deflection.

The component of deflection due to the local deformations around the opening is established using a similar process to that used for cellular beams which is in turn based on the method for castellated beams given in the SCI publication, “Design of castellated beams. For use with BS 5950 and BS 449".

The method works by applying a 'unit point load' at the position where the deflection is required and using a 'virtual work technique to estimate the deflection at that position.

For each opening, the deflection due to shear deformation, δ_s, and that due to local bending, δ_{bt}, is calculated for the upper and lower tee sections at the opening. These are summed for all openings and added to the result at the desired position from the numerical integration of primary bending deflection.

Note that in the original source document on castellated sections, there are two additional components to the deflection. These are due to bending and shear deformation of the web post. For castellated beams and cellular beams where the openings are very close together these effects are important and can be significant. For normal beams the openings are likely to be placed a reasonable distance apart. Thus in many cases these two effects will not be significant. They are not calculated for such beams but in the event that the openings are placed close together a warning is given.

Seismic design rules (Beams: AISC 360)

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341) (Ref. 5 (page 55)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems.

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

For a moment resisting frame

9. Special Moment Frame (SMF)

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

10. Intermediate Moment Frame (IMF)

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

11. Ordinary Moment Frame (OMF)

- 11.4. Classification

Moment resisting frame with a truss component

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

For a braced frame

13. Special Concentrically Braced Frames (SCBF)

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

14. Ordinary Concentrically Braced Frames (OCBF)

- 14.2. Classification

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

15. Eccentrically Braced Frames (EBF) - Beyond Scope

Buckling resistant braced frame

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

Frames containing composite beams

- Composite Special Concentrically Braced Frames (C-SCBF) - Beyond Scope
- Composite Ordinary Braced Frames (C-OBF) - Beyond Scope
- Composite Eccentrically Braced Frames (C-EBF) - Beyond Scope

Composite beam design to AISC 360

Design method (Composite beams: AISC 360)

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

The design method employed is consistent with the design parameters for simple composite beams as specified in Chapter I of the AISC Specification and associated 'Commentary' (Ref. 2 (page 55)), unless specifically noted otherwise. As the 2005, 2010 and 2016 versions are all supported, where clauses are specific to a particular version these are indicated as (360-05), (360-10), or (360-16) as appropriate.

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

Serviceability limit state (SLS) (Composite beams: AISC 360)

Section properties (SLS)

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

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

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

Loadcase type	Properties used
self-weight	bare beam
Slab Dry	bare beam
Dead	composite properties calculated using the modular ratio for long term loads[1]
Live, Roof Live	composite properties calculated using the effective modular ratio[2] appropriate to the long term load percentage for each load.
Wind, Snow, Earthquake	composite properties calculated using the modular ratio for short term loads
Total loads	these are calculated from the individual loadcase loads as detailed above.

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

n_S = the short term modular ratio = E_S/E_c

n_L = the long term modular ratio = $(E_S/E_c) * k_n$

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

$n_E = n_S + \rho_L * (n_L – n_S)$

ρ_L = the proportion of the load which is long term

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

NOTE All the beam deflections calculated above are “relative” deflections. For an illustration of the difference between relative and absolute deflection see Deflection checks.

Stress checks (SLS)

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

Tekla Structural Designer calculates the worst stresses in the extreme fibers of the steel and the concrete at serviceability limit state for each load taking into account the proportion which is long term and that which is short term. These stresses are then summed algebraically. The partial safety factors for loads are taken as those provided by you for the service condition on the Design Combinations page. The stress checks assume that full interaction exists between the steel and the concrete at serviceability state.

Natural frequency checks (SLS)

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

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

Simplified approach

The natural frequency is determined from,

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

Where:

- Δ_{NF} = the maximum static instantaneous deflection (in inches) that would occur under the effects of Slab Dry loading, and the proportion of dead loads and live loads specified by the user (as specified on the Natural Frequency page of the Design Wizard). It is based upon the composite inertia but not modified for the effects of partial interaction.
- g = the acceleration due to gravity (386.4 in/s^2)
- Factor of increased dynamic stiffness of concrete flange (default 1.35)

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

Construction stage (Composite beams: AISC 360)

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

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

Section classification (Composite beams: AISC 360)

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

Shear strength - I3.1b (360-05), I4.2 (360-10) (Composite beams: AISC 360)

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

Strength during construction - I3.1c (360-05), I3.1b (360-10) (Composite beams: AISC 360)

Flexure

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

Lateral torsional buckling checks

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

In other cases any incoming beams will be automatically identified.

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

Deflection checks (Composite beams: AISC 360)

Relative deflections are used in the composite beam design. (See: Deflection checks (page 15))

The following deflections are calculated for the loads specified in the construction stage load combination:

- the Dead load deflections i.e. those due to the beam self weight, the Slab Wet loads and any other included dead loads,
- the live load deflections i.e. those due to construction live loads,
- the Total load deflection i.e. the sum of the previous items.

The loads are taken as acting on the steel beam alone.

The "Service Factor" (default 1.0), specified against each loadcase in the construction combination is applied when calculating the above deflections.

If requested by the user, the total load deflection is compared with either a span-over limit or an absolute value. The initial default limit is span/200, (as per CC.1.1 of ASCE 7-05 (Ref. 7) or ASCE 7-10 (Ref. 8)).

Composite stage (Composite beams: AISC 360)

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

Equivalent steel section (Composite beams: AISC 360)

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

Shear strength - I3.1b (360-05), I4.2 (360-10) (Composite beams: AISC 360)

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

Strength of composite beams with shear connectors - I3.2 (Composite beams: AISC 360)

Section classification

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

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

Flexure

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

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

Shear connectors (Composite beams: AISC 360)

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

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

Ribs perpendicular

The reduction factor R_p is taken as,

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

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

Ribs parallel

R_p = 0.75 in all cases

Ribs at other angles

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

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

Where:

k_s = the adjusted value of the combined reduction factor $R_g * R_p$

k_1 = the value of the combined reduction factor $R_g * R_p$ for ribs perpendicular

k_2 = the value of the combined reduction factor $R_g * R_p$ for ribs parallel

Degree of shear connection

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

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

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

Where:

T_s = the tensile yield strength of the steel section

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

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

Q_n = the nominal strength of an individual shear connector

The degree of partial shear connection is given by,

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

Where:

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

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

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

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

Dimensional requirements

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

- the nominal rib height of the profiled deck, h_r should be not greater than 3 in
- the mean width of the ribs of the profiled sheet, w_r should be not less than 2 in (for re-entrant decks the "mean" is taken as the minimum opening at the top of the rib)
- the nominal diameter of stud connectors, d_{sc} should be not greater than ¾ in
- the height of the stud after welding, H_s should be at least 1½ in greater than the nominal rib height of the profiled deck – see Section I3.2c(b) (360-05), or Section I3.2c(2) (360-10).
- the total depth of the composite slab, d_{cs} should not be less than 3¾ in
- the thickness of concrete above the main flat surface of the top of the ribs of the sheeting, d_{cs} - h_r should not be less than 2 in
- concrete cover, d_{cs} – H_s over the connector should not be less than ½ in – see Section I3.2c(b) (360-05), or Section I3.2c(2) (360-10).

- the longitudinal spacing should not exceed the lesser of 36 in or 8 * the slab depth, d_{cs} (see Section 6.2.6.2 of Structural Steel Designer's Handbook. Second Edition (Ref. 5))
- where studs are spaced at greater than 18 in centers puddle welds or other appropriate means are required to ensure anchorage of deck – see Section I3.2c (360-05), or Section I3.2c(4) (360-10).
- the clear distance between the edge of a connector and the edge of the steel beam flange should be not less than ¾ in (as universal good practice).
- Section I8.2d of the AISC Specification (360-10) requires that the minimum edge distance from the center of an anchor to a free edge in the direction of the shear force shall be 8 in for normal weight concrete and 10 in for lightweight concrete. This requirement will apply only in a limited number of configurations and therefore is not checked.
- the spacing of connectors in the direction of shear i.e. along the beam should be not less than, 6 * the stud diameter
- the spacing of connectors transverse to direction of shear i.e. across the beam should be not less than 4 * the stud diameter except for the condition given in the next item
- where rows of studs are staggered, the minimum transverse spacing between longitudinal lines of studs should be not less than 3 * the stud diameter with the amount of stagger such that the diagonal distance between studs on adjacent longitudinal lines is not less than 4 * the stud diameter
- the stud connector diameter should not exceed 2.5 times the flange thickness unless located directly over the web.

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

Steel column design to AISC 360

- Seismic design rules (Columns: AISC 360) (page 38)

Design method (Columns: AISC 360)

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

The design method employed is consistent with the design parameters specified in the relevant chapters of the AISC Specification and associated 'Commentary' (Ref. 2 (page 55)), unless specifically noted otherwise. As the 2005, 2010 and 2016 versions are all supported, where clauses are specific to a particular version these are indicated as (360-05), (360-10), or (360-16) as appropriate.

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

Section classification (Columns: AISC 360)

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

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

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

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

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

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

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

D2. Axial tension (Columns: AISC 360)

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

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

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

E. Axial compression (Columns: AISC 360)

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

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

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

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

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

G2. Shear strength (Columns: AISC 360)

Shear checks are performed for the absolute value of shear force normal to the x-x axis and normal to the y-y axis, F_{vx} and F_{vy}, at the point under consideration in accordance with Section G2.

F2. Flexure (Columns: AISC 360)

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

About the x axis - within the LTB sub-length

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

- Lateral Torsional Buckling (only required for I and C sections)

About the y axis in the LTB sub-length

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

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

When the checks are required you can set the effective length of each sub-beam (the portion of the beam between one brace and the next) either by giving factors to apply to the physical length of the beam, or by entering the effective length that you want to use.

H1. Combined forces (Columns: AISC 360)

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

Seismic design rules (Columns: AISC 360)

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05) (Ref. 5 (page 55)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems.

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

For a moment resisting frame

9. Special Moment Frame (SMF)

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

10. Intermediate Moment Frame (IMF) • 10.4a. Classification

- 10.4. Column strength

11. Ordinary Moment Frame (OMF)

- 11.4. Classification
- 11.4. Column strength check

Moment resisting frame with a truss component

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

For a braced frame

13. Special Concentrically Braced Frames (SCBF)

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

14. Ordinary Concentrically Braced Frames (OCBF)

- 14.2. Classification
- 14.2. Column strength check

15. Eccentrically Braced Frames (EBF) - Beyond Scope

Buckling resistant braced frame

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

Frames containing composite beams

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

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

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

Column base plate design to AISC 360

Column Bases: only simple column bases are supported in the current release.

Unless explicitly stated all calculations will adopt either a load and resistance factor design (LRFD) or an allowable strength design (ASD) as consistent with the design parameters for column base connections as specified in the AISC Specification (Ref. 2 (page 55)) and its associated 'Commentary', the AISC Steel Design Guide 1: Base Plate and Anchor Rod Design (Ref. 4 (page 55)), and the ACI 318 Building Code Requirements (Ref. 1 (page 55)).

The following advice is written principally from the point of view of operating column base plate design from within Tekla Structural Designer.

Practical applications

In the current release of Tekla Structural Designer only simple column base plate design checks are supported, following primarily the design procedures given in AISC Design Guide 1: Base Plate and Anchor Rod Design (2nd Edition, 2nd printing, revised). (Ref. 4 (page 55))

Tekla Structural Designer will check the base plate size and thickness, with the latter check displaying a bearing stress calculation. Shear resistance of the rods, concrete and the welds are also checked.

The concrete foundation design is checked *separately* in accordance with ACI 318. (Ref. 1 (page 55))

Graphics are used to display the base plate in its current state. You can therefore graphically see the base that you are defining and the results that the design process has achieved. This allows you to see the effects of any modifications that you make instantly on the screen. Reinforcement in the foundation is not represented in the graphics.

Limitations

For the current release of Tekla Structural Designer the following limitations apply:

- Column sections - Only the following column sections will have their base plate design checked:
  - I section columns (W, M, S & HP shapes)
  - HSS
  - Pipe sections
- Anchor rod layout - Anchor rod layouts are restricted as follows:
  - The rod layout must be symmetric about both axes
- Base plate position - The column base plate can only be concentrically placed about the major axes of the steel column, so that there is no eccentricity in the line of action of the vertical load with respect to the baseplate. In addition, the base plate can only be positioned symmetrically on the concrete base.
- Concrete pedestal - No concrete pedestal will be defined.
- Welds - Although welds are not checked in this release, partial length welds are permitted only on the webs of I section columns. For gravity loading the column is assumed to be prepared for direct contact in bearing.

Theory and assumptions

This section describes the theory used in the development of column base plate design checks and the major assumptions that have been made, particularly with respect to interpretation of the AISC Specification (Ref. 2 (page 55)) and the AISC Steel Design Guide 1 (Ref. 4 (page 55)). A basic knowledge of the design methods for column bases is assumed.

NOTE Column base plates designed to the AISC Specifications 360-05, -10 and -16 (Ref. 2 (page 55)) should all produce the same results as each other.

Checks Performed

The following table summarizes the main checks performed:

<table>
<tr><th>Applied Load</th><th>Calculations</th></tr>
<tr><td>Positive vertical load</td><td>Base plate
Bearing strength of concrete foundation.
Base plate thickness.</td></tr>
<tr><td>(Major) shear load</td><td>Shear
Friction Strength
Shear strength of rods
Concrete anchorage strength for shear forces on rods
Concrete pry-out strength of anchor rod in shear
Weld
Weld shear strength</td></tr>
<tr><td>Positive vertical + (major) shear load</td><td>Base Plate
Bearing strength of concrete foundation.
Base plate thickness.
Shear
Friction Strength
Shear strength of rods
Concrete anchorage strength for shear forces on rods
Concrete pry-out strength of anchor rod in shear
Weld
Weld shear strength</td></tr>
</table>

Base plate

- **Bearing strength of concrete foundation**

  This check is only performed under positive vertical load (i.e. axial compression).

  In the calculation of the nominal bearing strength of the concrete, P_p, the Area A_2 is as defined in the AISC Design Guide 1 (Ref. 4 (page 55)), i.e. "the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area" with a maximum value of $4*A_1$, where A_1 is the area of the base plate.

- **Base plate thickness**

  This check is only performed under positive vertical load (i.e. axial compression).

  The check requires $t_p \geq t_{min}$ where t_p = the base plate thickness, and t_{min} = the minimum thickness, according to LRFD or ASD as appropriate.

  The required bearing stress, f_{pu} (LRFD) or f_{pa} (ASD), is reported within the calculations for t_{min}.

Shear

There are three options to choose from for (major) shear transfer:

1. Friction alone
2. Shear on rods alone
3. Friction and shear on rods

1. **Friction Alone**

   - **Friction Strength**

     This shear strength check is calculated in accordance with ACI 318 (Ref. 1 (page 55)) and considers the full (major) shear force. Since ACI 318 only takes account of LRFD requirements then this check is not performed for ASD load combinations.

     The default for μ, the coefficient of friction between the base plate and concrete, is taken as 0.4 per ACI 349-01 (section RB.6.1.4), 349-06 and -13 (section RD.6.1.4) (Ref. 8 (page 55)).

2. **Shear on Rods Alone**

   Checks for Shear on Rods Alone are divided into 3 separate parts and consider the full (major) shear force:

   a. Shear strength of rods.

   b. Concrete anchorage strength for shear forces on rods.

   c. Concrete pry-out strength of anchor rod in shear.

   The anchor rod strength is calculated in accordance with AISC 360 Ref. 2 (page 55) and AISC Design Guide 1 (Ref. 4 (page 55)) for both LRFD and ASD load combinations.

The concrete strength checks are calculated in accordance with ACI 318 (Ref. 1 (page 55)). Since ACI 318 only takes account of LRFD requirements then these checks are not performed for ASD load combinations.

a. **Shear Strength of Rods**

AISC Design Guide 1 (Ref. 4 (page 55)) recommends taking a cautious approach when using anchor rods to transfer horizontal shear loads. Unless special provisions are made to equalize the load to all anchor bolts, such as using field welded setting plates or field welded washer plates, Design Guide 1 recommends using only 2 of the rods to resist the shear. If all rods are used to resist the shear, Design Guide 1 requires that the bending of the rods within the depth of the base plate should be checked which in turn requires that the washer plate/setting plate thickness is also checked. These rod bending and washer/setting plate thickness checks are not available in the current release. If the option to use either ‘Shear on rods’ or 'Friction and Shear on rods' to resist shear is selected then a Warning is displayed in the results viewer:

"*To transfer the shear equally to all anchor rods special provisions need to be made such as the use of field welded setting plates or field welded washers. These provisions are beyond the scope of this release and instead the recommendation given in Design Guide 1 to use only 2 rods to resist shear has been adopted.*"

b. **Concrete Anchorage Strength for Shear Forces on Rods**

This check is carried out in accordance with the provisions of ACI 318-11 Appendix D and Chapter 17 of ACI 318-14 and -19, which all have similar provisions, and back-fitted to ACI 318-08 (which has slightly different provision).

For the purposes of resisting shear forces, an anchor rod may act as an individual or as part of an anchor rod group which is defined in ACI 318 as "a number of anchors spaced at less than $3*c_{a1}$ from one or more adjacent anchors when subject to shear".

As only 2 rods are considered effective in resisting shear, it is assumed that the effective rods are those closest to the edge of the concrete foundation and, further, that these rods act as a group if spaced at less than $3*c'_{a1}$ where c'_{a1} is the limiting value of c_{a1} defined in ACI 318. This returns a conservative result.

In the calculation of the nominal concrete breakout strength, V_{cbg}, the modification factor for eccentricity of shear load, $\Psi_{ec,V}$, and the modification factor for cracked concrete, $\Psi_{c,V}$ are both set to 1.0

c. **Concrete Pry-out Strength of Anchor Rod in Shear**

This check is carried out in accordance with the provisions of ACI 318-08 and -11 section D.6.3, ACI 318-14 section 17.5.3, and ACI 318-19 section 17.7.3

Calculation of V_{cp}, the nominal concrete pry-out strength for rods in shear, involves calculation of N'_{cbg}, the nominal concrete breakout strength for rods in shear, which is determined in accordance with the provisions of ACI 318-08 and -11 section D.6.2, ACI 318-14 section 17.5.2, and ACI 318-19 section 17.7.2

In the calculation of N'_{cbg}, the nominal concrete breakout strength, a limiting value of rod embedded depth, h_{ef}, is used and labelled as h'_{ef}. The limiting value is determined in accordance with the provisions of ACI 318-08 and -11 section D.5.2.3, ACI 318-14 section 17.4.2.3, and ACI 318-19 section 17.6.2.1.2

Also in the calculation of N'_{cbg}, the nominal concrete breakout strength, a “total number of effective rods'' is referred to. Typically this will be equal to the total number of rods in the layout of rods, but for I section columns with 4 rows of rods, where all 4 rows are positioned inside the flanges of the column section, the internal 2 rows of rods are taken as having a total of 2 rods in each row - the 2 rods adjacent to either side of the column web. For example, in a rod layout of 4 rows of 4 rods in each row, with all 4 rows positioned inside the flanges of an I section column, the total number of effective rods = 2 * 4 + 2 * 2 = 12 (rather than the 16 rods in the total layout).

Also in the calculation of N'_{cbg}, the nominal concrete breakout strength, the modification factor eccentricity of shear load, $\Psi_{ec,N}$, and the modification factor for cracked concrete, $\Psi_{c,N}$ are both set to 1.0

3. **Friction and Shear on Rods**

   When Friction and Shear on Rods is selected as the (major) shear transfer option, then the three Shear on Rods checks consider a net shear force i.e. the remaining (major) shear force not taken by frictional resistance alone.

   Checks for Friction and Shear on Rods are divided into the same 3 separate parts as for Shear on Rods Alone (as detailed above), with the addition of a Friction calculation (as detailed above), at the start of each of the three Shear on Rods parts, that derives the net (major) shear force.

Welds

The welds are checked for one design condition: shear.

- **Weld Shear Strength**

  For base plates with thickness ≤ 3/4 in (19 mm), the default weld leg length is 1/4 in (6mm) and for all other base plate thicknesses the default weld leg length is 5/16 in (8 mm). These defaults can be adjusted in the database [Materials > Welds > Defaults].

  AISC 360-10 and -16 stipulates that when the length of the weld exceeds 300 times the leg size, w, the effective length shall be taken as 180w (section J.2b.(d).(3)), and this is similarly applied when AISC 360-05 has been selected for steel design.

Analysis

Connection forces are established from a global analysis of the building as a whole. Column base plates in Tekla Structural Designer have a limited set of design forces for which they can be designed. Non-design forces are identified and, where their value is greater than a given limit, they are displayed to you in the results along with a Warning status. The given limits are defined on the Design Forces page of the Design Settings dialog available from the Design tab on the ribbon.

The forces from the global analysis are treated in the following manner:

- Simple column bases are designed for the positive axial force at the base of the column and the shear (foundation reaction) in the plane of the column web (column section minor axis). Bases are orientated to the column's major and minor axes and hence there is no requirement to resolve the force when the column is rotated. Columns can only be sloped in the plane of the web and the bottom stack axial force and shear are resolved into vertical and horizontal forces in the base.

Where the global analysis includes second-order (P-Delta) effects the Ultimate Limit State design forces will include these effects also. However, for column bases the design forces for soil bearing pressure calculations are taken from an elastic global analysis of the unfactored loadcases without second-order effects. All seismic combinations appear in the results. However, those deriving from ELF are considered for design while those from RSA result in Beyond Scope status.

Sign Conventions

The following sign conventions apply.

Convention looking at the column with face A on the right:

- Positive shear from face C to A,
- Positive axial into the base.

NOTE	The column member direction arrow is placed on face A.

Steel brace design to AISC 360

- Design method (Braces: AISC 360) (page 46)
- Section classification (Braces: AISC 360) (page 46)
- D2. Axial tension (Braces: AISC 360) (page 46)
- E. Axial compression (Braces: AISC 360) (page 46)
- Seismic design rules (Braces: AISC 360) (page 47)

Design method (Braces: AISC 360)

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

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

The design method employed is consistent with the design parameters specified in the relevant chapters of the AISC Specification and associated 'Commentary' (Ref. 2 (page 55)), unless specifically noted otherwise. As the 2005, 2010 and 2016 versions are all supported, where clauses are specific to a particular version these are indicated as (360-05), (360-10), or (360-16) as appropriate.

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

Section classification (Braces: AISC 360)

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

D2. Axial tension (Braces: AISC 360)

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

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

E. Axial compression (Braces: AISC 360)

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

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

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

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

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

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

Seismic design rules (Braces: AISC 360)

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (AISC 341-05) (Ref. 5 (page 55)). These additional design rules ONLY apply to members in Seismic Load Resisting Systems.

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

For a braced frame

13. Special Concentrically Braced Frames (SCBF)

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

14. Ordinary Concentrically Braced Frames (OCBF)

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

15. Eccentrically Braced Frames (EBF) - Beyond Scope

Buckling resistant braced frame

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

Frames containing composite beams

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

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

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

Truss member design to AISC 360

- Design method (Trusses: AISC 360) (page 48)

Design method (Trusses: AISC 360)

Unless explicitly stated all truss calculations will adopt either a load and resistance factor design (LRFD) or an allowable strength design (ASD) as consistent with the design parameters as specified in the AISC Specification and associated Commentary (Ref. 2 (page 55)).

Design checks (Trusses: AISC 360)

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

- Internal
- Side
- Bottom
- Top

Depending on the type, different design procedures are adopted.

Internal and side truss members

The design checks for internal and side truss members are the same as those for braces. With the exception that seismic forces are not designed for. See: Steel brace design to AISC 360 (page 45)

Top and bottom truss members

The design checks for top and bottom truss members are the same as those for beams. With the exception that seismic forces are not designed for. See: Steel beam design to AISC 360 (page 16)

Steel single, double angle and tee section design to AISC 360

- Deflection of single angles (Angles and tees: AISC 360) (page 54)

Design method (Angles and tees: AISC 360)

Either a load and resistance factor design (LRFD) or an allowable strength design (ASD) can be performed to determine the adequacy of the section for each condition. The design method adopted is dictated by the member characteristic type:

Beam, Truss member top, or Truss member bottom:

- Member is designed for axial tension, compression, shear, bending and combined forces
- This is consistent with the method detailed in Steel beam design to AISC 360

Brace, Truss internal, or Truss member side:

- Member is designed for axial tension, compression and compression buckling only
- This is consistent with the method detailed in Steel brace design to AISC 360

A basic knowledge of the design method for angles and tees in accordance with the specification is assumed.

NOTE For tees, single angles, and double angles - specific additional Angle and tee limitations apply to the above design methods.

NOTE When modeling a double angle, the Compression property 'a' (distance between connectors), has to be manually set (either in Compression section of the **Properties** window or on the Compression tab of the **Properties** dialog box). Refer to AISC 360 section E6 for further information.

Angle and tee limitations (AISC 360)

- All sections and in particular single angles are assumed to be effectively loaded through the shear center such that no additional torsion moments are developed. In addition no direct allowance is made for 'destabilizing loads'.
- Design excludes bending of the outstand leg of single and double angles loaded eccentrically e.g. supporting masonry.

Section axes (Angles and tees: AISC 360)

For all sections:

- x-x is the axis parallel to the flanges
- y-y is the axis perpendicular to the flanges
  - for Single Angles and Double Angles
    - y-y parallel to long side (leg) - single angles
    - y-y parallel to long side (leg) - double angles with long leg back to back
    - y-y parallel to short side (leg) - double angles with short leg back to back
- w-w is the major principal axis for single angles
- z-z is the minor principal axis for single angles

Design procedure for single angles (Angles and tees: AISC 360)

Single angles with continuous lateral-torsional restraint along the length are permitted to be designed on the basis of **geometric axis (x, y) bending**.

Single angles without continuous lateral-torsional restraint along the length are designed using the provision for **principal axis (w, z) bending** except where the provision for bending about geometric axis is permitted.

Geometric axis bending permitted:

- If single angles without continuous lateral torsional restraint and legs of angles are equal and there is no axial compression and bending about one of the geometric axis only

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NOTE Design on the basis of geometric axis bending should also be permitted if single angles without continuous lateral torsional restraint but with lateral torsional restraint at the point of maximum moment only and legs of angles are equal and there is no axial compression and bending about one of the geometric axis only. However this is beyond scope of the current release of the program.

Geometric axis design

1. Nominal flexural strength Mnx – about X axis (major geometric axis)
2. Nominal flexural strength Mny – about Y axis (minor geometric axis)

Check:

IF LRFD

a. $M_{rx} \le \varphi_b * M_{nx}$, where $\varphi_b = 0.9$

b. $M_{ry} \leq \varphi_b * M_{ny}$, where $\varphi_b = 0.9$

IF ASD

$M_{rx} \leq M_{nx} / \Omega_b$, where $\Omega_b = 1.67$

$M_{ry} \leq M_{ny} / \Omega_b$, where $\Omega_b = 1.67$

Principal axis design

1. Required flexural strength M_{rw} – about W axis
2. Required flexural strength M_{rz} – about Z axis
3. Nominal flexural strength M_{nw} – about W axis (major principal bending axis)
4. Nominal flexural strength M_{nz} – about Z axis (minor principal bending axis)

Check:

IF LRFD

a. $M_{rw} \leq \varphi_b * M_{nw}$, where $\varphi_b = 0.9$

b. $M_{rz} \leq \varphi_b * M_{nz}$, where $\varphi_b = 0.9$

IF ASD

a. $M_{rw} \leq M_{nw} / \Omega_b$, where $\Omega_b = 1.67$

b. $M_{rz} \leq M_{nz} / \Omega_b$, where $\Omega_b = 1.67$

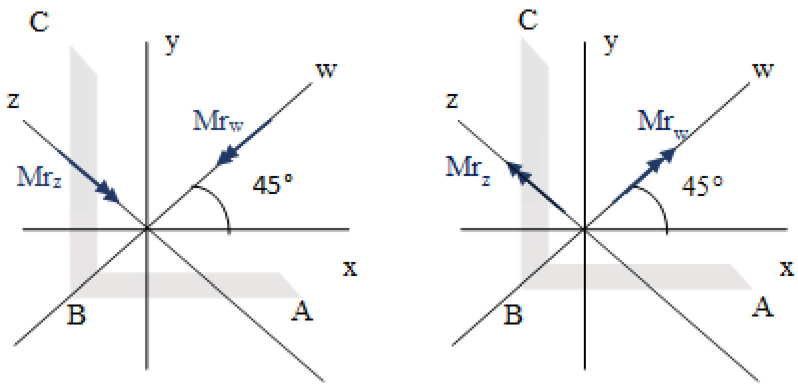
The principal axes moments are calculated from the following formulas for both LRFD and ASD:

$M_{rw} = M_{rx} \cos\alpha + M_{ry} \sin\alpha$

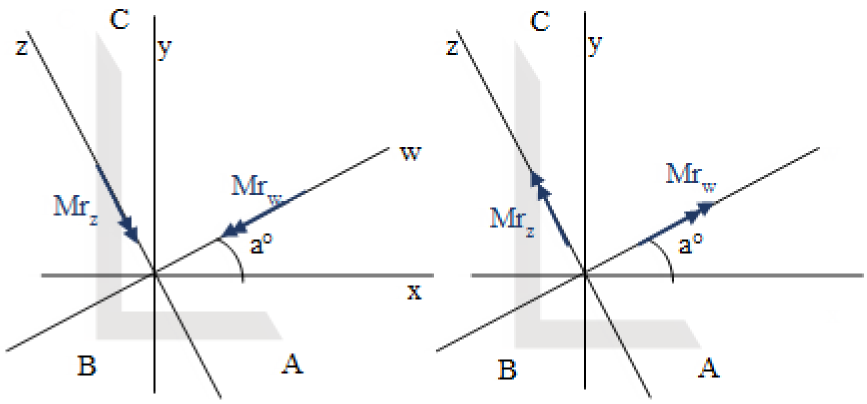
$M_{rz} = -M_{rx} \sin\alpha + M_{ry} \cos\alpha$

In the case of biaxial bending, or bending and axial force the combined stress ratio must be checked using the provisions of AISC, section H2.

For the **three points of the angle A, B, C** the combined ratio check should be performed.



Single Equal Angles - Sign of Moments



Single Unequal Angles - Sign of Moments

If the interaction of stresses at each point is seen to be less than 1.0 the member is adequate to carry the required load.

Check:[1]

Abs $(f_{ra} / F_{ca} + f_{rbw} / F_{cbw} + f_{rbz} / F_{cbz}) \leq 1.0$

NOTE In axial tension when the sum of the moment ratios about the major and minor axis bending is greater or equal to 0 then the axial stress ratio is taken as 0.0 in order to give conservative results and the axial stress ratio is renamed “effective”.

Design procedure for tee sections (Angles and tees: AISC 360)

The nominal flexural strength M_n is the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and leg local buckling.

$M_{nx} = M_{in} \{M_{nx,Yield}, M_{nx,LTB}, M_{nx,LLB}\}$

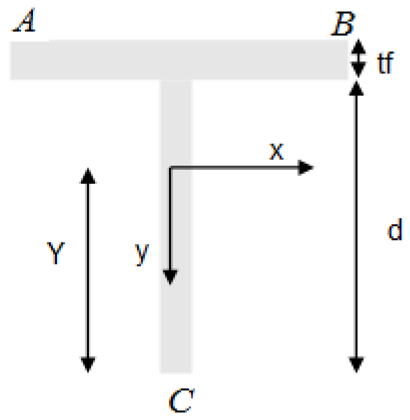
In the case of biaxial bending, or bending and axial force the combined stress ratio must be checked using the provisions of AISC, section H2.

The applied loads are

- P_r Axial
- M_{rx} Bending in x axis
- M_{ry} Bending in y axis

Check:[1]

Abs (f_{ra} / F_{ca} + f_{rbzx} / F_{cbx} + f_{rby} / F_{cby}) ≤ 1.0



Tees - Critical Points A, B & C

Design procedure for double angles (Angles and tees: AISC 360)

The nominal flexural strength M_n is the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and leg local buckling.

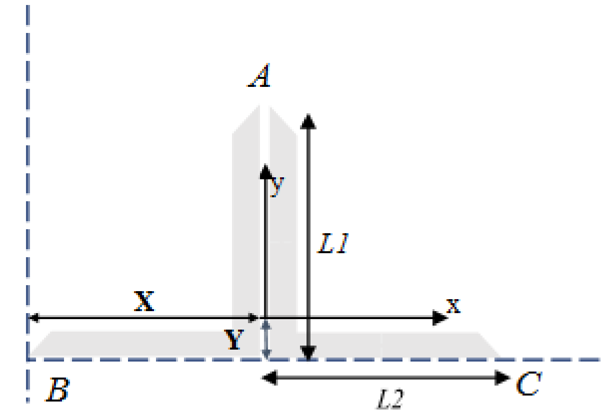
M_{nx} = Min {$M_{nx,Yield}$, $M_{nx,LTB}$, $M_{nx,LLB}$}

NOTE For the local buckling check of double angles the provisions of the 2010 code are used. In the 05 code, section F9.3 states Flange local Buckling of Tees and does not refer to double angles.

In the case of biaxial bending, or bending and axial force the combined stress ratio must be checked using the provisions of AISC, section H2. The applied loads are

- P_r Axial
- M_{rx} Bending in x axis
- M_{ry} Bending in y axis

Check:[1] Abs (f_{ra} / F_{ca} + f_{rbw} / F_{cbx} + f_{rby} / F_{cby}) ≤ 1.0



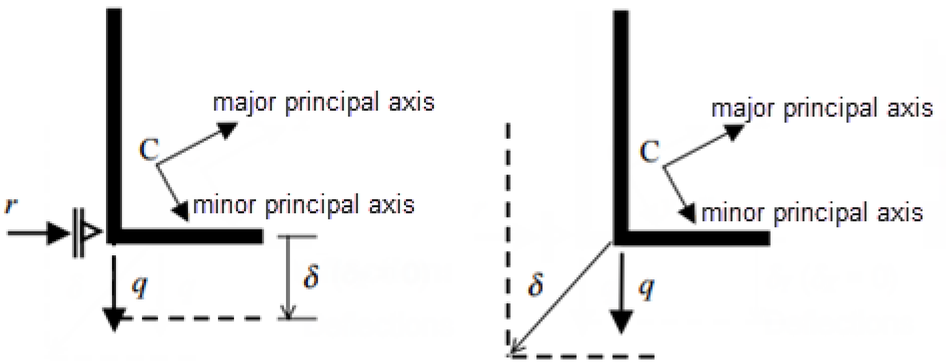
Double Angles - Critical Points A, B & C

Deflection of single angles (Angles and tees: AISC 360)

If a single angle is continuously restrained the major geometric moment and major geometric section properties are used in the general equation governing the beam deflection.

However, because single angle geometric axes are not coincident with the principal axes; a different procedure is required if the angle is not continuously restrained, the procedure being as follows:

1. External loads are transposed from the geometric axes to the principal axes.
2. The deflection equations are used to calculate deflections in the principal axes.
3. These principal axis deflections are then transposed to geometric axes again.



Single Angle Deflections (continuously restrained, unrestrained)

References (AISC 360)

1. American Concrete Institute. *Building Code Requirements for Structural Concrete and Commentary. ACI 318-08, -11, -14, -19.* ACI, 2008, 2011, 2014, 2019
2. American Institute of Steel Construction. *ANSI/AISC 360-05, -10, -16 Specification for structural steel buildings*. AISC, 2005, 2010, 2016
3. American Institute of Steel Construction. *Steel Design Guide Series 11. Floor Vibrations due to Human Activity.* AISC, 1997.
4. American Institute of Steel Construction. *Design Guide 1: Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing, revised)*. AISC, March 2014
5. American Institute of Steel Construction. *ANSI/AISC 341-05, -10, -16 Seismic Provisions for Structural Steel Buildings.* AISC, 2006, 2010, 2016.
6. American Society of Civil Engineers. *Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-05, -10.* ASCE, 2006, 2010.
7. Brockenbrough, R. L. & Merritt, F. S. *Structural Steel Designer's Handbook. Second Edition.* McGraw-Hill 1994 USA.
8. American Concrete Institute. *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary. ACI 349-01, 349-06, 349-13*. ACI 2001, 2006, 2013

1.3 Steel seismic design to AISC 341

Additional seismic provisions are required to be applied to members that are part of the seismic force resisting system (SFRS) of a structure. These provisions are applied in addition to any standard requirements for structural steel buildings as per AISC 360. The seismic provisions are contained in AISC 341. Tekla Structural Designer covers non-seismic steel design to AISC 360-05, AISC 360-10 and AISC 360-016. (Ref. 2 (page 55)). The purpose of this guide is to describe the matching seismic design requirements contained in AISC 341-05, AISC 341-10 and AISC 341-16 (Ref. 5 (page 55)).

- Criteria assumed to be met (page 56)
- Design philosophy (page 60)
- Changes introduced in AISC 341-16 (page 60)
- Common seismic requirements (page 61)
- Seismic checks - Beams (page 66)
- Seismic checks - Columns (page 70)

- Seismic checks - Braces (page 77)

Criteria assumed to be met (Seismic: AISC 341)

Seismic design in the current release of Tekla Structural Designer covers only those checks detailed in later sections and presupposes certain criteria are met e.g. that lateral braces to beams are sufficiently strong. These presuppositions are noted below.

Common

AISC 341-16 and AISC 341-10

- Column bases are assumed to comply with the requirements of D2.6.
- Steel material grades used in particular members and SFRS type are assumed to comply with A3.1.

AISC 341-05

- Column bases are assumed to comply with the requirements of 8.5.
- Steel material grades used in particular members and SFRS type are assumed to comply with 6.1.

OMF

AISC 341-16 and AISC 341-10

- Beam to column connections used in the SFRS are assumed to satisfy the requirements of E1.6.

AISC 341-05

- Beam to column connections used in the SFRS are assumed to satisfy the requirements of 11.2.
- Continuity plates are assumed to comply with the requirements of 11.5.
- As per 11.9 column splices are assumed to comply with the requirements of 8.4a.

IMF

AISC 341-16 and AISC 341-10

- The lateral braces themselves will not be designed to meet the additional criteria of D1.2a & c - it is assumed that the user will check this independently.

- The position of lateral braces will not be checked for the location of points of concentrated force or positions of plastic hinge per D1.2c.
- The protected zone is assumed to comply with E2.5c.
- Connections used in the SFRS are assumed to satisfy the requirements of E2.6.

AISC 341-05

- Beam to column connections used in the SFRS are assumed to satisfy the requirements of 10.2.
- Panel zones in beam to column connections used in the SFRS are assumed to satisfy the requirements of 10.3.
- Continuity plates are assumed to comply with the requirements of 10.5.
- The lateral braces themselves will not be designed to meet the additional criteria of 10.8 - it is assumed that the user will check this independently.
- The position of lateral braces will not be checked for the location of points of concentrated force or positions of plastic hinge per 10.8.
- As per 10.9 column splices are assumed to comply with the requirements of 8.4a.

SMF

AISC 341-16 and AISC 341-10

- The lateral braces themselves will not be designed to meet the additional criteria of D1.2b & c - it is assumed that the user will check this independently.
- The position of lateral braces will not be checked for the location of points of concentrated force or positions of plastic hinge per D1.2c.
- Beam column connections are always assumed braced as per E3.4c(1).
- The protected zone is assumed to comply with E3.5c.
- Connections used in the SFRS are assumed to satisfy the requirements of E3.6.

AISC 341-05

- Beam to column connections used in the SFRS are assumed to satisfy the requirements of 9.2.
- Panel zones in beam to column connections used in the SFRS are assumed to satisfy the requirements of 9.3.
- Continuity plates are assumed to comply with the requirements of 9.5.
- Beam column connections are always assumed braced as per 9.7a.

- The lateral braces themselves will not be designed to meet the additional criteria of 9.8 - it is assumed that the user will check this independently.
- The position of lateral braces will not be checked for the location of points of concentrated force or positions of plastic hinge per 9.8.
- Column splices are assumed to comply with the requirements of 9.9 whilst those not part of the SFRS are assumed to comply with 8.4b

OCBF

AISC 341-16 and AISC 341-10

- Column splices are assumed to comply with D2.5.
- It is assumed that the beams in OCBF are continuous between columns in accordance with F1.4a.
- It is assumed that the user will ensure that the ends of V and A braces are vertically released so that they provide no support for dead and live loads as per F1.4a (1).
- It is assumed that the user will apply the relevant lateral restraint at position of V/A braces or establish that the beam has sufficient out of plane strength and stiffness to ensure stability in order to comply with F1.4a (2).
- K braces are not permitted for OCBF in accordance with F1.4b.
- Coincident V and A braces giving X type braced frames are out of scope for additional beam checks required by AISC 341-10 F1.4a.
- Bracing connections in the SFRS are assumed to satisfy the requirements of F1.6.
- OCBF above seismic isolation systems are currently beyond scope

AISC 341-05

- Column splices are assumed to comply with 8.4a. Column splices in columns not part of the SFRS are assumed to comply with 8.4b.
- It is assumed that the beams in OCBF are continuous between columns in accordance with 14.3.
- It is assumed that the user will ensure that the ends of V and A braces are vertically released so that they provide no support for dead and live loads as per 14.3 (1).
- Lateral braces are not designed to meet the additional criteria in 14.3 (2) - it is assumed that the user will check this independently.
- It is assumed that the user will apply the relevant lateral restraint at position of V/A braces or establish that the beam has sufficient out of plane strength and stiffness to ensure stability in order to comply with 14.3 (2).
- K braces are currently beyond scope for OCBF.

- Coincident V and A braces giving X type braced frames are out of scope for additional beam checks required by AISC 341-05 14.3.
- Bracing connections in the SFRS are assumed to satisfy the requirements of 14.4.
- OCBF above seismic isolation systems are currently beyond scope

SCBF

AISC 341-16 and AISC 341-10

- Column splices are assumed to comply with D2.5.
- Coincident V and A braces giving X type braced frames are out of scope for additional beam checks required by AISC 341-10 F2.3a.
- It is assumed that the force resisted by tension braces is between 30% and 70% of the total horizontal force along the line of braces as per F2.4a.
- It is assumed that the beams in SCBF are continuous between columns in accordance with F2.4b (1).
- It is assumed that the user will apply the relevant lateral restraint at position of V/A braces or establish that the beam has sufficient out of plane strength and stiffness to ensure stability in order to comply with F2.4b (2).
- K braces are not permitted for SCBF in accordance with F2.4c.
- Tension only braces are not permitted for SCBF in accordance with F2.4d.
- Bracing connections in the SFRS are assumed to satisfy the requirements of F2.5b.
- The protected zone is assumed to comply with F2.5c

AISC 341-05

- It is assumed that the force resisted by tension braces is between 30% and 70% of the total horizontal force along the line of braces as per 13.2c.
- Bracing connections in the SFRS are assumed to satisfy the requirements of 13.3.
- It is assumed that the user will ensure that the ends of V and A braces are vertically released so that they provide no support for dead and live loads as per 13.4a (1).
- It is assumed that the beams in SCBF are continuous between columns in accordance with 13.4a (2).
- Lateral braces are not designed to meet the additional criteria in 13.4a (2) - it is assumed that the user will check this independently.
- It is assumed that the user will apply the relevant lateral restraint at position of V/A braces or establish that the beam has sufficient out of plane strength and stiffness to ensure stability in order to comply with 13.4a.

- K braces are not permitted for SCBF in accordance with 13.4b.
- Coincident V and A braces giving X type braced frames are out of scope for additional beam checks required by AISC 341-05 13.4.
- Column splices are assumed to comply with 13.5. Column splices in columns not part of the SFRS are assumed to comply with 8.4b.
- The protected zone is assumed to comply with 13.6.

Design philosophy (Seismic: AISC 341)

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

Additional design rules are required for seismic combinations. These are as per the AISC Seismic Provisions (Ref. 5 (page 55)). These rules are applied as follows:

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

For each of Direction 1 and Direction 2:

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

Where requirements are necessary then they apply only to the members of the SFRS and are only checked for the seismic combinations.

Changes introduced in AISC 341-16 (Seismic AISC 341)

Design is available to AISC 341-05, AISC 341-10 and AISC 341-16. The changes between the 10 and 16 versions, as implemented in Tekla Structural Designer are listed below.

Column required strength (D1.4a) (applies to all SFRS)

- Only for 'overstrength seismic load'
- Previously called 'amplified seismic load'
- Capacity analysis' now omitted

OCBF and V&A braces (F1.4a)

- Tension yield strength/amplified seismic + post buckling strength has been replaced by overstrength seismic load + post buckling strength

OCBF and beam design (F1.5c)

- New requirement, beams in SFRS are now designed for overstrength seismic load

SMF Moment ratio (E3.4a) - strong column weak beam

- Enhanced plastic moment capacity of beams 1.1 R_y F_y Z has been replaced by maximum probable moment, Cpr/α_s R_y F_y Z. Cpr from AISC 358-16

Common seismic requirements (Seismic: AISC 341)

Required strength

The required strength (including overstrength effects) for a member should be determined from:

The expected yield stress R_y x F_y

The expected tensile strength R_t x F_u

Grade	F_y	R_y	R_t
A36	36	1.5	1.2
A53B	35	1.6	1.2
A500B	42	1.4	1.3
A500B	46	1.4	1.3
A500C	46	1.4	1.3
A500C	50	1.4	1.3
A501	36	1.4	1.3
A529	50	1.2	1.2
A529	55	1.1	1.2
A572	42	1.3	1.0[1]
A572	50	1.1	1.1
A572	55	1.1	1.1
A913	50	1.1	1.1
A913	60	1.1	1.1
A913	65	1.1	1.1
A992	50	1.1	1.1

[1]This value is 1.1 in AISC 341-05.

AISC 341-16 and AISC 341-10 seismic classification - all members

When required by the seismic checks, the classification of elements of the cross section for various member types is as follows.

Compiled from Table D1.1 of AISC 341-10

Section	Element	Width thickness ratio	Application	λ_{hd} – highly ductile	λ_{md} – moderately ductile
I (rolled)	Flange	$b_f/(2 * t_f)$	Beams, Columns, Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$
	Web	h/t_w	Braces	$1.49 * \sqrt{(E/F_y)}$	$1.49 * \sqrt{(E/F_y)}$
	Web	h/t_w	Columns, Beams	LRFD - $C_a = P_u/(\phi_c * F_yA_g)$ $\phi c = 0.9$ ASD - $C_a = c * P_a/(F_y * A_g)$ $\Omega_c = 1.67$ $C_a \le 0.125$ $2.45 * \sqrt{(E/F_y)} * (1 - 0.93 * C_a)$ but for SMF only $\le 2.45 * \sqrt{(E/F_y)}$ $C_a > 0.125$ $0.77 * \sqrt{(E/F_y)} \times (2.93 - C_a)$ but $\ge 1.49 * \sqrt{(E/F_y)}$	LRFD - $C_a = P_u/(\phi_c * F_yA_g)$ $\phi_c = 0.9$ ASD - $C_a = c * P_a/(F_y * A_g)$ $\Omega_c = 1.67$ $C_a \le 0.125$ $3.76 * \sqrt{(E/F_y)} * (1 - 2.75 * C_a)$ but for IMF only $\le 3.76 * \sqrt{(E/F_y)}$ $C_a > 0.125$ $1.12 * \sqrt{(E/F_y)} \times (2.33 - C_a)$ but $\ge 1.49 * \sqrt{(E/F_y)}$
RHS and SHS	Walls	$(b_f - 3 * t)/t$ and $(d - 3 * t)/t$	Braces	$0.55 * \sqrt{(E/F_y)}$	$0.64 * \sqrt{(E/F_y)}$
RHS and SHS	Walls	$(b_f - 3 * t)/t$ and $(d - 3 * t)/t$	Columns	$0.55 * \sqrt{(E/F_y)}$	$1.12 * \sqrt{(E/F_y)}$

CHS		D/t	Braces	$0.038 * E/F_y$	$0.044 * E/F_y$
CHS		D/t	Columns	$0.038 * E/F_y$	$0.07 * E/F_y$
C (rolled)	Flange	b_f/t_f	Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$
	Web	h/t_w	Braces	$1.49 * \sqrt{(E/F_y)}$	$1.49 * \sqrt{(E/F_y)}$
Tees	Flange	$b_f/(2 * t_f)$	Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$
	Stem	d/t_w	Braces	$0.30 * \sqrt{(E/Fy)}$	$0.38 * \sqrt{(E/Fy)}$
Angles	Both legs	L_1/t and L_2/t	Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$
Double angles	Outstand leg – legs in continuous contact	L_1/t (long leg B to B) or L_2/t (short leg B to B)	Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$
Double angles	Both legs – legs separated	L_1/t and L_2/t	Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$

In the above table the terms have their usual meaning as follows:

- b_f = width of flange and for RHS width of shorter side
- t_f = thickness of flange of I/H, channel or Tee
- h = height of web inside flanges (d – 2 * t_f) of I/H or channel
- t_w = thickness of web
- d = depth of SHS and for RHS depth of longer side
- t = thickness of hollow section RHS, SHS, CHS
- D = diameter of CHS
- L_1 = Short leg (from root to toe) of single angle
- L_2 = Long leg (from root to toe) of single angle
- E = modulus of elasticity of steel – 29000 ksi
- F_y = minimum yield stress
- P_u = required axial strength using LRFD (seismic) combinations
- P_a = required axial strength using ASD (seismic) combinations
- A_g = gross area of section

AISC 341-05 seismic classification - all members

When required by the seismic checks, the classification of elements of the cross section for various member types is as follows.

Compiled from I-8-1 of AISC 341-05 and Table B4.1 of AISC 360-05.

Section	Element	Width thickness ratio	Application	λ_{ps} – seismically compact	λ_{ps} – conventionally compact
I (rolled)	Flange	$b_f/(2 * t_f)$	Beams, Columns[1], Braces	$0.30 * \sqrt{(E/F_y)}$	$0.38 * \sqrt{(E/F_y)}$
	Web	h/t_w	Columns[2], Beams, Braces	LRFD - $C_a = P_u/(\phi_c * F_y A_g)$ $\phi_c = 0.9$ ASD - $C_a = c * P_a/(F_y * A_g)$ $\Omega_c = 1.67$ $C_a \le 0.125$ $3.14 * \sqrt{(E/F_y)} * (1 - 1.54 * C_a)$ but for SMF only $\le 2.45 * \sqrt{(E/F_y)}$ $C_a > 0.125$ $1.12 * \sqrt{(E/F_y)} * (2.33 - C_a)$ but $\ge 1.49 * \sqrt{(E/F_y)}$	$3.76 * \sqrt{(E/F_y)}$
RHS and SHS	Walls	$(b_f-3t)/t$ and $(d-3t)/t$	Columns, Braces	$0.64 * \sqrt{(E/F_y)}$	$1.12 * \sqrt{(E/F_y)}$
CHS		D/t	Columns, Braces	$0.044 * E/F_y$	$0.070 * E/F_y$
C (rolled)	Flange	b_f/t_f	Braces	$0.30 * \sqrt{(E/F_y)}$	N/A

	Web	h/t_w	Braces	LRFD - $C_a = P_u/(\phi_c * F_yA_g)$ $\phi_c = 0.9$ ASD - $C_a = c * P_a/(F_y * A_g)$ $\Omega_c = 1.67$ $C_a \leq 0.125$ $3.14 * \sqrt{(E/F_y)} * (1 - 1.54 * C_a)$ $C_a > 0.125$ $1.12 * \sqrt{(E/F_y)} * (2.33 - C_a)$ but $\geq 1.49 * \sqrt{(E/F_y)}$	N/A
Tees	Flange	$b_f/(2 * t_f)$	Braces	$0.30 * \sqrt{(E/F_y)}$	N/A
	Stem	d/t_w	Braces	$0.30 * \sqrt{(E/F_y)}$	N/A
Angles	Both legs	L_1/t and L_2/t	Braces	$0.30 * \sqrt{(E/F_y)}$	N/A
Double angles	Outstand leg – legs in continuous contact	L_1/t (long leg B to B) or L_2/t (short leg B to B)	Braces	$0.30 * \sqrt{(E/F_y)}$	N/A
Double angles	Both legs – legs separated	L_1/t and L_2/t	Braces	$0.30 * \sqrt{(E/F_y)}$	N/A

Note 1: The relaxation on the compactness limit for columns in SMF as per note “b” to Table I-8-1 is not taken into account.

Note 2: These limits are not modified by Note [j] to Table I-8-1 i.e. this dispensation is not taken into account.

In the above table the terms have their usual meaning as follows:

- b_f = width of flange and for RHS width of shorter side
- t_f = thickness of flange of I/H, channel or Tee

- h = height of web inside flanges (d – 2 * t_f) of I/H or channel
- t_w = thickness of web
- d = depth of SHS and for RHS depth of longer side
- t = thickness of hollow section RHS, SHS, CHS
- D = diameter of CHS
- L_1 = Short leg (from root to toe) of single angle
- L_2 = Long leg (from root to toe) of single angle
- E = modulus of elasticity of steel – 29000 ksi
- F_y = minimum yield stress
- P_u = required axial strength using LRFD (seismic) combinations
- P_a = required axial strength using ASD (seismic) combinations
- A_g = gross area of section

Seismic checks - Beams (Seismic: AISC 341)

Classification

In all cases if the given "width to thickness ratio" is less than or equal to the given limit, then the seismic classification is satisfied.

AISC 341-16 and AISC 341-10

Beams in OMF and OCBF – No additional requirements.

Beams in IMF and SCBF – Beams must satisfy the requirements of clause D1.1b for "moderately ductile" members.

Beams in SMF – Beams must satisfy the requirements of clause D1.1b for "highly ductile" members.

The loading conditions affect the seismic classification in the following way,

- Axial tension only – no classification required.
- Any other loading condition – the appropriate rules in the section classification table are applied.

See: AISC 341-16 and AISC 341-10 seismic classification - all members (page 61)

AISC 341-05

Beams in OMF, SCBF and OCBF – No additional requirements.

Beams in IMF – Beams must satisfy clause 8.2a i.e. the requirements for Compact sections to AISC 360-05 in Table B4.1.

Beams in SMF – Beams must satisfy clause 8.2b for “seismically compact” sections.

The loading conditions affect the seismic classification in the following way,

- Axial tension – no classification required.
- Major axis bending only – applies to beams in SMF only.
- Any other loading condition – apply appropriate rules in the section classification table.

See: AISC 341-05 seismic classification - all members (page 63)

Stability bracing

AISC 341-16 and AISC 341-10

Beams in certain SFRS frame types must be provided with “stability bracing” to restrain lateral torsional buckling. There are two “levels” of requirement; one for ‘Moderately ductile members and one for “Highly ductile members”. The use of these depends upon the SFRS frame type as defined below.

Beams in OMF and OCBF – No additional requirements.

Beams in IMF and SCBF – Beams must satisfy clause D1.2a for “moderately ductile” members. For SCBF this only applies in the presence of V or A braces.

Beams in SMF – Beams must satisfy clause D1.2b for “highly ductile” members

Moderately Ductile

Beams shall be braced per D1.2a for moderately ductile members, i.e. maximum spacing per D1.2a(3)

$L_{pd} = 0.17 \times (E/F_y) \times r_y$

The design condition is,

$L_b \leq L_{pd}$

L_b = the laterally unbraced length of the compression flange taken as the beam length between locations where both the top flange and bottom flange are restrained for LTB.

Highly Ductile

Beams shall be braced per D1.2b for highly ductile members, i.e. maximum spacing,

$L_{pd} = 0.086 \times (E/F_y) \times r_y$

The design condition is,

$L_b \leq L_{pd}$

NOTE In both cases:

- The position of lateral braces is not checked for the location of points of concentrated force or positions of plastic hinge

- Lateral braces are not designed to meet the additional criteria for strength and stiffness.

AISC 341-05

OCBF and SCBF

Reference AISC 341-05 14.3 (2) and 13.4a (2) respectively.

For doubly symmetric I sections,

$L_{pd} = (0.12 + 0.076 \times (M_1/M_2)) \times (E/F_y) \times r_y$

Where

M_1 = the smaller moment at end of unbraced length – min(abs(M_a), abs(M_b))

M_2 = the larger moment at end of unbraced length – max(abs(M_a), abs(M_b))

NOTE Lateral braces are assumed to meet the strength and stiffness requirements of Equations A-6-7 and A-6-8 of AISC 360-05, Appendix 6.

NOTE Lateral braces are assumed to be provided at the intersection of the V/A brace and the beam or the beam has sufficient stiffness to satisfy the criteria in the “User Note” to AISC 341-05, 13.4a. In both cases the program assumes this intersection to be a braced point (with both flanges braced).

OMF

Reference AISC 341-05 11.8 – no additional requirements.

IMF

Reference AISC 341-05 10.8.

Max spacing of coincident restraints along top and bottom flange, continuous bracing is included while determining unbraced length for each flange,

$L_{pd} = 0.17 \times (E/F_y) \times r_y$

The design condition is,

$L_b \leq L_{pd}$

L_b = the laterally unbraced length of the compression flange taken as the beam length between locations where both the top flange and bottom flange are restrained for LTB.

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

NOTE Lateral braces are not designed to meet the additional criteria for strength and stiffness.

SMF

Reference AISC 341-05 9.8.

Max spacing of coincident restraints along the top and bottom flange, continuous bracing is included while determining the unbraced length for each flange,

$L_{pd} = 0.086 \times (E/F_y) \times r_y$

The design condition is,

$L_b \leq L_{pd}$

L_b = the laterally unbraced length of the compression flange taken as the beam length between locations where both the top flange and bottom flange are restrained for LTB.

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

NOTE Lateral braces are not designed to meet the additional criteria for strength and stiffness.

Design for brace forces in SCBF and OCBF

In both variants of the code the beams in SCBF and OCBF that are configured with “V” or “A” braces are required to resist a “push-pull” force generated by the brace pair. In general terms the compression brace is assumed to retain a percentage of its resistance post-buckling and the tension brace is assumed to have a level of defined “overstrength”.

NOTE In meeting the requirements of F1.4 and F2.3, the design of beams is separated into two distinct approaches – those beams with V&A braces at mid-span and those with diagonal braces at their ends. Clearly, frames that are braced using V&A braces have to meet both requirements e.g. in chevron systems but each is checked individually and this is believed to be conservative.

AISC 341-16 and AISC 341-10, V and A braces

OCBF

Reference AISC 341-10 F1.4a.

NOTE The tension yield strength/amplified seismic + post buckling strength in AISC 341-10 is replaced by overstrength seismic load + post buckling strength in AISC 341-16.

NOTE The lower bound on the force in the tension brace, “The maximum force that can be developed by the system”, according to F1.4a (1) (i) (c) is not applied.

SCBF

Reference AISC 341-10 F2.3.

AISC 341-16 and AISC 341-10, all other braces

OCBF

No requirements.

SCBF

Reference AISC 341-10 F2.3.

The expected tension strength, the expected compression strength and the expected post buckling strength are determined in the same way as for V and A braces.

AISC 341-05, V and A braces

OCBF and SCBF

Reference AISC 341-05 14.3 and 13.4a respectively.

Seismic checks - Columns (Seismic: AISC 341)

Classification

In all cases if the given “width to thickness ratio” is less than or equal to the given limit, then the seismic classification is satisfied.

AISC 341-16 and AISC 341-10

Columns in OMF and OCBF – No additional requirements.

Columns in IMF – Columns must satisfy the requirements of clause D1.1b for “moderately ductile” members.

Columns in SMF and SCBF – Columns must satisfy the requirements of clause D1.1b for “highly ductile” members.

The loading conditions affect the seismic classification in the following way,

- Axial tension only – no classification required.

- Any other loading condition – the appropriate rules in the section classification table are applied.

See: AISC 341-16 and AISC 341-10 seismic classification - all members (page 61)

AISC 341-05

Columns in OMF and OCBF – No additional requirements.

Columns in IMF – Columns must satisfy clause 8.2a i.e. the requirements for Compact sections to AISC 360-05 in Table B4.1.

Columns in SMF and SCBF – Columns must satisfy Clause 8.2b for “seismically compact” sections.

The loading conditions affect the seismic classification in the following way,

- Axial tension – no classification required.
- Any other loading condition – the appropriate rules in the section classification table are applied.

See: AISC 341-05 seismic classification - all members (page 63)

AISC 341-16, D1.4a Required strength

This check consists of two analyzes of the column - one that uses the 'amplified seismic load' and a second that uses a 'capacity analysis' as an upper bound. The moments and shears in the column are ignored and the column is designed for axial load only.

NOTE While applying the column strength requirements of D1.4a, it is assumed that there are no loads applied to the column between locations of lateral support. Therefore applied moments are ignored and only the axial strength is considered as permitted in AISC 341-10 D1.4a(2).

NOTE While applying the column strength requirements of D1.4a (2), the upper limit on the required strength with respect to overturning uplift as per D1.4a (2) (b) is not applied.

Overstrength seismic load

P_{amp}, the required axial strength (either tension or compression) including the “overstrength seismic load” is given by,

$P_{amp} = P_r + f_E/\rho * P_E * (\Omega_o - \rho)$

Where;

P_r = the axial force (-ve for tension and +ve for compression) determined from the analysis of the seismic load combination (LRFD or ASD). This may be the result of a first or second order analysis.

P_E = the axial force (-ve for tension and +ve for compression) determined from the analysis of the seismic loadcase(s) associated with the seismic load combination.

f_E = the strength load factor associated with the seismic load in the seismic load combination (base combination factor x ρ) (for example, 0.683 in the ASD combination, D + 0.75 L + 0.75 Lr + 0.683 E).

ρ = the redundancy factor, ρ_1 when the column is assigned to Direction 1 and r2 when the column is assigned to Direction 2 (from the Seismic Wizard).

Ω_o = the overstrength factor, Ω_{o1} when the column is assigned to Direction 1 and Ω_{o2} when the column is assigned to Direction 2 (from the **Seismic Wizard...**).

NOTE The axial force from the load combination including the amplified seismic loads is calculated by swapping out the component due to the seismic loadcase $E_h = \rho\ Q_E$ and replacing it with the amplified seismic load, $E_{mh} = \Omega_E\ Q_E$.

Design condition

For each stack, the required axial strength, P_r is compared with the nominal axial strength, P_n, i.e. the design condition is,

$P_r \leq \phi \times P_n$ (LRFD) or P_n/Ω (ASD)

Where

P_n = the nominal axial strength in tension or compression as appropriate to the sign of P_r

ϕ = the resistance factor for tension or compression as appropriate

Ω = the safety factor for tension or compression as appropriate

AISC 341-10, D1.4a Required strength

This check consists of two analyzes of the column - one that uses the 'amplified seismic load' and a second that uses a 'capacity analysis' as an upper bound. The moments and shears in the column are ignored and the column is designed for axial load only.

NOTE While applying the column strength requirements of D1.4a, it is assumed that there are no loads applied to the column between locations of lateral support. Therefore applied moments are ignored and only the axial strength is considered as permitted in AISC 341-10 D1.4a(2).

NOTE While applying the column strength requirements of D1.4a (2), the upper limit on the required strength with respect to overturning uplift as per D1.4a (2) (b) is not applied.

Amplified seismic load

P_{amp}, the required axial strength (either tension or compression) including the “amplified seismic load” is given by,

$P_{amp} = P_r + f_E/\rho * P_E * (\Omega_o - \rho)$

Where;

P_r = the axial force (-ve for tension and +ve for compression) determined from the analysis of the seismic load combination (LRFD or ASD). This may be the result of a first or second order analysis.

P_E = the axial force (-ve for tension and +ve for compression) determined from the analysis of the seismic loadcase(s) associated with the seismic load combination.

f_E = the strength load factor associated with the seismic load in the seismic load combination (base combination factor x ρ) (for example, 0.683 in the ASD combination, D + 0.75 L + 0.75 Lr + 0.683 E).

ρ = the redundancy factor, ρ_1 when the column is assigned to Direction 1 and r2 when the column is assigned to Direction 2 (from the Seismic Wizard).

Ω_o = the overstrength factor, Ω_{o1} when the column is assigned to Direction 1 and Ω_{o2} when the column is assigned to Direction 2 (from the **Seismic Wizard...**).

NOTE The axial force from the load combination including the amplified seismic loads is calculated by swapping out the component due to the seismic loadcase $E_h = \rho\ Q_E$ and replacing it with the amplified seismic load, $E_{mh} = \Omega_E\ Q_E$.

Capacity analysis

At any level on a column there can be SFRS members and non-SFRS members. The principle of this check is that the former might be operating at their “capacity” in an earthquake and so they are likely to apply more force to the column than the global analysis would indicate. “Capacity” in this context also includes the possibility that the material is stronger than its specified yield (typical).

The capacity calculation involves establishing the capacities of the incoming SFRS members at each node (level) in the column – these might be zero if there are only non-SFRS members at that level. The capacities so determined are then resolved into the local x-axis of the column. The capacities are calculated for beams and braces only, not the columns themselves.

The end result of the capacity analysis is that for each stack there is an axial force, P_{cap} which can be compression or tension. This will be included in the results and used in this design check to D1.4a but will only govern if smaller than that from the “Amplified seismic load” analysis.

Design condition

For each stack, the required axial strength, P_r (the smaller of P_{cap} or P_{amp}) is compared with the nominal axial strength, P_n, i.e. the design condition is,

$P_r \leq \phi \times P_n$ (LRFD) or P_n/Ω (ASD)

Where

P_n = the nominal axial strength in tension or compression as appropriate to the sign of P_r

ϕ = the resistance factor for tension or compression as appropriate

Ω = the safety factor for tension or compression as appropriate

AISC 341-16 and -10, E3.4a Moment ratio

At each level in a column where an SMF beam connects into the strong axis of the column (i.e. into the flange), a check is performed for each seismic combination to ensure that the plastic moment capacity of the column is greater than the plastic moment capacity of the incoming beams.

The design condition is,

$\sum M_{pcol} / \sum M_{pbeam} > 1.0$

NOTE The exceptions in E3.4a (a) and (b) are ignored and the check is performed for all SMFs.

NOTE Beam column connections are always assumed braced as per E3.4c (1)

NOTE All beams with pinned connections are excluded in this calculation. Any beam with a moment connection into the web of the column is ignored even if they are assigned to a SMF. On the other hand, any beam with a moment connection to the column flange is included in the calculation even if they are not assigned to a SMF.

NOTE The additional moment due to shear amplification from the location of the plastic hinge to the column centre line (Muv and Mav) is calculated from two components,

1. the shear inferred by the moment at the plastic hinge position based on the expected flexural strength of the beam,
2. the shear force in the beam at the plastic hinge position from the factored gravity loads in the current seismic combination.

No account of angle of incoming members is taken into account in this calculation.)

AISC 341-16 and -10, F2.3 Analysis

This check applies to SCBF type frames only, the procedure being similar to that for 'Column Strength' to D1.4a.

NOTE The approach taken to the “capacity analysis” per F2.3 assumes that the SCBF is reasonably isolated. That is, the influence of the remainder of the structure due to the braces operating at their capacity does not adversely affect the required strength.

Design condition

Moments in the column are permitted to be ignored as per F2.3 (1) and so only the axial check (compression or tension) is required.

For each stack, the required axial strength, P_r, is compared with the nominal axial strength, P_n, i.e. the design condition is,

$P_r \leq \phi \times P_n$ (LRFD) or P_n/Ω (ASD)

Where P_r = the required axial strength, P_{cap}

P_n = the nominal axial strength in tension or compression as appropriate to the sign of P_r

ϕ = the resistance factor for tension or compression as appropriate

Ω = the safety factor for tension or compression as appropriate

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NOTE While applying the column strength requirements of F2.3 (i) and (ii), it is assumed that there are no loads applied to the column between locations of lateral support. Therefore applied moments are ignored and only the axial strength is considered as permitted in AISC 341-10 F2.3 (1).

NOTE While applying the column strength requirements of F2.3 (i) and (ii), the upper limits on the required strength per F2.3 (2) (a), (b) and (c) are not applied.

NOTE It is assumed that braces do not carry significant gravity forces and therefore a separate analysis with braces omitted, in order to enhance the column gravity forces, is not carried out. [Ref. NEHRP Seismic Design Technical Brief No. 8].

AISC 341-05, 8.3 Required strength

The calculations for this check are exactly the same as those for the AISC 341-10, D1.4a Required Strength check except that they are only performed when the required axial force exceeds a certain limit as described below.

$P_r > 0.4 \times \phi_c \times P_n$ (LRFD)

$P_r > 0.4 \times P_n / \Omega_c$ (ASD)

Where

$\phi_c = 0.90$

$\Omega_c = 1.67$

P_n = the nominal axial strength of the stack in compression or tension as appropriate to the sign of P_r

Either,

$P_r = P_u$

= the maximum axial force in the stack from the current (LRFD) seismic combination

Or,

$P_r = P_a$

= the maximum axial force in the stack from the current (ASD) seismic combination

NOTE While applying the column strength requirements of 8.3, the upper limit on the required strength with respect to overturning uplift as per 8.3 (2) (b) is not applied.

AISC 341-05, 9.6 Moment ratio

At each level in a column where an SMF beam connects into the strong axis of the column (i.e. into the flange), a check is made to ensure that for each seismic combination the plastic moment capacity of the column is greater than the plastic moment capacity of the incoming beams. The calculations for this check are exactly the same as those for the AISC 341-10, E3.4a Moment Ratio check.

NOTE The exceptions in 9.6 (a) and (b) are ignored and the check is performed for all SMFs.

NOTE All beams with pinned connections are excluded in this calculation. Any beam with a moment connection into the web of the column is ignored even if they are assigned to a SMF. On the other hand, any beam with a moment connection to the column flange is included in the calculation even if they are not assigned to a SMF.

NOTE The additional moment due to shear amplification from the location of the plastic hinge to the column centre line (Muv and Mav) is calculated from two components,

(i) the shear inferred by the moment at the plastic hinge position based on the expected flexural strength of the beam,

(ii) the shear force in the beam at the plastic hinge position from the factored gravity loads in the current seismic combination. No account of angle of incoming members is taken into account in this calculation.

Seismic checks - Braces (Seismic: AISC 341)

Classification

In all cases if the given “width to thickness ratio” is less than or equal to the given limit, then the seismic classification is satisfied.

AISC 341-16 and AISC 341-10

Braces in OCBF – As per Clause F1.5a, braces must satisfy the requirements of clause D1.1b for “moderately ductile” members.

Braces in SCBF – As per Clause F2.5a, braces must satisfy the requirements of clause D1.1b for “highly ductile” members.

See: AISC 341-16 and AISC 341-10 seismic classification - all members (page 61)

AISC 341-05

Braces in OCBF – As per Clause 14.2, braces must satisfy the requirements of clause 8.2b for 'seismically compact' members.

Braces in SCBF – As per Clause 13.2d, braces must satisfy clause 8.2b for “seismically compact” sections.

See: AISC 341-05 seismic classification - all members (page 63)

Slenderness

AISC 341-16 and AISC 341-10

OCBF

In OCBF for V and A braces only, the design condition is checked for both major and minor axis as per F1.5b,

$KL/r \leq 4 * SQRT[E/F_y]$

Where

K = the effective length factor for the relevant axis

L = the system length of the brace

r = the radius of gyration of the brace for the relevant direction

E = modulus of elasticity of steel – 29000 ksi

F_y = minimum yield stress.

SCBF

For all braces in SCBF the design condition for both minor and major axis is checked as per F2.5b (1),

$KL/r \leq 200$

Where

K = the effective length factor for the relevant axis

L = the system length of the brace

r = the radius of gyration of the brace for the relevant direction.

For built-up braces i.e. double angles the requirements for interconnection are checked as per F2.5b (2). The minimum number of connectors required by this clause is two and thus the maximum interconnection slenderness of the individual angles is based on a buckling length of one third of the system length, (which is conservative).

Thus,

$a/r_i \leq 0.4 * MAX[KL/r]$

Where

a = the sub-length of the member between interconnections = taken as L/3

r_i = the minimum radius of gyration of the individual angle, taken as r_z

NOTE While checking the minimum slenderness of individual elements in built-up members to F2.5b (2), It is assumed the minimum number of shear connectors is provided i.e. two. The shear strength of the connectors is NOT checked against the tensile strength of each element.

NOTE The brace net area is NOT checked against the brace gross area as per F2.5b (3) and where this might be an issue suitable reinforcement is assumed to be provided.

AISC 341-05

OCBF

For V and A braces in OCBF the design condition for both minor and major axis is checked as per 14.2,

$KL/r \leq 4 * SQRT[E/F_y]$

Where

K = the effective length factor for the relevant axis

L = the system length of the brace

r = the radius of gyration of the brace for the relevant direction.

SCBF

For all braces in SCBF there is a three stage design condition and both minor and major axis are checked as per 13.2a,

$KL/r \leq 4 * SQRT[E/F_y]$ PASS

$KL/r > 200$ FAIL

ELSE WARNING

“Brace slenderness satisfies, $4\sqrt{(E/F_y)} < KL/r \leq 200$. The available strength of the associated column is NOT checked as per 13.2a."

Where all variables are as given above.

For built-up braces i.e. double angles the requirements for interconnection are checked as per 13.2e. The minimum number of connectors required by this clause is two and thus the maximum interconnection slenderness of the individual angles is based on a buckling length of one third of the system length, (this will be conservative). Thus,

$a/r_i \leq 0.4 * MAX[KL/r]$

Where

a = the sub-length of the member between interconnections = taken as L/3

r_i = the minimum radius of gyration of the individual angle, taken as r_z

---

NOTE While checking the minimum slenderness of individual elements in built-up members to 13.2e, it is assumed the minimum number of shear connectors is provided i.e. two. The shear strength of the connectors is NOT checked against the tensile strength of each element.

NOTE The brace net area is NOT checked against the brace gross area and where this might be an issue suitable reinforcement is assumed to be provided.

Brace strength

AISC 341-16 and AISC 341-10

OCBF

No additional requirements.

SCBF

Where the effective net area is less than the gross area the provisions of F2.5b (3) apply. This is more aimed at gusset plate connections where the cross section of the brace is reduced. The effective net area is specified by the user as a percentage or actual area.

The design condition should be (!),

$\phi_t * F_u * A_e \geq R_y * F_y * A_g$ LRFD

$F_u * A_e / \Omega_t \geq R_y * F_y * A_g / 1.5$ ASD

Where,

ϕ_t = resistance factor for tension

Ω_t = safety factor for tension

F_u = specified minimum tensile strength of steel

F_y = specified minimum yields stress of steel

A_e = effective area of brace (user input)

A_g = gross area of brace

R_y = the overstrength factor – see Section.

Note that for 50 ksi steel this will always fail but providing there is no reduction in area the brace is expected to yield. The Commentary in AISC 341 Comm. F2.5b indicates:

“Where there is no reduction in the section, or where the section is reinforced so that the effective net section is at least as great as the brace gross section, this requirement does not apply. The purpose of the requirement is to prevent net section fracture prior to significant ductility; having no reduction in the section is deemed sufficient to ensure this behavior.”

Consequently the design condition in Tekla Structural Designer is presented as follows, and considers the effective net area provided, $A_{e.prov}$, and the effective net area required, $A_{e.reqd}$, to satisfy F2.5b (3),

$A_{e.reqd} = MAX[A_g, (R_y * F_y * A_g/(F_u * \phi_t)]$ LRFD

$A_{e.reqd} = MAX[A_g, (R_y * F_y A_g * \Omega_t/(F_u * 1.5)]$ ASD

The design condition then becomes,

$A_{e.reqd} \leq A_{e.prov}$

AISC 341-05

OCBF

No additional requirements.

SCBF

The calculations for this check are exactly the same as those for the AISC 341-10 check.

NOTE The brace required strength to 13.2b is NOT limited to the "maximum load effect" as per 13.2b (b).

1.4 Concrete design to ACI 318

Setting the Tekla Structural Designer head code to United States (ACI/AISC) sets the Concrete Design Resistance Code to ACI 318. The code year can set to 2008, 2011, 2014, or 2019.

Design is then performed in accordance with the ACI 318 if using US Customary units, or ACI 318M if using metric units.(Ref. 1-8) (page 201)

The following topics are covered:

NOTE Unless explicitly noted otherwise, all clauses, figures and tables used in the Reference Guides are from ACI 318-11.

ACI 318-19 updates

The following updates to gravity design have been made when working to the 2019 version of ACI 318.

- Concrete column design (ACI 318-19 update) (page 83)
- Concrete wall design (ACI 318-19 update) (page 83)
- Slab and mat design (ACI 318-19 update) (page 83)
- Punching checks (ACI 318-19 update) (page 83)
- Pad bases, strip footings & pile caps (ACI 318-19 update) (page 84)
- Beam-column joint shear strength (ACI 318-19 update) (page 84)

The following updates to gravity design have been made when working to the 2019 version of ACI 318.

- Seismic design - general (ACI 318-19 update) (page 86)
- Concrete beam seismic design (ACI 318-19 update) (page 87)
- Concrete column seismic design (ACI 318-19 update) (page 87)
- Concrete wall seismic design (ACI 318-19 update) (page 87)

Gravity design - general (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Concrete design to ACI 318 (page 81) checks are amended as follows:

- Clause 20.2.2.4 - Revision to reinforcement strength - Limited allowance of 100 ksi reinforcement so added grade of 100 ksi for reinforcement.
- Reinforcement yield strength (f_y) shall be limited to a maximum of 100 ksi in the calculation of all members for non-seismic design.
- As per Table 21.2.2 - Revision of strain limit from 0.004 to ϵ_{ty} + 0.003 for tension controlled section. Updated changes wherever required in the calculation of flexure checks.
- Clause 19.2.1.1 - Added minimum compressive strength (f_{ck}) check.

Concrete beam design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Concrete beam design to ACI 318 (page 89) checks are amended as follows:

- Clause 9.6.1.2 - The value of fy is limited to a maximum of 80 ksi in the calculation of minimum area of longitudinal reinforcement.
- Clause 9.7.6.2.2- Addition of maximum stirrup spacing across beam width.
- Clause 24.2.3.5- Updated equation for calculation of effective moment of inertia in deflection check.

Concrete column design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Concrete column and wall design to ACI 318 (page 130) checks are amended as follows:

- Clause 22.4.2.1 - The value of f_y is limited to a maximum of 80 ksi in the calculation of ultimate compressive strength.
- Clause 22.5.5.1 - Updated shear design as per revised shear equations.
- Clause 22.5.1.10 - Added new biaxial shear check.

Concrete wall design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Concrete column and wall design to ACI 318 (page 130) checks are amended as follows:

- Clause 22.4.2.1 - The value of f_y is limited to a maximum of 80 ksi in the calculation of ultimate compressive strength.
- Clause 11.6.1 & 11.6.2 - Equations updated for unreinforced shear limitation and minimum reinforcement.
- Clause 11.5.4.3 - Updated in-plane shear strength equations.
- Clause 22.5.5.1 - Updated out-of-plane shear design as per revised shear equation.

Slab and mat design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Concrete slab design to ACI 318 (page 185) checks are amended as follows:

- As per Clause 8.6.1.1 - updated calculation for minimum area of flexural reinforcement.
- As per clause 8.3.1, 8.3.1.1 & R8.3.1.1 ,8.3.1.2 - updated changes in span to effective depth considerations.

Punching checks (ACI 318-19 update)

Summary

When working to the 2019 version of the code the punching checks to ACI 318 checks are amended as follows:

- ACI 318-19 Section 22.6.4.3, Figure R22.6.4.3 - Modification of critical perimeters to take account of slab openings - Limit change from 10 h to 4h.
- Clause 22.6.5.2 - A new size modification factor λ_s has been considered in concrete shear strength calculation.

Pad bases, strip footings & pile caps (ACI 318-19 update)

Summary

When working to the 2019 version of the code the pad base, strip footing (page 185) and pile cap (page 195) ACI-318 checks are amended as follows:

- Clause 8.6.1.1 - Updated calculations for minimum area of flexural reinforcement.
- Clause 22.5.5.1 - Updated shear design as per revised shear equations.
- Clause 22.6.5.2 - A new size modification factor λ_s has been considered in concrete shear strength calculation.

Beam-column joint shear strength (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Concrete column and wall design to ACI 318 (page 130) checks are amended as follows:

- As per clause 15.4 added joint shear strength check for gravity load conditions in line with the seismic design.

Joint shear strength check

- A new node of "Joint Shear Strength" is added to the column design results tree under Shear Links
- The strength reduction factor φ is taken as 0.75
- Overstrength factor, η taken as 1.0
- Probable moment strength is changed to nominal moment strength

NOTE Beam nominal strength is based on the rectangular section and slab reinforcement is not considered.

The **γ** factor used in computing the joint shear strength is adjusted as per table 15.4.2.3,

Column	**Beam in the direction of V_u**	**Confinement (Consider Transverse Beam)**	**γ**
Column is continuous at joint (b) AND length of stack above ≥ column depth in direction of shear (a)	Beam is continuous at joint AND length of beam on both sides ≥ beam depth	Transverse Beams covering more than or equal than ¾ of the column face and Length of transverse beam ≥ transverse Beam Depth	2.0 (24)
		Transverse Beams not covering more than or equal than ¾ of the column face OR Length of transverse beam < transverse Beam Depth	1.7 (20)
	Beam is not continuous at joint OR length of beam on either side < beam depth	Transverse Beams covering more than or equal than ¾ of the column face and Length of transverse beam ≥ transverse Beam Depth	1.7 (20)
		Transverse Beams not covering more than or equal than ¾ of the column faceOR Length of transverse beam < transverse Beam Depth	1.3 (15)
Column is not continuous at joint (b) OR length of stack above <	Beam is continuous at joint AND length of beam on both	Transverse Beams covering more than or equal than ¾ of the	1.7 (20)

Column	Beam in the direction of V_u	Confinement (Consider Transverse Beam)	γ
column depth in direction of shear (a)	sides ≥ beam depth	column face and Length of transverse beam ≥ transverse Beam Depth	
		Transverse Beams not covering more than or equal than ¾ of the column face OR Length of transverse beam < transverse Beam Depth	1.3 (15)
	Beam is not continuous at joint OR length of beam on either side < beam depth	Transverse Beams covering more than or equal than ¾ of the column face, and Length of transverse beam ≥ transverse Beam Depth	1.3 (15)
		Transverse Beams not covering more than or equal than ¾ of the column face OR Length of transverse beam < transverse Beam Depth	1.0 (12)

Seismic design - general (ACI 318-19 update)

Summary

When working to the 2019 version of the code the Seismic Design to ACI 318 (page 199) checks are amended as follows:

- Clause 20.2.2.4 - Revision to reinforcement strength
- Clause 19.2.1.1 - Updated minimum compressive strength (f_{ck}) check

- Clause 20.2.2.4 -Revision to reinforcement yield strength limit in design calculation

Concrete beam seismic design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the ACI 318 seismic checks are amended as follows:

- Clause 18.6.3.1 - Changes in calculation of maximum area of flexural reinforcement for special moment frames (SMF)
- Clause 18.8.2.3 - Changes in check for maximum allowable bar diameter of flexural reinforcement for SMF
- Clause 18.6.4.4 - Changes in check for maximum allowable hoop spacing for SMF

Concrete column seismic design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the ACI 318 seismic checks are amended as follows:

- Clause 18.7.3.1 - Modified check of minimum flexural strength
- Clause 18.8.4.3 - Modified check of joint shear strength for SMF
- Clauses 18.7.5.3, 18.8.3.1, 18.7.5.5 & 18.4.3.3 - Changes in check for maximum allowable hoop spacing in support region and span region for SFRS members

Concrete wall seismic design (ACI 318-19 update)

Summary

When working to the 2019 version of the code the ACI 318 seismic checks are amended as follows:

- Clause 20.2.2.4 -Revision to reinforcement yield strength limit in design calculation.
- Clause 19.2.1.1 - Updated minimum compressive strength (f_{ck}) check.
- Clause 18.10.3.1 - Changes in shear strength design.
- Clauses 18.10.6.4(e), 18.10.6.5 - Changes in check for maximum allowable hoop spacing in boundary confinement for SFRS wall.

Limitations (concrete members: ACI 318)

The following general exclusions apply.

the current release will not:

- design members in lightweight concrete
- design members with coated reinforcement
- design members with stainless steel
- design prestressed concrete
- Consider 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],
- design structures subject to very aggressive exposure
- design watertight structures
- design multi-stack reinforcement lifts for columns/walls
- design beams as “deep beams” - beams classified as “deep” are designed as if they are regular beams and a warning is displayed.

NOTE Deep beams according to ACI 318 are:

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

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

material limitations for concrete:

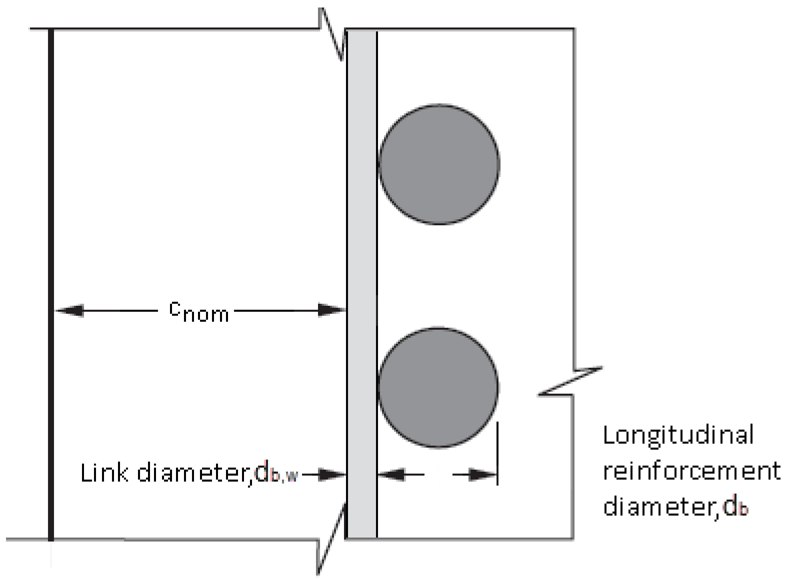
- for structural concrete compressive strength of concrete f_c' shall not be less than 17 MPa (2500psi)
- durability requirements are not implemented

material limitations for reinforcement:

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

Cover to Reinforcement (ACI 318)

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



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

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

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

Concrete beam design to ACI 318

Click the links below to find out more about the application of ACI 318 with regard to:

- Slender beams (page 91)
- Design parameters for longitudinal bars (page 91)
- Side skin reinforcement in beams (page 95)
- Effective depth of section (page 95)
- Design for bending for rectangular sections (page 96)
- Design for bending for flanged sections (page 99)
- Shear strength (page 100)

If working to **ACI 318-19** click the below link to view additional important information:

- Concrete beam design (ACI 318-19 update) (page 82)

See also

Limitations (concrete members: ACI 318) (page 87)

Cover to Reinforcement (ACI 318) (page 88)

Seismic Design to ACI 318 (page 199)

Limitations (concrete beam: ACI 318)

The following general exclusions apply.

the current release will not:

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

> **NOTE** Deep beams according to ACI 318 are:
>
> (a) Members with clear spans equal to or less than 4 times overall member depth
>
> (b) Members with concentrated loads within twice the member depth from the support

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

material limitations for concrete:

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

material limitations for reinforcement:

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

Slender beams (ACI 318)

Spacing of lateral supports for a beam shall not exceed 50*b[1]

In the program the lateral supports are taken as the distance between the faces of the supports, and for simplification, b is taken as the web width b_w

If the above check fails then a 'Slender span' warning is displayed.

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

Design parameters for longitudinal bars (ACI 318)

For each of these parameters, any user defined limits (as specified on the appropriate Reinforcement Settings page within Design Options) are considered in addition to any ACI code recommendations.

Minimum and maximum diameter of reinforcement

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

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

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

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

For primary reinforcement there is no limit on bar size.

Minimum distance between bars

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

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

[1] ACI 318-08 and ACI 318-11 Section 10.4

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

IF the above check fails then a Warning is displayed.

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

Maximum spacing of tension bars

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

s	≤	MIN[380mm*280 MPa/fs-2.5*c_c, 300mm*(280MPa/ f_s)]	metric-units
s	≤	MIN[15in*40000psi/f_s-2.5*c_c, 12in*(40000psi/ f_s)]	US-units

where

c_c	=	the least distance from surface of reinforcement to the tension face
f_s	=	calculated stress in reinforcement at service load; it shall be permitted to take
	=	$(2/3)*f_y*(A_{s,reqd}/A_{s,prov})$

IF the above check fails then a Warning is displayed

IF torsional reinforcement is required:

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

[1] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.6.4

[2] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.5.6.2

Minimum area of beam reinforcement

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

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

$A_{s,min} \geq MAX[(3* f'_c{}^{0.5}/(f_y)*b_w*d, 200Psi*b_w*d/ f_y]$ US-units

where

f'_c	=	specified compressive strength of concrete
f_y	=	specified yield strength of reinforcement
b_w	=	web width; for statically determinate members with a flange in tension $b_w = MIN(2*b_w, b_{eff})$ [1]
d	=	distance from extreme compression fiber to centroid of longitudinal compression reinforcement

[1]: Assumption; the member is statically determinate in design

The above equation is used wherever reinforcement is needed, except where such reinforcement is at least one-third greater than that required by analysis, in which case $A_{s,min}$ not required.

Minimum area of slab reinforcement

For ACI 318-08 and ACI 318-11[5]

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

For US-units:

IF Grade 40 to 50 deformed bars are used

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

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

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

For metric units:

IF Grade 280 to 350 deformed bars are used

[3] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.5.1

[5] ACI 318-08 and ACI 318-11 and ACI 318M-11 Section 7.12.2.1 and 10.5.4

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

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

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

IF yield stress exceeding 420 MPa

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

Maximum area of reinforcement

Net tensile strain in extreme layer of longitudinal tension steel, ε_t should not be less than 0.004;

ε_t		0.004 ≥
$A_{s,max}$		$0.85*(f'_c / f_y)*\beta_1*b_w*d*[0.003/(0.003+0.004)]$ [1] ≤
		$0.85*(f'_c / f_y)*\beta_1*b_w*d*(3/7)$ ≤

[1]: Notes on ACI 318-08 Chap. 6 Section 10.3.5

where

A_g	=	the gross area of the concrete section	
β_1	=	stress block depth factor [1]	
metric units			
	=	0.85	for $f`_c \leq 28MPa$
	=	$0.85 - 0.05 *[(f'_c - 28MPa)/7MPa]$	for $28MPa < f`_c < 55MPa$
	=	0.65	for $f`_c \geq 55MPa$
US-units			

[1]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.2.7.3

	=	0.85	for $f`_c \le 4000$ psi
	=	0.85 - 0.05 *[(f'_c - 4ksi)/1ksi]	for 4000 psi < $f`_c$ < 8000psi
	=	0.65	for $f`_c \ge 8000$ psi

[1]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.2.7.3

Side skin reinforcement in beams (ACI 318)

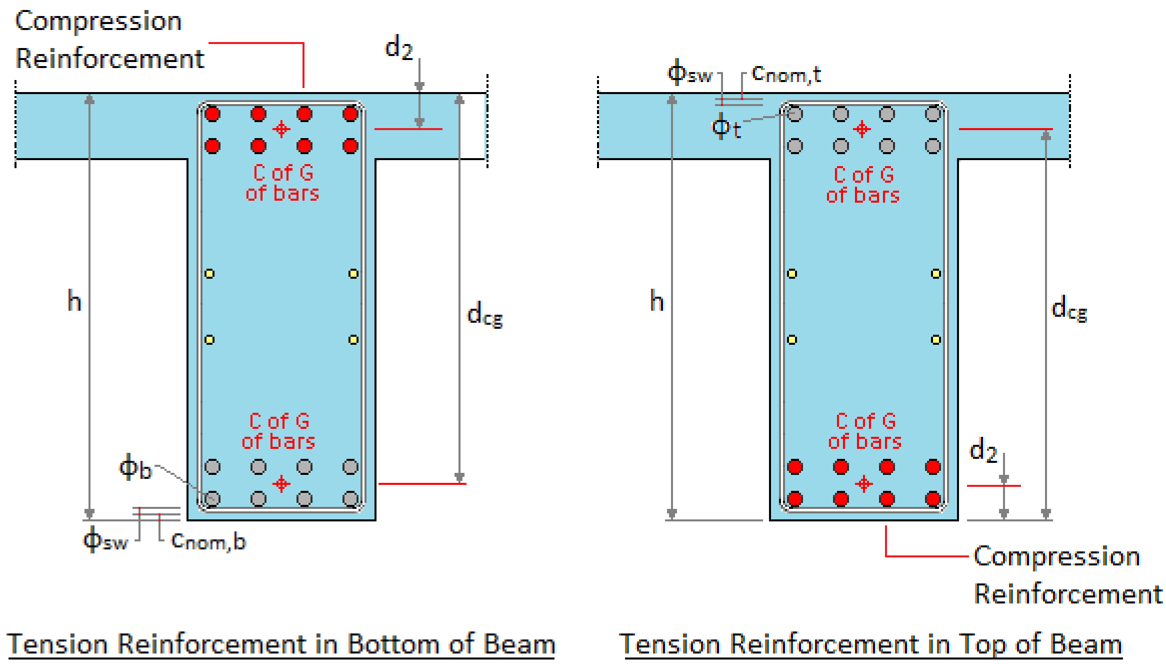
Where h of a beam or joist exceeds 900mm (36 in.), longitudinal skin (side) reinforcement is uniformly distributed along both side faces of the member.

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

Effective depth of section (concrete beam: ACI 318)

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

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



Design for bending for rectangular sections (beams and slabs: ACI 318)

Determine if compression reinforcement is needed

Nominal strength coefficient of resistance is given;[1]

R_n	=	M_u / (φ*b*d_2)
where		
M_u	=	factored moment at section
d	=	depth to tension reinforcement
b	=	width of the compression face of the member
φ	=	strength reduction factor[1]
	=	0.9 (corresponds to the tension-controlled limit)

[1]: ACI 318-08 and ACI 318-11 Section 9.3

[1] Notes on ACI 318-08 Chap 7.

IF R_n	≤	R_{nt}	THEN compression reinforcement is not required.
IF R_n	>	R_{nt}	THEN compression reinforcement is required.

where			
R_{nt}	=	Limit value for tension controlled sections without compression reinforcement for different concrete strength classes[1]	
	=	ωt *(1-0.59 ωt)* $f`_c$	
$f`_c$	=	compressive strength of concrete	
ω_t	=	0.319*b1	
β_1	=	stress block depth factor [2]	
metric units			
	=	0.85	for $f`_c \leq 28MPa$
	=	0.85 - 0.05 *[(f'$_c$ - 28MPa)/7MPa]	for 28MPa < $f`_c$ < 55MPa
	=	0.65	for $f`_c \geq 55MPa$
US-units			
	=	0.85	for $f`_c \leq 4000$ psi

[1]: Notes on ACI 318-08 Section 10.3.4

[2]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.2.7.3

	=	0.85 - 0.05 *[(f'$_c$ - 4ksi)/1ksi]	for 4000 psi < f`$_c$ < 8000psi
	=	0.65	for f`$_c$ ≥ 8000 psi

[1]: Notes on ACI 318-08 Section 10.3.4
[2]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.2.7.3

Compression reinforcement is not required

The tension reinforcement ratio is given by;

$\rho = 0.85 * f`_c / f_y * [1-(1- 2 * R_n / \sqrt{(0.85 * f`_c)})] \leq \rho t = 0.319 * b_1 * f`_c / f_y$

where

f_y = yield strength of reinforcement

The area of tension reinforcement required is then given by;

$A_s = \rho * b * d$

The area of compression reinforcement required is then given by;

$A_s` = M_n`/[(d-d`) * f_s`]$

where

$M_n` = M_n - M_{nt}$

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

M_{nt} = nominal moment resisted by the concrete section[5]

$= R_n * b * d_2$

The area of tension reinforcement required is then given by[6];

$A_s = A_s` * f_s`/f_y + \rho * b * d$

where

$f_s` = MIN[E_s * (\varepsilon_u * (c-d`)/c) , f_y]$

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

$\rho_t = 0.319 * b_1 * f`_c / f_y$

$\varepsilon_u = 0.003$[7]

$c = 0.375 * d_t$

[5] Notes on ACI 318-08 Section 10.3.4

[6] Notes on ACI 318-08 Chap. 7

[7] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.2.3

Design for bending for flanged sections (beams: ACI 318)

IF $h_f < 0.5*b_w$ THEN treat the beam as rectangular[1]

where

b_w = web width

Depth of the equivalent stress block is given [2];

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

where

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

$R_n = (M_u/\varphi) / (b_{eff}*d_2)$ assumption $\varphi = 0.9$

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

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

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

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

where

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

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

Required reinforcement is given;

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

Nominal moment strength of **<u>flange</u>**;

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

Required nominal moment strength to be carried by the beam **<u>web</u>** is given;

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

Can be written as;

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

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

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[1] ACI 318-08 and ACI 318-11 Section 8.12.4

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

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

where

$\omega_w = \rho * f_y / f`_c = 0.85 * f`_c / f_y * [1-(1- 2*(M_{nw} / (b*d_2))/\sqrt{(0.85*f`_c)}] * f_y / fc`$

Can be written as;

$A_{sw} = b*d*0.85*f`_c/f_y*[1-(1- 2*(M_{nw} / (b*d_2))/\sqrt{(0.85*f`_c)}]$

Total required reinforcement is given;

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

Check to see if the section is tension-controlled;

IF

$\rho_w \le \rho_t$ section is tension- controlled (φ=0.9)

ELSE add compression reinforcement where

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

Can be simplified as;

$\omega_w \le 0.319 * \beta_1$ section is tension-controlled (φ=0.9)

ELSE add compression reinforcement

Shear strength (beams: ACI 318)

Determine shear strength provided by the concrete[1];

Members subject to axial compression not applied at this stage

φV_c		$\varphi*0.17*\lambda*\sqrt{(f`_c)}*b_w*d$ =	metric-units
		$\varphi*2*\lambda*\sqrt{(f`_c)}*b_w*d$ =	US-units
where			

[1]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 9.3.2.3
[2]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 8.6.1
[3]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.1.2.1

[1] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.2.1.1

φ		0.75 for shear[1] =
λ		1.0 for normal weight concrete[2] =
$f`_c^{0.5}$		square root of specified compressive strength of concrete[3] =

[1]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 9.3.2.3
[2]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 8.6.1
[3]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.1.2.1

NOTE If the structure is defined as a joist construction V_c shall be permitted to be 10% more than that specified in above[5].

IF

V_u - $φV_c$		$φ*0.66*\sqrt{(f`_c)}*b_w*d$[1] ≤	metric-units
		$φ*8*\sqrt{(f`_c)}*b_w*d$ =	US-units

[1]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.7.9

where

V_u		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 or strength of the concrete is inadequate for shear. No further shear calculations are carried out in the region under consideration and the user is warned accordingly.

[5] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.1.2.1

The design shear capacity of the minimum area of shear links actually provided, $V_{s,min}$ is given by[7];

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

where

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

For each beam determine the following;

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

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

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

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

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

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

where

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

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

In any region, *i*;

IF

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

where

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

OR

The structure is defined as a joist construction[8].

THEN

Minimum shear reinforcement shall be used;

And the nominal shear strength is given;

$\varphi V_n = \varphi V_c + V_{s,min}$ [9]

ELSE

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[7] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.7.2

[8] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.6.1 - Terms (d) and (e) not applied.

[9] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.1

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

THEN shear links are required in the region.

The area of shear reinforcement required is then given[10];

metric-units;

$(A_v /s)_{Si}$ = MAX[$(V_u - \varphi * V_c)/ (\varphi * f_{yt} * d)$, 0.062* f`$_c$0.5 *$b_w$ /f_{yt}, 0.35Pa*b_w/f_{yt}]

US-units;

((A_v /s)$_{Si}$ = MAX[$(V_u - \varphi * V_c)/ (\varphi * f_{yt} * d)$, 0.75* f`$_c$0.5 *$b_w$ /f_{yt}, 50psi*b_w/f_{yt}]

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

IF

$Vs \leq 0.66 * f'_c{}^{0.5} * b_w * d$[11] (metric -units) $8 * f'_c{}^{0.5} * b_w * d$ (US-units)

THEN the shear design process passes.

And the nominal shear strength is given;

$\varphi V_n = \varphi V_c + V_s$

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

Minimum area of shear reinforcement (beams: ACI 318)

The minimum area of shear reinforcement required, Av,min is given by[1];

	$A_{v,min}$		MAX (0.062* f`$_c{}^{0.5}$ *b_w*s/ f_{yt} , 0.35MPa*b_w*s/ f_{yt}, $A_{v,min,u}$) =	metric-units
			MAX (0.75* f`$_c{}^{0.5}$ *b_w*s/ f_{yt} , 50psi*b_w*s/ f_{yt}, $A_{v,min,u}$) =	US-units
	where			
	s		the spacing of the shear reinforcement along the longitudinal axis of the beam	
	f_{yt}		yield strength of transverse reinforcement =	

[10] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.6.3

[11] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.7.9

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

Spacing of shear reinforcement (beams: ACI 318)

For Longitudinal spacing, *s* between the legs of shear reinforcement is given by[1];

IF

V_u - φ*V_c ≤ φ*0.33* $f`_c^{0.5}$ *b_w*d metric-units

φ*4* $f`_c^{0.5}$ *b_w*d US-units

THEN

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

ELSE

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

where

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

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

Moreover IF **compression reinforcement** is required the compression reinforcement shall be enclosed by ties[2]. This is an additional limit, not an alternative.

Vertical spacing of ties is then given by[3];

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

where

d_b = the nominal diameter of the bar

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

NOTE Unlike other design codes, ACI 318 does not specify a limit for maximum spacing of link legs across a beam. However, attention is

[1] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.6.3

[1] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 11.4.5

[2] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 7.11.1

[3] ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 7.10.5.2

drawn to an ACI Structural Journal Technical Paper - “Shear Reinforcement Spacing in Wide Members”, which suggests a limit of around "d".

Deflection check (beams: ACI 318)

Deflection checks are divided between two deflection types: Immediate short-term deflections and long-term deflections which are resulting from creep and shrinkage of flexural members.

Two methods are given for controlling deflections:

1. By limiting span to depth ratio

For beams provision of a minimum overall thickness (min. total depth) as required by the following table satisfies the requirements of the code for members not supporting or attached to partitions or other construction likely be damaged by large deflections.

Support Conditions	**Minimum thickness, h_t**
Simply Supported	$l_n/16$
One end continuous	$l_n/18.5$
Both ends continuous	$l_n/2126$
Cantilever	$l_n/8$

If h		h_{min}	the design passes and no further calculations are required ≥

where

h		overall height of member =
h_{min}		$h_t * f_{y,mod}$ =
h_t		minimum thickness from above table =
l_n		clear span length =

$f_{y,mod}$		$0.4 + f_y$ / 700 MPa =	metric-units
		$0.4 + f_y$ / 100000 psi =	US-units

If the deflection check fails the rigorous method below is used.

2. By calculating deflections using the rigorous method

For beams that do not meet minimum thickness requirements above, or that support or are attached to partitions or other constructions likely be damaged by large deflections, deflections are calculated by following method.

1. Firstly, the beam’s cracked section moment of inertia, Icr is calculated.
2. Then the cracking moment Mcr is calculated.
3. The Long Term Deflection Period is read from the user specified value in Design settings. - 3 months to 5 years (default value: 5 years).
4. The Time at which brittle finishes are introduced is read from the user specified value in Design settings. - 1 month to 6 month (default value: 1 month).
5. For each loadcase with type = "dead", the % of load applied prior to sensitive finishes is read from the user specified value in the loadcase dialog (default value: 50%).
6. For each loadcase with type = “imposed”, the % of load which is long term is read from the user specified value in the loadcase dialog (default value: 33%).
7. For each span in the element the critical gravity combination is determined from the analysis. The combination reporting the max relative deflection is the one considered in the deflection check.
8. The maximum deflections for the different situations below can then be determined:
    - Dead load deflection $(\Delta_i)_d$
    - Dead and live load deflection $(\Delta_i)_{dlive}$
    - Live load deflection $(\Delta_i)_{live}$
    - Sustained load deflection $(\Delta_i)_{sus}$
    - Total load deflection$(\Delta_i)_{tot}$

- Deflection affecting sensitive finishes $(\Delta_i)_{af}$

The check passes if the calculated deflections are less than the deflection limits specified in the beam properties.

Seismic design and detailing (beams: ACI 318)

For overall limitations and assumptions, see:

- Limitations (beams seismic: ACI 318) (page 107)

For beam design in moment resisting frames, see:

- General requirements (beams seismic: ACI 318) (page 119)
- Flexural requirements (beams seismic: ACI 318) (page 122)
- Transverse reinforcement (beams seismic: ACI 318) (page 124)

For beams not part of a SFRS, see:

- Requirements when in SDC D - F (beams seismic: ACI 318) (page 128)
- Seismic cantilevers (beams seismic: ACI 318) (page 128)

For seismic detailing, see:

- Flexural reinforcement (beams seismic: ACI 318) (page 129)
- Confinement reinforcement for ductility (beams seismic: ACI 318) (page 129)

See also

Seismic Design to ACI 318 (page 199)

Limitations (beams seismic: ACI 318)

The following limitations and assumptions apply.

- Seismic design is only performed for beams marked as part of a Seismic Force Resisting System and for seismic cantilevers.
- Requirements for beams particularly in the case of members not part of any SFRS when in Seismic Design Categories D through F are not considered in the current release.
- The design and detailing requirements of members part of Special Moment Frames is beyond scope (some checks are implemented but only due to their existence in lower toughness systems).

  NOTE A full list of the code checks that have and have not been implemented is provided in the table below.

- Seismic design checks are mostly based on capacity design obtained from the main reinforcement provided. This can lead to an over-design of structural members if the designer does not take steps to minimize excess capacity.

- Beam seismic design and detailing in the current release is based on the beam rectangular section and takes under consideration the beam reinforcement only. In particular cases allowances for the slab presence and reinforcement might be required on top of the current design.
- Seismic design and detailing requirements for structural diaphragms according to ACI318-11 sections 21.3.6 and 21.11 are not considered in the current release.

ACI 318 Seismic Code Checks for beams that have been implemented in Tekla Structural Designer

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.1.4.2	Minimum required compressive strength of concrete	SMF	-	-	-	✓
21.1.4.3	Maximum allowed compressive strength of light-weight concrete	SMF	-	-	-	✓
21.1.5.2	Maximum allowed steel characteristic yield strength of longitudinal reinforcement	SMF	-	-	-	✓
21.1.5.5	Maximum allowed longitudi	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	nal reinforcement yield strength used in the calculation of transverse reinforcement					
21.1.6.2	Mechanical Splices within twice the member depth from column/ beam face or yielding regions	SMF	-	-	-	✖
21.1.6.2	Mechanical Splices outside twice the member depth from column/ beam face or yielding regions	SMF	-	-	-	✖
21.1.7.1	Welded Splices within twice the member depth from column/ beam	SMF	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	face or yielding regions					
21.1.7.2	Welding of stirrups or other elements to longitudinal reinf. required by design	SMF	-	-	-	✗
21.2.2	Minimum number of bars at top/ bottom faces continuous throughout	OMF	-	✓	-	-
21.2.2	Minimum number of bars at top/ bottom faces continuous throughout	IMF	-	-	✓	-
21.3.2	Maximum allowed factored axial force	IMF	-	-	✓	-
21.3.3.1	Minimum Design shear force	IMF	-	-	✓	-
21.3.4.1	Minimum +ve	IMF	-	-	✓	-

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	requirement (moment strength/ steel) at a joint face					
21.3.4.1	Minimum moment strength anywhere on a beam	IMF	-	-	✓	-
21.3.4.2	Type of transverse reinforcement in confinement regions (hook/ extension)	IMF	-	-	✓	-
21.3.4.2	Length of support regions measured from the face of the joint	IMF	-	-	✓	-
21.3.4.2	Maximum hoop spacing in support regions	IMF	-	-	✓	-
21.3.4.2	Maximum distance between first hoop and joint face in support regions	IMF	-	-	✓	-

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.3.4.3	Maximum hoop spacing outside confinement regions	IMF	-	-	✓	-
21.5	Beams of Special Moment Frames will frame into columns of SMF	SMF	-	-	✓	-
21.5.1.1	Maximum allowed factored axial force	SMF	-	-	✓	-
21.5.1.2	Maximum allowed effective depth	SMF	-	-	-	✓
21.5.1.3	Maximum allowed width	SMF	-	-	-	✓
21.5.1.4	Maximum allowed width	SMF	-	-	-	✓
21.5.2.1	Minimum number of bars at top/ bottom faces continuous throughout	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.5.2.1	Minimum allowed area of reinforcement at top/ bottom face throughout	SMF	-	-	-	✓
21.5.2.1	Maximum allowed area of reinforcement: Max steel ratio at top/ bottom/ side face of the beam, ρmax	SMF	-	-	-	✓
21.5.2.2	Minimum +ve requirement (moment strength / steel) at a joint face	SMF	-	-	-	✓
21.5.2.2	Minimum moment strength anywhere on a beam	SMF	-	-	-	✓
21.5.2.3	Lap splice location restrictions	SMF	-	-	-	✓
21.5.2.3	Lap Splice transverse	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	reinforcement type					
21.5.2.3	Maximum allowed hoop spacing at lap splices	SMF	-	-	-	✓
21.5.3.1	Length of support regions measured from the face of the joint	SMF	-	-	-	✓
21.5.3.1	Non-reversing plastic hinges: Flexural Yield region size (centered)	SMF	-	-	-	✗
21.5.3.2	Maximum hoop spacing in support regions	SMF	-	-	-	✓
21.5.3.2	Maximum distance between first hoop and joint face in support regions	SMF	-	-	-	✓
21.5.3.2	Non-reversing plastic	SMF	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	hinges: Maximum horizontal center spacing					
21.5.3.3	Maximum allowed spacing of flexural reinforcing bars	SMF	-	-	-	✓
21.5.3.3	Maximum allowed lateral link leg spacing in confinement regions	SMF	-	-	-	✓
21.5.3.4	Maximum hoop spacing outside confinement regions	SMF	-	-	-	✓
21.5.3.6	Type of transverse reinforcement in confinement regions (hook/ extension)	SMF	-	-	-	✗
21.5.3.6	Type of transverse reinforce	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	ment in beam sections that extend laterally beyond the column core (hook/ extension)					
21.5.4.1	Minimum Design shear force	SMF	-	-	-	✓
21.5.4.2	Unreinforced concrete shear resistance at confinement regions	SMF	-	-	-	✓
21.7.2.1	Stress in the beam flexural tensile reinforcement at joints face for joint shear calculation	SMF	-	-	-	✓
21.7.2.2	Tension anchorage length of beam long. Reinf. at external	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	joints (beyond column inner face)					
21.7.2.3	Maximum longitudinal reinforcement bar size	SMF	-	-	-	✓
-	Minimum beam depth at a joint where it contributes to joint shear	SMF	-	-	-	✓
21.7.3.3	Spacing of confinement reinf. for longitudinal bars of beams outside the column core	SMF	-	-	-	✖
21.7.3.3	Maximum distance between link legs in beam sections that extend laterally beyond the	SMF	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	column core					
21.7.5.1	Development length for beam longitudinal bars with a standard 90° hook	SMF	-	-	-	✓
21.7.5.2	Development length for beam longitudinal straight bars	SMF	-	-	-	✓
21.7.5.4	Development length for epoxy-coated or zinc and epoxy dual-coated beam longitudinal bars	SMF	-	-	-	✓
21.8.2	Minimum distance from joint face for beam reinf. mechanical splices in ductile connections	SMF	-	-	-	✗
21.8.2	Minimum nominal shear	SMF	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	strength of ductile connections					
21.8.3	Minimum nominal strength of the strong connection	SMF	-	-	-	✘
ASCE7/10 12.4.4	Extra design loads for horizontal cantilevers	-	-	-	-	✓

NOTE

- For further details of the checks that have been implemented, see: General requirements (beams seismic: ACI 318) (page 119), Flexural reinforcement (beams seismic: ACI 318) (page 129), Transverse reinforcement (beams seismic: ACI 318) (page 124) or consult the respective clause reference in the code.
- Most of the requirements will be fulfilled through automatic design. In some cases specific design options will need to be set by the user.
- Additional requirements may apply to members that are not part of the SFRS when in SDC's D, E or F
- Confinement regions: - support regions; - Probable flexural yield regions; - Lap splice regions.

General requirements (beams seismic: ACI 318)

Maximum allowed factored axial force

Flexural elements with high axial loading values under any load combination are handled in the seismic design and detailing as compressive members.

If SFRS Type = Ordinary Moment Frame, then no axial compression limit applies.

If SFRS Type = Intermediate Moment Frame or SFRS Type = Special Moment Frame

	P_{max}	=	A_g * f'_c / 10
	where		
	P_{max}	=	Maximum allowed compression value on the member
	A_g	=	Gross area of the concrete section
	f'_c	=	specified compressive strength of concrete

The check passes if;

	P_u	≥	P_{max}
	where		
	P_u	=	Maximum factored compressive axial force anywhere in the span considering all load combinations

Maximum allowed effective depth

The maximum allowed effective depth of a beam part of a Special Moment Frame is proportional to its clear span length to limit the overloading of the adjacent joints and columns.

If SFRS Type = Special Moment Frame

	d		d_{max} ≤
	where		
	d		= Distance between the extreme compression fiber and the longitudinal tension reinforcement centroid

	d_{max}		Maximum allowed distance between the extreme compression fiber and the longitudinal tension reinforcement centroid =
	d_{max}		$0.25 * l_n$ =
	where		
	l_n		Length of the clear span measured from face-to-face of supports =

Minimum allowed width

Beams part of Special Moment Frames in buildings subjected to earthquake effects have a minimum width limit for their web.

If SFRS Type = Special Moment Frame

	b_w		$b_{w,min}$ ≥	
	where			
	b_w		Beam web width =	
	$b_{w,min}$		Minimum allowed beam web width =	
			MAX(0.3 * h, 250 mm) =	Metric-units
			MAX(0.3 * h, 10 in) =	US-units
	where			
	h		Overall depth of the concrete section =	

Maximum allowed width

Despite not being advisable, beams in Special Moment Frames are allowed to be wider than the supporting columns up to a fixed limit.

NOTE The maximum lateral extension of a beam on each side of the joining column is beyond scope in the current release of Tekla Structural Designer.

Flexural requirements (beams seismic: ACI 318)

Minimum number of bars

The minimum allowed number of bars continuous along the beam span is required to be checked in the layers closest to the top and bottom faces of any beam in the SFRS.

The number of bars should be ≥ 2.

Maximum allowed bar size

This applies to end regions of beams where the beam reinforcement extends into the column core. The required development length of reinforcement bars extending into the column core restricts the minimum size of the column and vice-versa.

If SFRS Type = Special Moment Frame

“Anchorage requirements at the joint of special moment frames limit the maximum bar size at each end of the beam”

Minimum flexural strength

The minimum area of top and bottom steel required at any section of a beam part of a Moment Resisting Frame needs to comply with flexural strength requirements when considering earthquake effects.

Note that no seismic design requirements apply to beams that are part of Ordinary Moment Frames. All other Moment Resisting Frame types have minimum longitudinal moment requirements.

Minimum allowed area of reinforcement

The minimum allowed area of steel throughout the bottom and top faces of a beam part of a Special Moment Frame is limited as per ACI 318-11 equation (10-3).

Maximum allowed area of reinforcement

For the purpose of increasing the ductility response of beams in Special Moment Frames the area of reinforcement both at the top and bottom faces is limited.

If SFRS Type = Special Moment Frame

	(A_s- and As +)		$A_{s,max} = 0.025 * b_w * d$ ≤
	where		
	A_s		Area of non-prestressed longitudinal tension reinforcement =
	b_w		Beam web width =
	d		Distance from extreme compression fiber to centroid of longitudinal tension reinforcement =

No other Moment Resisting Frame type has a maximum area of steel requirement.

Maximum allowed center spacing of longitudinal bars

Limitations on the longitudinal bar spacing apply to beams part of Special Moment Frames.

If SFRS Type = Special Moment Frame, then the maximum allowed center spacing is checked for confinement regions as follows:

	$s_{cr,max}$		350 mm =	Metric-units
	A_s		14 in. =	US-units
	where			
	$s_{cr,max}$		maximum allowed center spacing =	

Non-reversing plastic hinges

Non-reversing plastic hinges are regions along the span of the beam where flexural yielding is likely to occur.

NOTE Non-reversing plastic hinges are beyond scope in the current release of Tekla Structural Designer.

Splices

Restrictions apply to the locations of reinforcement lap splices along the span of a beam part of Special Moment Frames.

Strength design of mechanical splices and restrictions to the use of welded splices as required by ACI318-11 apply to Special Moment Frames.

NOTE These restrictions are not implemented in the current release of Tekla Structural Designer.

Transverse reinforcement (beams seismic: ACI 318)

Seismic requirements relating to transverse reinforcement take into account properties, strengths and outcomes which are shear related ignoring any reinforcement intended to deal with torsional effects.

Design shear force

The design shear force for members subjected to earthquake effects is obtained by consideration of the minimum required shear strength of the member. The required nominal shear strength of a flexural member part of a Moment Resisting Frame is checked considering the sum of shears resultant from the moment strengths due to reverse curvature bending acting at each end of the beam and from the tributary factored gravity loads.

Beams are checked for shear in three regions:

- Left region, S1;
- Central region, S2;
- Right region, S3.

Shear design is performed considering the Major axis shear force only. Shear Force in the minor axis is checked against the ignorable threshold.

If SFRS Type = Ordinary Moment Frame, then no shear seismic check applies.

If SFRS Type = Intermediate Moment Frame

	V_e		$\phi MIN(V_{e,Mn}+V_{e,gravity},V_{e,2E})$ =
	where		
	ϕ		Strength reduction factor = 1.0 =
	V_e		Minimum design shear force for load combinations including earthquake effects

	$V_{e,gravity}$		Shear due to factored gravity loads from seismic combinations (including vertical earthquake effects) retaining the sign from analysis
	$V_{e,2E}$		Maximum shear resultant from seismic combinations, withdoubled earthquake effect [i.e.: $V_{e.non\text{-}seismic} + V_{e.E} \times 2$]
	$V_{e,Mn}$		Maximum shear associated with the development of reversed curvature bending due to nominal resisting moments at both ends of the member, considering both the clockwise and counter-clockwise cases

If SFRS Type = Special Moment Frame

	V_e		$\phi(V_{e,Mpr}+V_{e,gravity})$ =
	where		
	ϕ		Strength reduction factor = 1.0 =
	V_e		Minimum design shear force for load combinations including earthquake effects
	$V_{e,gravity}$		Shear due to factored gravity loads from seismic combinations (including vertical earthquake effects) retaining the sign from analysis
	$V_{e,Mpr}$		Maximum shear associated with the development of reversed curvature bending due to the probable flexural moment strength for both the clockwise and counter-clockwise situations, at both ends of the member

Maximum hoop spacing

The maximum allowed horizontal center spacing of hoops in confinement regions of beams is limited by ACI 318 depending on the type of Seismic Force Resisting System considered.

NOTE This check is performed for support regions only, it is beyond scope in the current release of Tekla Structural Designer for other confinement regions.

NOTE Non-reversing plastic hinge regions along the span have the same requirements as support regions, but these are beyond scope in the current release of Tekla Structural Designer.

For Support Regions:

If SFRS Type = Special Moment Frame

The maximum allowed center hoop spacing in support regions, $s_{cr,max,sup}$ is calculated as follows:

According to ACI318-08[1]:

	$s_{cr,max,sup}$	=	MIN(d/4, 8 * $d_{b,smallest}$, 24 * $d_{b,w}$, 300mm)	Metric-units
	$s_{cr,max,sup}$	=	MIN(d/4, 8 * $d_{b,smallest}$, 24 * $d_{b,w}$, 12 in.)	US-units

According to ACI318-11[1]:

	$s_{cr,max,sup}$	=	MIN(d/4, 6* $d_{b,smallest}$, 150 mm)	Metric-units
	$s_{cr,max,sup}$	=	MIN(d/4, 6 * $d_{b,smallest}$, 6 in.)	US-units

	where		
	d	=	Distance from extreme compression fiber to centroid of longitudinal tension reinforcement
	$d_{b,smallest}$	=	Smallest longitudinal reinforcement bar diameter
	$d_{b,w}$	=	Link (hoop) diameter

If SFRS Type = Intermediate Moment Frame

[1] ACI318-08 Section 21.5.3.2. This requirement has changed from ACI318-08 to ACI318-11

The maximum allowed center hoop spacing in support regions, $s_{cr,max,sup}$ is calculated as follows:

	$s_{cr,max,sup}$		MIN(d/4, 8* $d_{b,smallest}$, 24 * $d_{b,w}$, 300mm) =	Metric-units
	$s_{cr,max,sup}$		MIN(d/4, 8 * $d_{b,smallest}$, 24 * $d_{b,w}$, 12 in.) =	US-units

	where		
	d		Distance from extreme compression fiber to centroid of longitudinal tension reinforcement
	$d_{b,smallest}$		Smallest longitudinal reinforcement bar diameter =
	$d_{b,w}$		Link (hoop) diameter =

For Span Regions:

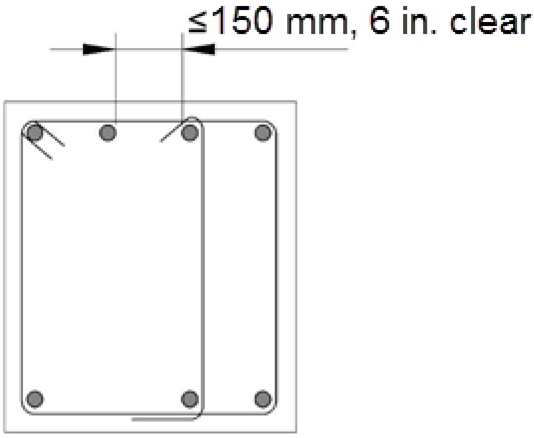
If SFRS Type = Special Moment Frame, or Intermediate Moment Frame

The maximum allowed hoops spacing outside confinement regions, $s_{cr,max,span}$ is calculated as follows:

	$s_{cr,max,span}$		d/2 =

Maximum allowed lateral link leg spacing

The clear spacing between link legs at right angles to the span is limited in confinement reinforcement regions of members part of Special Moment Frames only.



NOTE This check is performed for support regions only, it is beyond scope in the current release of Tekla Structural Designer for other confinement regions.

Requirements when in SDC D - F (beams seismic: ACI 318)

When designing members for earthquake effects, beams not part of the SFRS when in Seismic Design Categories D through F are required to be designed with seismic provisions all the same.

NOTE With the exception of seismic cantilevers, the design of these members for seismic provisions is beyond scope in the current release of Tekla Structural Designer.

Seismic cantilevers (beams seismic: ACI 318)

Horizontal cantilever structural members in structures assigned to Seismic Design Category D, E or F are required to be designed to the applicable load combinations plus an isolated minimum net upward force of 0.2* times the dead load.

If a cantilever beam has been marked as a seismic cantilever, then provided the seismic design category = D, E or F the minimum design moment at the restrained end is checked as follows:

Calculate minimum positive design moment:

	Mmin +	=	$0.2 * M_{e,dead}^{-}$
	where		
	M_{min}+	=	Minimum positive design bending moment at the restrained end.

	$M_{e,dead}-$		Critical negative bending moment at the restrained end due to dead loads only obtained for the considered seismic combination.

Check the minimum design moment at the restrained end:

	M_u+		$M_{min}+$ ≥
	where		
	M_u+		Critical negative bending moment at the restrained end due to dead loads only obtained for the considered seismic combination.

If the check fails the region is designed for $M_u+ = M_{min}+$

Flexural reinforcement (beams seismic: ACI 318)

Longitudinal reinforcement at the top and bottom faces of a beam in any Moment Resisting Frame are required to have at least two continuous steel bars along the span for structural integrity and constructability purposes.

Anchorage

Longitudinal reinforcement terminated at a column in beams that are part of Special Moment Frames shall be anchored within the element confined core for a length measured from the critical section at the element's face.

Tekla Structural Designer performs the calculation steps for the required development length at the supports of bars in tension for both the case where straight bars are used and where hooks are provided.

Lap Splices

Specific seismic requirements apply only to lap splices in flexural members that are part of Special Moment Frames.

For beams that are part of Special Moment Frames lap splices are not allowed to be located:

- Within joints or within a distance of twice the members depth from the face of a joint
- Within regions where flexural yielding is likely to occur.

NOTE The latter requirement to avoid lap splices in regions where flexural yielding is likely to occur is beyond scope in the current release of Tekla Structural Designer

Confinement reinforcement for ductility (beams seismic: ACI 318)

Reinforcement Type

Confinement reinforcement should consist of hoops, i.e. closed or continuously wound ties with a seismic hook at each end.

Detailing Regions

Confinement reinforcement is required to be provided over the following confinement regions in beams that are part of Intermediate and Special Moment Frames. This is not a requirement for beams that are part of Ordinary Moment Frames

- Support regions: These are probable flexural yielding regions flexural yielding regions next to beam-column joints

  NOTE Beam-wall moment frames are beyond scope in the current release of Tekla Structural Designer.

- Non-reversing plastic hinge regions: These are probable flexural yielding regions outside support regions.

  NOTE Non-reversing plastic hinge regions are not identified in the current release of Tekla Structural Designer.

- Lap splices: Over the full length of lap splices in members that are part of Special Moment Frames

Concrete column and wall design to ACI 318

Click the links below to find out more about the application of ACI 318 with regard to:

- Design parameters for longitudinal bars (columns) (page 131)
- Design parameters for vertical bars (walls) (page 132)
- Design parameters for horizontal bars (walls) (page 133)
- Ultimate Axial Load Limit (page 134)
- Effective length calculations (page 134)
- Column stack and wall panel classification (page 136)
- Design Moment Calculations (page 138)
- Design for combined axial and bending (page 140)
- Design parameters for shear (page 141)
- Design for in plane shear (walls) (page 141)
- Column confinement (page 142)
- Seismic design and detailing (columns) (page 142)
- Seismic design (walls) (page 165)

If working to **ACI 318-19** click the below links to view additional important information:

- Concrete column design (ACI 318-19 update) (page 83)
- Beam-column joint shear strength (ACI 318-19 update) (page 84)

See also

Limitations (concrete members: ACI 318) (page 87)

Cover to Reinforcement (ACI 318) (page 88)

Seismic Design to ACI 318 (page 199)

Design parameters for longitudinal bars (concrete column: ACI 318)

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

For design to ACI ;

minimum clear distance		MAX (1.5 * longitudinal db , 1.5in., 1.33*hagg) ≥	US units
minimum clear distance		MAX (1.5 * longitudinal db , 40mm, 1.33*hagg) ≥	metric units

where

d_b	=	bar diameter
h_{agg}	=	aggregate size

Maximum Longitudinal Bar Spacing

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

Minimum Longitudinal Total Steel Area

For design to ACI;

1% * column area

Maximum Longitudinal Total Steel Area

For design to ACI;

8% * column area

Design parameters for vertical bars (concrete wall: ACI 318)

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 > Wall > Reinforcement Layout.

Minimum and Maximum Vertical Bar Diameter

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

Minimum and Maximum Vertical Loose Bar Spacing

Limiting minimum horizontal spacing of the vertical bars, $s_{v,lim,min}$ is controlled by the diameters of the 2 adjacent bars and aggregate size[1].

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

where

c_{gap}		min. clear distance bet. bars =	
		max $(1.5*d_{bv,i}$, $1.5*d_{bv,(i+1)}$, 1.33* hagg, 1.5 in.)=	US units
		max $(1.5*d_{bv,i}$, $1.5*d_{bv,(i+1)}$, 1.33* h_{agg}, 38mm) =	metric units
where			
$d_{bv,i}$ and $d_{bv,(i+1)}$		the diameters of the two adjacent vertical bars =	

[1] Clause 7.6.3

h_{agg}		aggregate size =	

Limiting maximum horizontal spacing of the vertical bars, sv,lim,max is controlled by the wall thickness.

$s_{v,lim,max}$		min (3*hw, 18 in.) =	US units
		min (3*hw, 450mm) =	metric units

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

Minimum and Maximum Reinforcement Area

The code provisions which control the vertical reinforcement area are,

- Limiting minimum ratio of vertical reinforcement area to gross concrete area, *r*v,lim,min
- Limiting maximum ratio of vertical reinforcement area to gross concrete area, *r*v,lim,max

The controlling values are:

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

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

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

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

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

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

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

Design parameters for horizontal bars (concrete wall: ACI 318)

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

Minimum and Maximum Reinforcement Area

The code provisions which control the horizontal reinforcement area are,

- Limiting minimum ratio of horizontal reinforcement area to gross concrete area, *r*h,lim,min
- Limiting maximum ratio of horizontal reinforcement area to gross concrete area, *r*h,lim,max

The controlling values are:

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

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

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

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

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

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

Minimum and Maximum Horizontal Bar Spacing

This is identical in principle to min vertical bar spacing.

Minimum and Maximum Confinement Bar Spacing

There are Code provisions that control the maximum spacing:

The recommended values are,

Limiting maximum transverse spacing, $s_{w,lim,max}$ = min ($16*d_{bv}$, $48*d_{bw}$, h_w)

Ultimate Axial Load Limit (column and wall:ACI 318)

The strength of a column under truly concentric axial load is

P_{no}	=	$0.85*f'_c*(A_g - A_{st}) + f_y*A_{st}$

For nonprestressed compression members with tie reinforcement,

φPnmax	=	$0.80*\varphi*[0.85*f'_c*(A_g - A_{st}) + f_y*A_{st}]$

Effective length calculations (column and wall:ACI 318)

Unsupported Length

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

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

Effective Length

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

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

$0.5 \leq l_e / l_u \leq 1$

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

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

$l_e = k * l_u$

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

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

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

$-45° < \beta \leq 45°$

where

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

Fixed Column Base

Since in practical structures there is no such thing as a truly fixed end, Tekla Structural Designer limits $\psi \geq 0.20$. This being the practical lower limit suggested in “RC Mechanics” by McGregor and Wright.

Pinned Column End

In any situation where the end of a column anywhere in the structure is pinned, ψ = 1000. (This being the upper limit on ψ that is imposed by Tekla Structural Designer.)

No Effective Beams Found

If no effective beams are found to restrain the end of the stack in the direction in question, then the program will consider whether there is a flat slab restraining the stack at this end. If a flat slab is found it will either be considered as a restraint, or not, in each direction at each end of the stack - this is controlled by checking the option Use slab for calculation... located as a Stiffness setting in the column properties.

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

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

If the stack is restrained by a flat slab, then the slab will be considered to act as a beam in this direction - note that it is one beam in the direction and NOT a beam on each side of the column. The beam's length is taken as four times its width.

If the stack is restrained by a slab on beams, this will have a zero contribution to the stiffness. This thoretically has the effect of setting ψ = infinity, though it is limited to 1000 in Tekla Structural Designer before being used in the calculations.

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

Column stack and wall panel classification (column and wall:ACI 318)

Slenderness ratio

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

$(k\ lu/r)_y = k * lu_y / (\sqrt{(I_y / A_g)})$

$(k\ lu/r)_z = k * lu_z / (\sqrt{(I_z / A_g)})$

where

slenderness ratio = k*lu/r

k is an effective length factor

lu_y is the unsupported column length in respect of major axis (y axis)

lu_z is the unsupported column length in respect of minor axis (z axis)

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

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

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

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

A_g is the cross-sectional area of the stack section

For unbraced columns

IF $(k\ lu/r)_y \le 22$

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

ELSE, column is considered as slender

IF $(k\ lu/r)_z \le 22$

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

ELSE, column is considered as slender

For braced columns

IF $(k\ lu/r)_y \le$ MIN((34-12*M1/M2), 40)

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

ELSE, column is considered as slender

IF $(k\ lu/r)_z \le$ MIN((34-12*M1/M2), 40)

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

ELSE, column is considered as slender

where

M1 = the smaller factored end moment on the column, to be taken as positive if member is bent in single curvature and negative if bent in double curvature

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

M2 = the larger factored end moment on the column always taken as positive

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

Design Moment Calculations (column and wall: ACI 318)

For each combination and for each analysis model (Building Analysis, Grillage Analysis, FE Analysis) the end moments about the two local member axes, 'major' and 'minor' are established. From these and the local load profile, the moments and axial force at any position in the member can be established. These moments will be from a first-order or second-order analysis at user choice - (in making the choice, the value of the 'stability index', Q should be taken into account).

Note that M2 and M1 are the end moments with M2 being the larger numeric value.

Step 1, minimum moment

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

M_{min}	=	Pu*(0.6+0.03*h)] in	(US units)
	=	Pu*(15+0.03*h)] mm	(metric units)

where

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

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

Step 2 - member slenderness

It is determined whether the member is slender or not. Note that in the determination for braced columns M1 and M2 are always the end moments even if lateral loading is present.

Step 3 - non-slender column

Calculate the design moment at the top, bottom and mid-fifth of the column in each direction taking into account if lateral loads are “significant”, or “not significant”.

As the column is non-slender no further calculations are required to establish design moments.

Step 4 - slender member amplifier

Calculate the “amplifier” due to buckling about each of the major and minor axes excluding the uniform moment factor which is dealt with separately,

k_{ns}	=	$1/(1 - (P_u / (0.75*Pc))) \geq$ zero

where

P_c	=	The critical buckling load	
	=	$\pi^2*(EI)/(k*lu)^2$	

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

Step 5 - uniform moment factor

For lateral loads that are “not significant”,

	C_m		0.6 + 0.4 *(M1 / M2)	

retain

					ing moment signs

Else

	C_m		1.0 =

Step 6 - moment magnifier

Calculate the moment magnifier from Equ. 10.12 as,

d_{ns}	=	MAX [C_m * k_{ns}, 1.0]

Step 7 - amplified minimum moment

Calculate the amplified minimum moment as,

M_{min_amp}	=	M_{min} * k_{ns}

Step 8 - design moments

Calculate the design moment at the top, bottom and mid-fifth of the column in each direction taking into account if lateral loads are "significant", or "not significant".

Design for combined axial and bending (column and wall:ACI 318)

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.

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

Where

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

Design parameters for shear (column and wall:ACI 318)

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

Minimum Shear Link Diameter

For Ties, minimum shear reinforcement size

IF maximum longitudinal bar ≤ 1.27 in. (32.3mm)

shear reinforcement diameter = 0.375 in. (9.5mm)

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

Maximum Span Region Shear Link Spacing

Controlled by seismic detailing requirements.

Maximum Support Region Shear Link Spacing

Controlled by seismic detailing requirements.

Design for in plane shear (walls:ACI 318)

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

V_u	≤	ΦV_n

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

V_u	≤	$\Phi V_c + \Phi V_s$

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

V_n	$\leq$	$10 * \sqrt{f'_c} * h * d$	US units
V_n	$\leq$	$0.83 * \sqrt{f'_c} * h * d$	metric units

where

h	=	wall thickness
d	=	$0.8 * l_w$
l_w	=	length of the wall

Out of plane the shear design calculations are the same whether the design element is a column or a wall - see: Design parameters for shear (column and wall:ACI 318) (page 141)

Column confinement (column and wall:ACI 318)

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

Seismic design and detailing (columns: ACI 318)

For overall limitations and assumptions, see:

- Limitations and assumptions (columns seismic: ACI 318) (page 142)

For column design in moment resiting frames, see:

- General requirements (columns seismic: ACI 318) (page 155)
- Flexural requirements (columns seismic: ACI 318) (page 157)
- Transverse reinforcement (columns seismic: ACI 318) (page 160)

For seismic detailing, see:

- Flexural reinforcement (columns seismic: ACI 318) (page 164)
- Confinement reinforcement for ductility (columns seismic: ACI 318) (page 164)

See also

Seismic Design to ACI 318 (page 199)

Limitations and assumptions (columns seismic: ACI 318)

The following limitations and assumptions apply.

- Seismic design is only performed for columns marked as part of a Seismic Force Resisting System.
- Requirements for columns particularly in the case of members not part of any SFRS when in Seismic Design Categories D through F are not considered in the current release.
- The design and detailing requirements of members part of Special Moment Frames is beyond scope (some checks are implemented but only due to their existence in lower toughness systems).

NOTE A full list of the code checks that have and have not been implemented is provided in the table below.

- No height limitations apply to Seismic Force Resisting Systems in the form of Moment Resisting Frames according to ASCE7-10, Table 12.2-1.
- The use of spiral reinforcement as well as all seismic design checks and related assumptions are not considered due to the fact that this type of reinforcement is not currently available in Tekla Structural Designer.
- The Seismic Force Resisting System set by the user in each direction through the Seismic Wizard for analysis purposes is not checked for applicability against the allowed types of the resultant Seismic Design Category. This is a user responsibility.
- Seismic design checks are mostly based on capacity design obtained from the main reinforcement provided. This can lead to an over-design of structural members if the designer is not careful enough to minimize excess capacity, especially in columns considering the weak beam - strong column philosophy.

ACI 318 Seismic Code Checks for columns that have been implemented in Tekla Structural Designer

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.1.4.2	Minimum required compressive strength of concrete	SMF	-	-	-	✓
21.1.4.3	Maximum allowed compress	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	ive strength of light-weight concrete					
21.1.5.2	Maximum allowed steel characteristic yield strength of longitudinal reinforcement	SMF	-	-	-	✓
21.1.5.4	Maximum yield strength of transverse reinforcement in confinement regions of columns	SMF	-	-	-	✓
21.1.5.5	Maximum allowed longitudinal reinforcement yield strength used in the calculation of transverse reinforcement	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.1.5.5	Maximum allowed characteristic yield strength of shear reinforcement	SMF	-	-	-	✓
21.2.3	Design shear force (page 160)	OMF	-	✓	-	-
21.3.2	Minimum factored axial force (page 155)	IMF	-	-	✓	-
21.3.3.2	Design shear force (page 160)	IMF	-	-	✓	-
21.3.5.2	Minimum Support Region size (confinement reinforcement applies)	IMF	-	-	✓	-
21.3.5.2	Maximum allowed center hoop spacing (page 160) in confinement regions	IMF	-	-	✓	-

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.3.5.3	First hoop placing distance from face of joint in support regions	IMF	-	-	✓	-
21.3.5.3	First hoop placing distance from face of joint in support regions	SMF	-	-	-	✓
21.3.5.5	Minimum depth of column transverse reinforcement into the joint	IMF	-	-	✓	-
21.3.5.5	Minimum area of column rectangular transverse reinforcement	IMF	-	-	✓	-
21.3.5.5	Minimum depth of column transverse reinforcement into the joint	SMF	-	-	-	✓
21.3.5.5	Minimum volumetric ratio / area of	IMF	-	-	✓	-

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	column spiral or circular transverse reinforcement					
21.3.5.6	Length of confinement region when inside footings, mats or pile caps	IMF	-	-	✓	-
21.3.5.6	Length of confinement region for columns supporting/above discontinuous stiff members (walls)	IMF	-	-	✓	-
21.6.1	Minimum factored axial force (page 155)	SMF	-	-	-	✓
21.6.1.1	Minimum overall dimension	SMF	-	-	-	✓
21.6.1.2	Minimum shortest dimension to perpendicular dimension ratio	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.6.2.2/21.6.2.3	Minimum flexural strength (page 157)	SMF	-	-	-	✓
21.6.3.1	Minimum allowed area of reinforcement	SMF	-	-	-	✓
21.6.3.1	Maximum allowed area of reinforcement (page 164)	SMF	-	-	-	✓
21.6.3.2	Minimum allowed number of bars in columns with circular hoops	SMF	-	-	-	✓
21.6.3.3	Lap splice allowed locations	SMF	-	-	-	✓
21.6.3.3	Mechanical Splices within twice the member depth from column/ beam face or yielding regions	SMF	-	-	-	✗
21.6.3.3	Mechanical Splices outside twice the	SMF	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	member depth from column/ beam face or yielding regions					
21.6.3.3	Welded Splices within twice the member depth from column/ beam face or yielding regions	SMF	-	-	-	✗
21.6.3.3	Welding of stirrups or other elements to longitudinal reinf. required by design	SMF	-	-	-	✗
21.6.3.3	Minimum lap splice length	SMF	-	-	-	✓
21.6.4.1	Minimum Support Region size (confinement reinforcement applies)	SMF	-	-	-	✓
21.6.4.1	Minimum length of confinem	SMF	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	ent region at other flexural yielding sections					
21.6.4.2	Type of confinement reinforcement (hook/ extension)	SMF	-	-	-	✓
21.6.4.2	Maximum allowed cross section center link leg spacing in confinement regions	SMF	-	-	-	✓
21.6.4.3	Maximum allowed center hoop spacing (page 160) in confinement regions	SMF	-	-	-	✓
21.6.4.3	Maximum hoop spacing at lap Splices (page 157)	SMF	-	-	-	✗
21.6.4.4a)	Minimum volumetri	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	c ratio / area of spiral or circular confinement reinforcement					
21.6.4.4b)	Minimum area of rectangular confinement reinforcement	SMF	-	-	-	✓
21.6.4.5	Maximum center link/ stirrup spacing in non special transverse reinf. regions	SMF	-	-	-	✓
21.6.4.6	Length of confinement region when inside footings, mats or pile caps	SMF	-	-	-	✓
21.6.4.6	Length of confinement region for columns supporting/above discontinuous stiff	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	members (walls)					
21.6.4.7	Spacing of transverse reinforcement for non-structural extensions	SMF	-	-	-	✗
21.6.5.1	Design shear force (page 160)	SMF	-	-	-	✓
21.6.5.2	Unreinforced Shear resistance at confinement regions	SMF	-	-	-	✓
21.7.3.1	Minimum volumetric ratio / area of column spiral or circular transverse reinforcement	SMF	-	-	-	✓
21.7.3.1	Minimum area of column rectangular transverse reinforcement	SMF	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.7.3.1	Type of column transverse reinforcement (hook/ extension)	SMF	-	-	-	✓
21.7.3.1	Maximum allowed column cross section center link leg spacing	SMF	-	-	-	✓
21.7.3.1	Maximum allowed center hoop spacing (page 160)	SMF	-	-	-	✓
21.7.3.1	Spacing of column transverse reinforcement for non-structural extensions	SMF	-	-	-	✗
21.7.3.2	Minimum column transverse reinf.with beams in all directions	SMF	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	≥ 3/4 the column's width					
21.7.4.1	Maximum nominal shear strength for joints confined by beams on all four faces	SMF	-	-	-	✓
21.7.4.1	Maximum nominal shear strength for joints confined by beams on three faces or two opposite faces	SMF	-	-	-	✓
21.7.4.1	Maximum nominal shear strength for joints not confined by beams	SMF	-	-	-	✓
21.8.3	Minimum nominal strength of the strong connection for column-to-column	SMF	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	connections					
21.8.3	Minimum nominal moment strength of the strong connection for column-to-column connections	SMF	-	-	-	✖
21.8.3	Minimum nominal shear strength of the strong connection for column-to-column connections	SMF	-	-	-	✖

NOTE

- For further details of the checks that have been implemented, see: General requirements (columns seismic: ACI 318) (page 155), Flexural reinforcement (columns seismic: ACI 318) (page 164), Transverse reinforcement (columns seismic: ACI 318) (page 160), or consult the respective clause reference in the code.
- Most of the requirements will be fulfilled through automatic design. In some cases specific design options will need to be set by the user.
- Additional requirements may apply to members that are not part of the SFRS when in SDC's D, E or F
- Confinement regions: - support regions; - Probable flexural yield regions; - Lap splice regions.

General requirements (columns seismic: ACI 318)

End Fixity

Reinforced concrete columns assigned to Moment Resisting Frames have their end fixities at the base of the building limited to:

- Fixed base;
- Pinned base;
- Spring base (foundation flexibility).

Minimum factored axial force

Members experiencing axial compression forces higher than the minimum threshold in the code from any of the load combinations are required to be checked for flexural strength and to consider flexural detailing within the strong column - weak beam design philosophy according to their assigned SFRS type.

If SFRS Type = Ordinary Moment Frame, then no No axial compression load requirement applies.

If SFRS Type = Intermediate Moment Frame, or Special Moment Frame

	P_{min}		A_g * f'_c / 10 =
	where		
	P_{min}		Minimum required axial compression =
	A_g		Gross area of the concrete section =
	f'_c		Specified compressive strength of concrete =

The check passes and the member is designed for seismic provisions as a compressive member if Pu > Pmin

	where		
	P_u		Largest factored compressive axial force at the top of the stack from any load combination. =

NOTE	This check is no longer required in ACI 318-14.

Maximum recommended axial force

ACI 318 allows for the maximum design axial load to be as high as $0.8*\phi*P_{n,max}$, where $P_{n,max}$ is the maximum compression resistance of the section composed of concrete and steel. However in the event of a severe earthquake a full height beam yielding mechanism could occur inducing higher compression strain on the columns than the one predicted by elastic design.

Good practice recommends that the maximum compressive strain in a column part of a Special Moment Frame should remain below the balanced value.

NOTE	The current release of Tekla Structural Designer does not check if the compressive strain is below the balanced point.

Minimum cross-section dimension

ACI 318 limits the shortest cross-sectional dimension of a column that is part of a Special Moment Frame to a lower fixed value in any direction measured in a straight line passing through the section centroid and also to a fraction of the length of the perpendicular dimension.

If designing to ACI 318-14, there is an additional limit of half the height of the deepest beam connecting at the joint.

These minimum dimension restrictions are calculated and applied accordingly when the column that is part of a Special Moment Frame.

Flexural requirements (columns seismic: ACI 318)

Minimum flexural strength

Columns that are part of Special Moment Frames are required by ACI 318 to have a minimum amount of flexural strength depending on the connecting beams flexural capacities so as to promote the formation of beam yielding mechanisms in the case of an earthquake. This is done by establishing a ratio between the beam and column moment strengths in the moment resisting frame direction.

The design of the main reinforcement in a column is done for the top, middle and bottom region of the stack and moment capacity is calculated for the factored axial force in each region for the Major and Minor directions.

If SFRS Type = Special Moment Frame then flexural strength checks are performed at the joints:

	$\Sigma M_{nc,l}$		$M_{nc,l,bot} + M_{nc,l,top}$ =

	$\Sigma M_{nc,r}$		$M_{nc,r,bot} + M_{nc,r,top}$ =
	where		
	$\Sigma M_{nc,l}$, $\Sigma M_{nc,r}$		Sum of the Nominal Flexural Strengths of the columns framing into the joint in the relevant direction.
	$M_{nc,l,bot}$, $M_{nc,r,bot}$		Nominal Moment Strength of the stack below the joint obtained for the axial force value consistent with the minimum Nominal Moment Strength respectively for the left and right sway cases.
	$M_{nc,l,top}$, $M_{nc,r,top}$		Nominal Moment Strength of the stack above converging on the same joint for the axial force value consistent with the minimum Nominal Moment Strength respectively for the left and right sway cases.

The sum of the beam strengths are obtained as follows:

	$\Sigma M_{nb,l}$		$M_{nb,l}$- + $M_{nb,l}$+ =
	$\Sigma M_{nb,r}$		$M_{nb,r}$- + $M_{nb,r}$+ =
	where		
	$\Sigma M_{nb,l}$, $\Sigma M_{nb,r}$		Sum of the Nominal Flexural Strengths of the beams framing into the joint in the relevant direction.
	$M_{nb,l}$- , $M_{nb,r}$-		Nominal Moment Strength at the joint from the beam on the left from current reinforcement arrangement respectively for the left and right sway cases.

	$M_{nb,l}+$, Mnb,r+		Nominal Moment Strength at the joint from the beam on the right from current reinforcement arrangement respectively for the left and right sway cases.

The minimum strength ratio between columns and beams in both left and right sway cases is checked as follows:

	$\Sigma M_{nc,l}$	≥	$6/5 * \Sigma M_{nb,l}$
	$\Sigma M_{nc,r}$	≥	$6/5 * \Sigma M_{nb,r}$

Maximum allowed area of reinforcement

The maximum area of longitudinal reinforcement in columns part of Special Moment Frames is limited as follows.

If SFRS Type = Special Moment Frame

Then calculate maximum area of steel, $A_{s,max}$ as follows:

	$A_{s,max}$	=	$0.06 * A_g$
	where		
	$A_{s,max}$	=	Maximum allowed area of reinforcement
	A_g	=	Gross area of the concrete section.

Maximum allowed longitudinal bar center spacing

Limitations on the longitudinal bar spacing emerge from the code requirement for maximum allowed cross-section center link spacing of the confinement reinforcement due to the method of link leg distribution across the column section.

If SFRS Type = Special Moment Frame

Then check maximum longitudinal reinforcement bar distance, scr,max as follows:

	$s_{cr,max}$	=	350 mm	Metric-units
	$s_{cr,max}$	=	14 in	US-units

NOTE In ACI 318-14, scr,max is limited to 200mm for non-circular columns with, Pu > 0.3*Ag*fc OR fc > 70 MPa (10,000 psi).

Non-reversing plastic hinges

Non-reversing plastic hinges are regions along the stack of the column where flexural yielding is likely to occur.

NOTE Non-reversing plastic hinges are beyond scope in the current release of Tekla Structural Designer.

Splices

Columns that are part of Special Moment Frames have restrictions on the allowed locations of lap splices.

Strength design of mechanical splices and restrictions to the use of welded splices as required by ACI 318-11 apply to Special Moment Frames

NOTE These restrictions are not implemented in the current release of Tekla Structural Designer.

Transverse reinforcement (columns seismic: ACI 318)

Joint shear strength

The calculation of joint shear strength is a requirement of ACI 318 for joints of Special Moment Frames and it is obtained by considering both the free body diagram of the column and of the joint. The stress in the beam's tensile reinforcement at the joint's face is assumed as at least η *fy by considering the probable moment strengths of framing beams.

The calculation is performed on the following basis:

- Probable Moment Strengths are obtained from beams in the same direction as the column's considered direction
- Whether beams in the column SFRS Direction are included in the SFRS or not

- At the top region of stack only
- For both sway right and sway left cases
- Beams with pinned connection are ignored
- Axial stress in beams is assumed to be zero
- Contribution of the slab longitudinal reinforcement in the beam effective flange width is recommended to be considered, but remains beyond scope in the current release.

Design shear force

The Design Shear Force of a member subjected to flexure as well as axial loading part of a Moment Resisting Frame is checked taking into consideration the shear from the moment strengths of the connected flexural members due to reverse curvature bending.

- Shear design for columns is done for the entire stack as a single region and checked for the minimum requirements from the design code.
- The design is performed independently for both the orthogonal directions.

Minimum area of transverse reinforcement

For columns that are part of Special Moment Frames, the minimum area of transverse reinforcement required in confinement regions of a column is obtained as below:

NOTE For Special Moment Frames the amount of confinement reinforcement in joints with beams on all 4 sides wider than ¾ of the column width is allowed to be reduced to half and the spacing to be relaxed within the depth of the shallowest member - this is beyond scope in the current release of Tekla Structural Designer.

NOTE Non-reversing plastic hinge regions along the span have the same requirements as support regions - Non-reversing plastic hinge regions are beyond scope in the current release of Tekla Structural Designer.

For circular columns:

ρ_s	=	MAX[0.12 * (f'_c / f_{yt}) , 0.45 * [(A_g / A_{ch})-1] * (f'_c / f_{yt})]
where		
ρ_s	=	ratio of volume of circular reinforcement to total volume of confined concrete core.
f'_c	=	Specified compressive strength of concrete.

f_{yt}		Specified yield strength of transverse reinforcement. =
A_g		Gross area of concrete section. =
A_{ch}		Area of concrete member section measured to the outside of the transverse reinforcement.

For other supported column geometries

ACI 318-11		
A_{sh}		MAX[0.3 * s * b_c * (f'_c / f_{yt}) * [(A_g / A_{ch})-1] , 0.09 * s * b_c * (f'_c / f_{yt})] =
ACI 318-14		
A_{sh}		MAX[0.3 * s * b_c * (f'_c / f_{yt}) * [(A_g / A_{ch})-1] , 0.09 * s * b_c * (f'_c / f_{yt}), = 0.2 * k_f * k_n * (P_u /(MIN[f_{yt} , 700MPa] * A_{ch})]
where		
A_{sh}		total cross-section of transverse reinforcement, including cross-ties, within spacing s and perpendicular to dimension bc.
s		Center to center spacing of transverse reinforcement along the region's height.
b_c		Cross section dimension of the member core measured to the outside of the transverse reinforcement and in the direction perpendicular to the considered reinforcement link legs.
A_g		Gross area of concrete section. =
A_{ch}		Area of concrete member section measured to the outside of the transverse reinforcement.

Support regions of columns belonging to any other Moment Resisting Frame type have the minimum area of transverse reinforcement as per conventional design requirements.

Maximum allowed center hoop spacing

The maximum allowed horizontal center spacing of hoops in confinement regions of columns part of Moment Resisting Frames is limited as below.

If SFRS Type = Ordinary Moment Frame:

• No spacing requirement applies to support regions when designing for seismic provisions.

If SFRS Type = Special Moment Frame and the region is a support region:

$s_{cr,max,sup}$		MIN[6*$d_{b,smallest}$, 0.25*MIN(c_1, c_2), 100 mm ≤ 100+((350 – h_x)/3) ≤ 150 mm]	metric units
$s_{cr,max,sup}$		MIN[6*db,smallest, 0.25*MIN(c_1, c_2), 4 in. ≤ 4+((14 - h_x)/3) ≤ 6 in.]	US units
where			
$d_{b,smallest}$		Smallest longitudinal reinforcement bar diameter	
c_1		Rectangular or equivalent rectangular column dimension in the direction of the span for which moments are being considered	
c_2		Dimension of the column perpendicular to c_1	
h_x		Maximum center-to-center horizontal spacing of crossties at any face of the column	

If SFRS Type = Special Moment Frame and the region is not a support region:

$s_{cr,max,span}$		MIN[6 * $d_{b,smallest}$, 150 mm]	metric units
$s_{cr,max,span}$		MIN[6 * $d_{b,smallest}$, 6 in.]	US units

If SFRS Type = Intermediate Moment Frame and the region is a support region:

$s_{cr,max,sup}$		MIN[8*$d_{b,smallest}$, 24 *$d_{b,w}$, 1/2 * MIN(c_1, c_2), 300 mm]	metric units
$s_{cr,max,sup}$		MIN[8*$d_{b,smallest}$, 24 **$d_{b,w}$, 1/2 * MIN(c_1, c_2), 12 in.]	US units

where			
$d_{b,w}$		= Link (hoop) diameter	

If SFRS Type = Intermediate Moment Frame and the region is not a support region:

- No spacing requirement applies beyond the conventional design requirements.

Flexural reinforcement (columns seismic: ACI 318)

Development Length at the Foundation

Columns that are part of Special Moment Frames shall have their longitudinal reinforcement extended into supporting footings, foundation mats or pile caps for a length not less than the full development length in tension.

Lap Splices

Specific seismic requirements apply only to lap splices in compressive members that are part of Special Moment Frames.

For columns that are part of Special Moment Frames:

- lap splices are only allowed at the center half of the column
- lap splices regions should be properly confined - hoop spacing should not exceed the maximum allowed hoop spacing

NOTE Both of these requirements are beyond scope in the current release of Tekla Structural Designer.

Confinement reinforcement for ductility (columns seismic: ACI 318)

Reinforcement Type

Confinement reinforcement in columns at regions where provided should consist of hoops, i.e. closed or continuously wound ties with a seismic hook at each end.

NOTE Confinement reinforcement in the form of spiral reinforcement is beyond scope in the current release of Tekla Structural Designer.

Detailing Regions

Confinement reinforcement is required to be provided over three types of regions along reinforced concrete columns that are part of Intermediate Moment Frames and Special Moment Frames:

- Support regions: These are probable flexural yielding regions at the top and bottom of the stack next to column-beam joints;
- Non-reversing plastic hinge regions: These are probable flexural yielding regions outside support regions.

  NOTE Non-reversing plastic hinge regions are not identified in the current release of Tekla Structural Designer.

- Lap splices: Confinement reinforcement in the form of hoops is required to be provided over the length of lap splices in reinforced concrete columns part of Special Moment Frames

NOTE The requirement for hoop spacing not to exceed the maximum allowed hoop spacing at lap splices is beyond scope in the current release of Tekla Structural Designer.

Seismic design (walls: ACI 318)

Direction dependant seismic checks are performed in the in plane direction only as this is the only direction in which shear walls are considered to act as Seismic Force Resisting Systems.

Walls included in the SFRS as Ordinary Reinforced Concrete Structural Walls (ORCSW) have no specific seismic design provisions according to ACI318-11.

For overall limitations and assumptions, see:

- Limitations and assumptions (walls seismic: ACI 318) (page 165)

For seismic design, see:

- General requirements (walls seismic: ACI 318) (page 182)

See also

Seismic Design to ACI 318 (page 199)

Limitations and assumptions (walls seismic: ACI 318)

The following limitations and assumptions apply.

- For design purposes Tekla Structural Designer recognizes walls as isolated elements and as such the influence of flanges from adjacent walls are not to be considered when fulfilling seismic design requirements in those elements. Requirements of ACI318-11 section 21.9.5.2 and section 21.9.6.4(b) do not apply.
- Additional requirements for wall piers not part of any SFRS when in Seismic Design Categories D through F are not considered in the current release.

- Design and/or detailing requirements of Special Reinforced Concrete Structural Walls is beyond scope (some checks are implemented but only due to their existence in lower toughness systems).
- The use of spiral reinforcement as well as all seismic design checks and related assumptions are not considered due to the fact that this type of reinforcement is not currently available in Tekla Structural Designer.
- Special boundary elements in walls are directly linked with wall end-zones in Tekla Structural Designer. The current settings do not allow for end-zones of width different from the width of the wall itself.
- The Seismic Force Resisting System set by the user in each direction through the Seismic Wizard for analysis purposes is not checked for applicability against the allowed types from the resultant Seismic Design Category. This is user responsibility.
- Seismic design checks are mostly based on capacity design obtained from the main reinforcement provided. This can lead to an over-design of structural members if the designer is not careful enough to minimize excess capacity.
- The seismic design requirements for end-zone reinforcement are beyond scope in the current release.
- Construction joints are beyond scope in the current release.
- Seismic design provisions specific for beam-wall frames are beyond scope in the current release.
- Seismic design of walls with openings is beyond scope in the current release.
- Seismic design of wall piers is beyond scope in the current release.
- Seismic design of coupling beams is beyond scope in the current release.
- Seismic detailing requirements apply to Special Reinforced Concrete Structural Walls these are beyond scope in the current release.
- **NOTE** A full list of the code checks that have and have not been implemented is provided in the table below.

ACI 318 Seismic Code Checks for walls that have been implemented in Tekla Structural Designer

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.1.4.2	Minimum required compressive strength	SRCSW	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	of concrete					
21.1.4.3	Maximum allowed compressive strength of light-weight concrete	SRCSW	-	-	-	✓
21.1.5.2	Maximum allowed steel characteristic yield strength of longitudinal reinforcement	SRCSW	-	-	-	✓
21.1.5.4	Maximum yield strength of transverse reinforcement in confinement regions of columns	SRCSW	-	-	-	✗
21.1.5.5	Maximum allowed longitudinal reinforcement yield strength used in	SRCSW	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	the calculation of transverse reinforcement					
21.1.6.1a)	Mechanical Splices outside twice the member depth from column/ beam face or yielding regions	SRCSW	-	-	-	✖
21.1.6.1b)	Mechanical Splices within twice the member depth from column/ beam face or yielding regions	SRCSW	-	-	-	✖
21.1.7.1	Welded Splices within twice the member depth from column/ beam face or yielding regions	SRCSW	-	-	-	✖
21.1.7.2	Welding of	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	stirrups or other elements to longitudinal reinf. required by design					
21.12.2.3	Confinement reinf.: Length inside footing when Special Boundary Element is within half the foundation depth from the footing edge	SRCSW	-	-	-	✖
21.4.3	Minimum yield strength of connection not designed to yield	IPCSW	-	-	✖	-
21.4.4/21.9.8	Design of wall piers as columns ($l_w/b_w \leq$ 2.5)	SRCSW	-	-	-	✖
21.9.2.1	Minimum reinforcement ratio in each of the wall plane	SRCSW	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	orthogonal directions					
21.9.2.1	Maximum allowed center spacing	SRCSW	-	-	-	✓
21.9.2.2	Minimum number of reinforcement layers (page 182)	SRCSW	-	-	-	✓
21.9.2.3a)	Minimum discontinuous vertical bar extension	SRCSW	-	-	-	✗
21.9.2.3a)	Development length at locations where flexural yielding is likely to occur	SRCSW	-	-	-	✗
21.9.2.3c)	Minimum yield strength for development length and lap splices	SRCSW	-	-	-	✗
21.9.3	Factored shear force at any section	SRCSW	-	-	-	✓

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	from lateral load analysis					
21.9.4.1	Maximum nominal shear strength	SRCSW	-	-	-	✓
21.9.4.3	Reinforcement in wall plane provided in both directions	SRCSW	-	-	-	✓
21.9.4.3	Minimum in plane reinforcement ratios (page 182)	SRCSW	-	-	-	✓
21.9.4.4	Maximum wall segment combined nominal shear strength	SRCSW	-	-	-	✗
21.9.4.4/21.9.4.5	Maximum individual vertical or horizontal wall segment or coupling beam shear strength	SRCSW	-	-	-	✗
21.9.5.2	Effective width of flanged	SRCSW	-	-	-	✗

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	wall sections					
21.9.6.3	Minimum extreme fiber compressive stress to require Special Boundary Elements	SRCSW	-	-	-	✖
21.9.6.3	Minimum stress to discontinue Special Boundary Element	SRCSW	-	-	-	✖
21.9.6.4(a)	Minimum length of end-zone towards the center of the cross-section when Special Boundary Elements are required	SRCSW	-	-	-	✖
21.9.6.4(b)	Minimum length of end-zone towards the center of the cross-section in flanged sections with Special	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	Boundary Elements					
-	Minimum width of end-zone with special boundary elements	SRCSW	-	-	-	✖
21.9.6.4(c)	Special confining reinforcement type (hook/ extension)	SRCSW	-	-	-	✖
21.9.6.4c)	Confining reinf.: Maximum spacing allowed between cross ties	SRCSW	-	-	-	✖
21.9.6.4c)	Confining reinf.: Maximum allowed longitudinal center link spacing	SRCSW	-	-	-	✖
21.9.6.4c)	Confinement reinf.: Minimum volumetric ratio / area of spiral or circular reinforcement	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.9.6.4c)	Confinement Reinf.: Minimum area of rectangular transverse reinforcement	SRCSW	-	-	-	✖
21.9.6.4d)	Confining reinf.: Length of region inside footings, mats or pile caps, l_d	SRCSW	-	-	-	✖
21.9.6.4d)	Confining reinf.: Length of region into support	SRCSW	-	-	-	✖
21.9.6.4e)	Development / anchorage of horizontal reinforcement with boundary elements	SRCSW	-	-	-	✖
21.9.6.5a)	Ordinary Boundary Confinement region length - towards the	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	center of the cross-section - at an end where special boundary elements are not required					
21.9.6.5a)	Maximum spacing allowed between cross ties in high compression confinement reinf. at a boundary where special boundary elements are not required	SRCSW	-	-	-	✖
21.9.6.5a)	Maximum long. center spacing of high compression confinement reinforcement at an end where special boundary elements	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	are not required					
21.9.6.5a)	Maximum spacing allowed between cross ties or legs of hoops	SRCSW	-	-	-	✖
21.9.6.5b)	Design shear threshold for ignoring the need of engaging horizontal bars at the ends with standard hooks	SRCSW	-	-	-	✖
21.9.7.1	Minimum value of aspect ratio (l_n/h) to consider diagonal reinf.	SRCSW	-	-	-	✖
21.9.7.2	Maximum shear allowed before considering diagonal reinf.	SRCSW	-	-	-	✖
21.9.7.4a)	Nominal Shear Strength of a	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	coupling beam					
21.9.7.4b)	Minimum number of bars to be provided along each diagonal	SRCSW	-	-	-	✖
21.9.7.4b)	Minimum length of diagonal bars embedded into the wall	SRCSW	-	-	-	✖
21.9.7.4c)	Minimum breadth of the concrete core measured to the external face of the confining reinforcement	SRCSW	-	-	-	✖
21.9.7.4c)	Minimum dimension of the concrete core in any direction than not the parallel to b_w, measured to the external face of	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	the confining reinf.					
21.9.7.4c)	Special confining reinforcement (hook/ extension)	SRCSW	-	-	-	✖
21.9.7.4c)	Confining reinf.: Maximum allowed center link spacing	SRCSW	-	-	-	✖
21.9.7.4c)	Confining reinf.: Minimum volumetric ratio / area of spiral or circular reinf.	SRCSW	-	-	-	✖
21.9.7.4c)	Confining Reinf.: Minimum area of rectangular transverse reinf.	SRCSW	-	-	-	✖
21.9.7.4c)	Minimum allowed total area of the additional longitudinal reinforcement	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
21.9.7.4c)	Maximum allowed spacing between the additional longitudinal bars	SRCSW	-	-	-	✖
21.9.7.4c)	Minimum allowed area of the additional transverse reinforcement	SRCSW	-	-	-	✖
21.9.7.4c)	Maximum allowed spacing between the additional transverse bars	SRCSW	-	-	-	✖
21.9.8.1a)	Design shear force calculation for wall piers with l_w/b_w > 2.5 and not designed as columns	SRCSW	-	-	-	✖
21.9.8.1b)	Nominal shear strength and distributed shear	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	reinforcement					
21.9.8.1c)	Reinforcement type requirement for wall piers with l_w/b_w > 2.5 and not designed as columns	SRCSW	-	-	-	✖
21.9.8.1d)	Maximum allowed vertical spacing of transverse reinforcement. Wall piers with l_w/b_w > 2.5 and not designed as columns	SRCSW	-	-	-	✖
21.9.8.1e)	Length of the transverse reinf. region above and below the wall pier. Wall piers with l_w/b_w > 2.5 and not	SRCSW	-	-	-	✖

Code Ref.	Requirement	SFRS	SDC A	SDC A	SDC B	SDC D,E,F
	designed as columns					
21.9.8.1f)	Consider boundary elements	SRCSW	-	-	-	✖
21.9.8.2	Horizontal reinf. In adjacent walls when wall pier is at the edge of a wall	SRCSW	-	-	-	✖
ASCE7/10 - 12.2.1	Limiting Height (page 182)	SRCSW	-	-	-	✖
ASCE7/10 - 12.2.1	Limiting Height (page 182)	IPCSW	-	-	-	✖
R21.9.1	Vertical Segment Classification (page 182): Conditions for wall segments to require specific wall pier design	SRCSW	-	-	-	✖

NOTE
- For further details of the checks that have been implemented, see: General requirements (walls seismic: ACI 318) (page 182), or consult the respective clause reference in the code.
- Most of the requirements will be fulfilled through automatic design. In some cases specific design options will need to be set by the user.

- Additional requirements may apply to members that are not part of the SFRS when in SDC's D, E or F
- Confinement regions: - support regions; - Probable flexural yield regions; - Lap splice regions.

General requirements (walls seismic: ACI 318)

Maximum recommended axial force

As to impose the wall ductile behaviour axial force values are recommended to be kept low in the design of Special Reinforced Concrete Structural Walls resisting earthquake effects. The maximum axial force value in walls is recommended to be kept below the balanced point. Compression controlled walls should be avoided.

NOTE The current release of Tekla Structural Designer does not check if the axial force is below the balanced point.

Limiting Height

Buildings in which Special Reinforced Concrete Structural walls compose the SFRS in any of the main directions should have their height limited. Where dual systems of SRCSW and Moment Frames resisting at least 25% of the total shear exist there are no height restrictions.

NOTE The current release of Tekla Structural Designer does not check the maximum allowed building height based on Seismic Design Category.

Vertical Segment Classification

Isolated vertical segments within a wall - wall with openings - can be classified as wall segments or wall piers. For Special Reinforced Structural Walls and depending on the classification of the segment the governing design provisions can be split into provisions for walls and provisions for wall piers.

NOTE This is beyond scope in the current release of Tekla Structural Designer.

Mid-zone Reinforcement

Vertical and horizontal bars in mid-zones of Special Reinforced Concrete Structural Walls are designed according to the requirements of the following sections.

Minimum number of reinforcement layers

The minimum number of reinforcement layers allowed to be used in a Special Reinforced Concrete Structural Wall is governed by the amount of in-plane shear sustained by the wall

If the wall thickness b_w is greater than 250mm (10 in.) then at least two curtains of reinforcement are required.

If the wall thickness b_w is less than 250mm (10 in.) and the SFRS Type = Special Reinforced Concrete Structural Wall, then the minimum number of layers is dependent on the maximum shear force in the panel (ACI 318-11); or the maximum shear force and wall geometry (ACI 318-14). This is checked accordingly.

Additionally, reinforcement is required to be provided in both of the orthogonal directions in the wall plane - This requirement is automatically met as Tekla Structural Designer does not design walls with reinforcement in only one of the orthogonal directions in wall plane.

Minimum in plane reinforcement ratios

The minimum reinforcement ratio required in each of the orthogonal direction in the wall plane is dependent on the maximum panel factored shear force from analysis for seismic combinations and calculated as follows:

NOTE Grouped bars reinforcing the edges of the walls (end-zones) are not considered for the purpose of calculating reinforcement ratios.

IF SFRS Type = Special Reinforced Concrete Structural Wall then the maximum factored shear force at the panel is checked:

	If V_u		$V_{u,lim}$ >
	where		
	V_u		Maximum factored shear force in the wall panel obtained from the analysis for seismic combinations.
	$V_{u,lim}$		Minimum factored shear force in the wall above which horizontal and vertical main reinforcement minimum ratios need to be checked.
	$M_{nc,l,top}$, $M_{nc,r,top}$		Nominal Moment Strength of the stack above converging on the same joint for the axial force value consistent with the minimum Nominal Moment Strength respectively for the left and right sway cases.

Then a check for minimum reinforcement ratio in each orthogonal direction on the wall plane is performed as follows:

	ρ_l		ρ_{min} ≥

	ρ_t		ρ_{min} ≥
	where		
	ρ_l, ρ_t		Respectively the ratio of area of distributed vertical and horizontal reinforcement to gross concrete area perpendicular to each of those reinforcements.
	ρ_{min}		Minimum allowed ratio of reinforcement in the wall plane = 0.0025. =

Else if V_u		$V_{u,lim}$ ≤

ρ_l and ρ_t are allowed to be taken as the wall design conventional values.

Depending on the overall dimensions of a wall the vertical reinforcement ratio, ρl in Special Reinforced Concrete Structural Walls is limited to be of the same value or larger than the horizontal reinforcement ratio, ρ_t

End-zone Reinforcement

NOTE The seismic design requirements for end zone reinforcement are beyond scope in the current release of Tekla Structural Designer.

Shear Strength

Minimum shear strength

The basic design requirement for shear reinforcement in a wall is to have the reduced shear strength higher or the same as the maximum factored shear force at the considered section resultant from earthquake combinations. Some level of over-strength is expected when designing to multiple load combinations.

IF SFRS Type = Special Reinforced Concrete Structural Wall then the following check for shear strength is performed:

	ϕV_n		V_u ≥
	where		

	ϕ		Strength reduction factor. For purposes of checking the nominal shear strength = 0.6
	V_n		Maximum nominal shear strength at the considered panel. =
	V_u		Maximum factored shear force in the wall panel obtained from the analysis for seismic combinations.

Concrete slab design to ACI 318

Click the links below to find out more about the application of ACI 318 with regard to:

- Cover to Reinforcement (ACI 318) (page 88)
- Design parameters for longitudinal bars (page 91)
- Design for bending for rectangular sections (page 96)

If working to **ACI 318-19** click the below links to view additional important information:

- Slab and mat design (ACI 318-19 update) (page 83)

Pad and strip base design to ACI 318

Click the links below to find out more about the application of ACI 318 with regard to:

- Checks performed (page 186)
- Foundation bearing capacity (page 186)
- Design for shear (page 189)
- Check for transfer forces at column base (page 190)
- Check for transfer of horizontal forces by shear friction (page 191)
- Check for overturning forces (page 192)
- Check for sliding (page 192)
- Check for uplift (page 193)
- Checks for limiting parameters (page 193)

If working to **ACI 318-19** click the below link to view additional important information:

- Pad bases, strip footings & pile caps (ACI 318-19 update) (page 84)

See also

Cover to Reinforcement (ACI 318) (page 88)

Design for bending for rectangular sections (beams and slabs: ACI 318) (page 96)

Checks performed (pad and strip base:ACI 318)

The checks performed for both directions are:

- Max soil bearing pressure must not exceed allowable bearing pressure.
- Provided steel must be greater than As(min) for both vertical directions.
- Provided bar spacing must be inside the limiting spacing
- Provided bar size must be inside the limiting sizes
- Check for bending moment capacity
- Check for shear capacity - wide beam action at 'd' from column face
- Punching check at 'd/2' from column face - two-way action
- Check for transfer of forces at column base
- Check for transfer of horizontal forces by shear friction theory
- Check for overturning forces - **not in the current release**
- Check for sliding
- Check for uplift

Foundation Bearing Capacity (pad and strip base:ACI 318)

Check for Pad Base Bearing Capacity

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

Total base reaction:

T	=	$F_{swt} + F_{soil} + F_{dl,sur} + F_{ll,sur} - P$

Moment about X axis:

$M_{x,c}$	=	$M_{x,sup} - P * e_y - t_{ftg} * F_{y,sup}$

Moment about Y axis:

$M_{y,c}$		=	$M_{y,sup} + P * e_x + t_{ftg} * F_{x,sup}$

Where:

L_x		=	Length of foundation in X-direction
L_y		=	Length of foundation in Y-direction
A_f		=	$L_x * L_y$ = Foundation area
t_{ftg}		=	Depth of foundation
D_s		=	Depth of soil above the foundation
l_x		=	Length of column/wall in X-direction
l_y		=	Length of column/wall in Y-direction
A_c		=	cross section of the column/wall segment
e_x		=	eccentricity in X direction
e_y		=	eccentricity in Y direction
ρ_c		=	density of concrete
ρ_s		=	density of soil
F_{swt}		=	$A_f * t_{ftg} * \rho_c$ = foundation self-weight
F_{soil}		=	$(A_f - A_c)*D_s* \rho_s$ = soil self-weight
$F_{dl,sur}$		=	$(A_f - A_c)*sc_{dl}$ = Dead load from surcharge
$F_{ll,sur}$		=	$(A_f - A_c)*sc_{ll}$ = Live load from surcharge
sc_{dl}		=	Surcharge in dead loadcase
sc_{dl}		=	Surcharge in live loadcase
P		=	axial load acting on support in service combinations

$M_{x,sup}$		=	Moment acting on support around X-axis in service comb.
$M_{y,sup}$		=	Moment acting on support around Y-axis in service comb.
A_c		=	cross section of the column/wall
$F_{x,sup}$		=	Horizontal force acting on support X-direction in service comb.
$F_{y,sup}$		=	Horizontal force acting on support Y-direction in service comb.

Eccentricity of base reaction in X-direction:

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

Eccentricity of base reaction in Y-direction:

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

If abs(e_{Tx}) / L_x + abs(e_{Ty}) / L_y ≤ 0.167

Then base reaction acts within kern distance - no loss of contact in X-direction, and:

Pad base pressures:

q1		=	T/A_f – 6* $M_{y,c}$ / (L_x*A_f) + 6* $M_{x,c}$ / (L_y*A_f)
q2		=	T/A_f – 6* $M_{y,c}$ / (L_x*A_f) - 6* $M_{x,c}$ / (L_y*A_f)
q3		=	T/A_f + 6* $M_{y,c}$ / (L_x*A_f + 6* $M_{x,c}$ / (L_y*A_f)
q4		=	T/A_f + 6* $M_{y,c}$ / (L_x*A_f - 6* $M_{x,c}$ / (L_y*A_f)

Max base pressure:

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

Else base reaction acts outside kern distance - loss of contact.

In this case the pressure calculations are more complex - in Tekla Structural Designer these are done using sets of equations presented in an article by Kenneth E. Wilson published in the Journal of Bridge Engineering in 1997

Check for Strip Base Bearing Capacity

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

If abs(e_{Tx}) / L_x ≤ 0.167

Then - no loss of contact, and:

max base pressures for segment:

q_{max}	=	T/A_f + max[- 6* $M_{y,c}$ / (Lx*A_f) , 6*$M_{y,c}$ / (L_x*A_f)]

Else - loss of contact and

max base pressures for segment:

q_{max}	=	2*T/[3* L_y* (Lx /2 - abs(e_{Tx}))]

where

L_y = segment width

Design for shear (pad and strip base:ACI 318)

Pad base shear design check

The nominal shear strength of the concrete in beam action, is given by[1]

v_n	=	2*λ*MIN(√($f`_c$), 100psi)*d
	=	0.17*λ*MIN(√($f`_c$) ,8.3MPa)*d

where

λ	=	modification factor related to the density of the concrete

[1] ACI 318-08 Sections 11.1.2 and 11.2.1.1 Eqn (11-3)

λ	=	1.0 for normal weight concrete

If

v_u	$\leq$	$\Phi_{shear}*v_n$

Then the foundation thickness is adequate for shear -

Utilization ratio is then;

U-ratio	=	max [v_u / ($\Phi_{shear}*v_n$)]

Else the check has failed, the foundation thickness is inadequate.

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

Strip base shear design check

The principle of the strip base shear design check is similar to that for the pad base. Only the direction X is checked (around Y-axis) using segment widths.

Check for transfer forces at column base (pad and strip base:ACI 318)

This check applies when a concrete column is attached to the foundation.

Determinate the bearing strength of the column:

$\Phi*P_{nb,c}$	=	$\Phi_{bearing}$ (0.85*f'$_c$ * A_c)
If		
$\Phi*P_{nb,c}$	<	- P_u
Then check fails		
Else determinate the bearing strength of the footing:		
$\Phi*P_{nb,f}$	=	min [√(A_2/A_c), 2] * Φbearing (0.85*f'$_c$ * A_c)
where for rectangular columns:		
A_2	=	min {L_x , $2t_{ftg}$ + l_x+ min [(L_x-l_x)/2 - abs(e_x), $2t_{ftg}$]} * min {L_y , $2t_{ftg}$ + l_y + min [(L_y-l_y)/2 - abs(e_y), $2t_{ftg}$]}

NOTE Circular columns are treated as square members with the same area.

If

$\Phi*P_{nb,f}$		- Pu <
Then check fails		
Required min. area of dowel bars between column and footing is then:[1]		
$A_{s,min}$		$0.005*A_c$ =
Currently dowel bars are not designed.		
The area of the provided column reinforcement $A_{s,prov,column}$ is the same as the provided reinforcement of starter/dowel bars.		
If		
$A_{s,min}$		$A_{s,prov,column}$ >
Then check fails		

[1]: ACI 318-08 Section 15.8.2.1

Check for transfer of horizontal forces by shear friction (pad and strip base:ACI 318)

This check applies when a concrete column is attached to the foundation.

Determinate if the shear-friction design method applicable[1]:

When surface is not intentionally roughened (conservative assumption)

If	$V_u \le \Phi_{shear}$ * Ac min(0.2*f'_c , 800psi)	for US units
	$V_u \le \Phi_{shear}$ * Ac min(0.2*f'_c , 5.5MPa)	for metric units
	where	
	V_u = max[abs(Fy,sup,u) , abs(Fx,sup,u)]	

Then maximum shear transfer is permitted at the base of the column.

[1] ACI 318-08 Section 15.8.2.1

Required area of dowel reinforcement:

A_{vf}	=	V_u / (f_y * Φ_{shear}*μ)

where

μ = 0.6 when concrete not intentionally roughened (assumption)

μ = 1.0 when concrete intentionally roughened

μ = 1.4 when concrete placed monolithically

Currently dowel bars are not designed.

The area of the provided column reinforcement As,prov,column is the same as the provided reinforcement of starter/dowel bars.

If

A_{vf}	>	$A_{s,prov,column}$

Then check fails

Check for overturning forces (pad and strip base:ACI 318)

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

Check for sliding (pad and strip base:ACI 318)

The check for sliding is carried out for pad foundations only.

If there is no horizontal force acting on foundation check for sliding is not required.

Resultant Force on foundation:

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

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

where

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

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

Resistance to sliding due to base friction:

$H_{friction}$ = [-P + F_{swt}] * tanδ

where

δ = design base friction – user input

Passive pressure coefficient:

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

where

Φ' = design shear strength of soil – user input

Passive resistance of soil in X direction:

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

Passive resistance of soil in Y direction:

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

Resultant Passive Resistance:

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

Total resistance to sliding:

$R_{Hd} = (H_{friction} + H_{res,pas}) / 1.5$

If

$R_{Hd} \geq {}_{Hd}$

The check for stability against sliding passes

Check for uplift (pad and strip base:ACI 318)

For combinations producing tension at the support, the tension value is compared to the stabilizing loads and checked against a factor of safety (FOS).

Auto-design can automatically increment the base size to achieve a passing status.

The FOS considered for the uplift check is specified under Design Options > Concrete > Foundations > Isolated Foundations > General Parameters (default value = 1.50).

Checks for limiting parameters (pad and strip base: ACI 318)

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

Limits on bar size and reinforcement quantities

For structural foundations of uniform thickness the minimum areaof tensile reinforcement shall be:

For metric units:

IF Grade 280 to 350 deformed bars are used

As,min,reqd		≥	b*h*0.0020

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

As,min,reqd		≥	b*h*0.0018

IF yield stress exceeds 420 MPa

As,min,reqd		≥	b*h* [MAX(0.0014 , 0.0018*420/fy)]

For US-units:

IF Grade 40 to 50 deformed bars are used

As,min,reqd		≥	b*h*0.0020

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

As,min,reqd		≥	b*h*0.0018

IF yield stress exceeds 60000 psi

As,min,reqd		≥	b*h*[MAX(0.0014 , 0.0018*60000/fy)]
where			
b		=	unit width

The maximum area of tensile reinforcement shall be:

As,max		0.85*(f`c/fy)*β1*b*d*(3/7) ≤
where		
Ag		the gross area of the concrete section =
V		stress block depth factor[1] =

[1]: ACI 318-08, ACI 318-11, ACI 318M-08 and ACI 318M-11 Section 10.2.7.3

metric-units

β1		0.85	for f`c ≤ 28Mpa

		=	
		0.85 – 0.05 *[(f'c – 28MPa)/7MPa] =	for 28MPa < f`c < 55Mpa
		0.65 =	for f`c ≥ 55MPa

US-units

β1		min(max(0.85 - 0.05 ´ (f'c - 4 ksi) / 1ksi, 0.65), 0.85) =	
		0.85 =	for f`c ≤ 4000 psi
		0.85 – 0.05 *[(f'c – 4ksi)/1ksi] =	for 4000 psi < f`c < 8000psi
		0.65 =	for f`c ≥ 8000 psi

Pile cap design to ACI 318

The forces acting on a pile cap are applied to the foundation at the foundation level. The foundation can take axial load and bi-axial shear and moment.

Pile cap design is divided between pile design (pile capacity check) and structural design of the pile cap which includes bending, shear and punching shear design checks.

Click the links below to find out more about the application of ACI 318 with regard to:

- Pile axial capacity (page 196)
- Pile lateral capacity (page 196)
- Design for bending (page 196)
- Shear design (page 196)
- Checks for limiting parameters (page 197)

If working to **ACI 318-19** click the below link to view additional important information:

- Pad bases, strip footings & pile caps (ACI 318-19 update) (page 84)

Pile axial capacity (ACI 318)

The pile axial capacity is compared to the axial service load acting on pile:

Pile capacity passes if:

R_c	≥	$P_n \geq - R_t$
Where:		
R_c	=	Pile compression resistance
R_t	=	Pile tension resistance
P_n	=	Pile load

Pile lateral capacity (ACI 318)

NOTE This check is only performed if Check piles for lateral load is selected in Design Settings.

The pile lateral capacity is compared to the lateral service load acting on pile:

Pile capacity passes if:

H_R	≥	H_F
Where:		
H_R	=	Pile lateral resistance
H_F	=	Pile lateral load

Design for bending (pile cap:ACI 318)

The pile cap is treated as a beam in bending, where the critical bending moments for the design for the bottom reinforcement are taken at the face of the column.

The basic design method is identical to that for beams - see:Design for bending for rectangular sections (beams and slabs: ACI 318) (page 96)

Shear design (pile cap: ACI 318)

Pile shear capacity passes if:

v_{su}	$\leq$	$\Phi\ v_c$
and		
$v_{su,d}$	$\leq$	$\Phi\ v_{c,d}$

for both sides and directions.

Refer to CRSI Design Handbook 2002 - Chapter 13, page 13-18...13-21

Checks for limiting parameters (pile cap: ACI 318)

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

Check for distance of pile cap overhang

The check passes if:

min e_i		min (e_{min} , $e_{min,user}$) $>$		
where:				
e_{min}		MAX[230mm, 380mm - 0.5 * l_p] $=$	when $R_c \leq 534$ kN	Metric
e_{min}		MAX[9in, 15in - 0.5 * l_p] $=$	when $R_c \leq 120$ kips	US Customary
e_{min}		MAX[230mm, 530mm - 0.5 * l_p] $=$	when 534 kN $< R_c \leq$ 1068 kN	Metric
e_{min}		MAX[9in, 21in - 0.5 * l_p] $=$	when 120 kips $< R_c \leq$ 240 kips	US Customary
e_{min}		MAX[230mm, 685mm - 0.5 * l_p] $=$	when 1068 kN $< R_c \leq$ 1779 kN	Metric

e_{min}	=	MAX[9in, 27in - 0.5 * l_p]	when 240 kips < R_c ≤ 400 kips	US Customary
e_{min}	=	MAX[230mm, 760mm - 0.5 * l_p]	when R_c > 1779 kN	Metric
e_{min}	=	MAX[9in, 30in - 0.5 * l_p]	when R_c > 400 kips	US Customary
l_p	=	least width/diameter of the pile		

Check for minimum pile spacing

Check center to center spacing “s” between piles “i” and “j” in a pile group:

The check passes if:

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

where

$s_{min,user}$ = user input

s_{min} = max (least width of the pile + 0.6m , 0.9m) for metric units

s_{min} = max (least width of the pile + 2ft. , 3 ft) for US customary units[1]

Check for maximum pile spacing

Check center to center maximum spacing “s” between piles “i” and “j” in a pile group:

The check passes if:

If s_{ij} < smax,user

$s_{max,user}$ = user input

Other checks

The remaining checks are identical to those for pad bases.

See: Pad Base and Strip Footing Design - Checks for limiting parameters (pad and strip base: ACI 318) (page 193).

[1] CRSI – Design handbook page 13-18

Seismic Design to ACI 318

Reinforced concrete structures in buildings subjected to earthquake effects are designed elastically to the strains and displacements both from static and dynamic forces which they are subjected to. It is recognized that during an earthquake the building and its structural elements are very likely to be exposed to displacements well into their inelastic range and special precautions need to be taken as to increase the strength of critical sections in members which contribute to the building's lateral resistance while also contributing to the ductile behavior of the building in order to allow for the dissipation of induced stresses.

In the case of reinforced concrete structures, particular design and detailing requirements (provisions) need to be fulfilled beyond the conventional design of the elements as to provide them with ductile response capabilities. Such requirements are mainly addressed to structural elements part of structural systems built for the purpose of resisting seismic lateral forces - Seismic Force Resisting Systems [SFRS]. Seismic provisions also apply to reinforced concrete elements not part of the SFRS when the building is assigned to a higher Seismic Design Category - SDC D, E or F.

The seismic design checks and detailing requirements of reinforced concrete members in Tekla Structural Designer are based mainly on ACI318, Chapter 21 - Earthquake-Resisting Structures.

Seismic Force Resisting Systems

The level of design and detailing required of members that are part of a seismic resisting structural system can differ depending on the amount of toughness they are intended to provide to the building. ACI318-11 groups the main structural systems into "Ordinary", "Intermediate" and "Special" groups. Different types of structural systems have limitations to their application in each of the Seismic Design Categories.

Structural System	Allowed in SDC
Ordinary Moment Frames	A, B
Ordinary Cast in Place Structural Walls	A, B, C
Intermediate Moment Frames	A, B, C
Intermediate Precast Walls	A, B, C
Special Moment frames	A, B, C, D, E, F
Special Structural Walls (Precast / Cast in Place)	A, B, C, D, E, F

As the current release of Tekla Structural Designer does not fully include the design requirements for all the Seismic Force Resisting Systems, SFRS types have been classified as included or excluded from the member design.

Members in included SFRS types are fully covered for seismic design provisions while those in excluded types are covered to a limited extent only.

- Seismic Force Resisting Systems included in the design:
  - Intermediate Moment Frames
  - Ordinary Moment Frames
  - Ordinary Reinforced Concrete Structural Walls
- Seismic Resisting Systems excluded from the design:
  - Special Moment Frames
  - Special Reinforced Concrete Structural Walls
  - Intermediate Precast Structural Walls

Consequently the current release of Tekla Structural Designer can be said to consider the design requirements for each of the Seismic Design Categories as follows:

Seismic Design Category	Seismic Requirements
SDC A	N/A
SDC B	Considered
SDC C	Considered
SDC D, E, F	Not Considered

Materials

Additional seismic material requirements apply to:

- concrete beams and columns assigned to a Special Moment Frame SFRS
- concrete walls assigned as Special Reinforced Concrete Structural Walls

Concrete Compressive Strength

The requirements for compressive strength of concrete are limited:

Minimum compressive strength of normal weight concrete	f'_c	=	21 MPa (3,000 psi)
Maximum compressive strength of lightweight concrete	f'_c	=	35 MPa (5,000 psi)

Reinforcement Steel

Reinforcement steel shall comply with ASTM 706(M), Grade 420 (60,000 psi).

$(f_y)_{actual}$ - $(f_y)_{specified}$	$\leq$	125 MPa (18,000 psi)
$(f_u)_{actual}$ / $(f_y)_{actual}$	$\geq$	1.25

where,

$(f_y)_{actual}$	=	Actual yielding strength of the reinforcement based on mill tests, MPa (psi)
$(f_y)_{specified}$	=	Specified yield strength of reinforcement, MPa (psi)
$(f_u)_{actual}$	=	Actual ultimate tensile strength of the reinforcement, MPa (psi)

Reinforcement Characteristic Yield Strength

Requirements for the characteristic yield strength of the reinforcement steel are:

- Maximum allowed characteristic yield strength of longitudinal reinforcement: f_y = 420 MPa (Grade 60 - 60,000 psi);
- Maximum allowed characteristic yield strength of shear reinforcement: f_{yt} = 420 MPa (Grade 60 - 60,000 psi);

ACI 318-19

If working to **ACI 318-19** click the below links to view additional important information:

- Seismic design - general (ACI 318-19 update) (page 86)
- Concrete beam seismic design (ACI 318-19 update) (page 87)
- Concrete column seismic design (ACI 318-19 update) (page 87)
- Concrete wall seismic design (ACI 318-19 update) (page 87)

References ACI 318

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

4. American Concrete Institute. *Metric Building Code Requirements for Structural Concrete (ACI 318M-11) and Commentary.* ACI 2011.
5. American Concrete Institute. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary.* ACI 2014.
6. American Concrete Institute. *Metric Building Code Requirements for Structural Concrete (ACI 318M-14) and Commentary.* ACI 2014.
7. American Concrete Institute. *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary.* ACI 2019.
8. American Concrete Institute. *Metric Building Code Requirements for Structural Concrete (ACI 318M-19) and Commentary.* ACI 2019.

1.5 Vibration of floors to DG11

These topics describe the DG11 floor vibration calculations that can be performed in Tekla Structural Designer.

The following topics are covered:

Introduction to DG11 floor vibration

This handbook describes the DG11 floor vibration calculations that can be performed in Tekla Structural Designer.

With the advent of long span floors, multiple openings in webs, minimum floor depth zones etc. the vibration response of floors in multistory buildings under normal occupancy has increasingly become of concern to clients and their Engineers and Architects.

Detailed guidance on the subject is available through the AISC Steel Design Guide Series 11: Vibrations of Steel-Framed Structural Systems Due to Human Activity Second Edition, otherwise referred to as DG11. (page 222)

This handbook describes the method for the assessment of floor vibration in accordance with DG11 that has been adopted in Tekla Structural Designer. The method seeks to establish, with reasonable accuracy, the response of the floor to dynamic excitation expected in offices of normal occupancy. This excitation is almost solely based on occupants walking. With appropriate design criteria, the approach is likely to be equally applicable to sectors other than offices.

The existing solution to checking this type of criterion - a simple calculation of the natural frequency of an individual beam - is felt in many cases to be insufficiently accurate. More importantly, such calculations do not consider two important factors,

- the natural frequency is only the 'response side' of the equation. The 'action' side of the equation is also important i.e. the dynamic excitation - this is the activity that might cause an adverse response from the floor. Walking, dancing and machine vibration are all on the 'action' side of the equation and are all very different in their potential effect.
- the natural frequency of an isolated beam is exactly that and takes no account of the influence (good or bad) of the surrounding floor components. In particular, with composite floors, the slabs will force other beams to restrict or sympathize with the beam under consideration.

The culmination of the calculations carried out by Tekla Structural Designer is the calculation of the peak acceleration of the floor system due to walking excitation expressed as a fraction of the acceleration of gravity.

Scope of DG11 floor vibration

The reference upon which Tekla Structural Designer's floor vibration check is based is the main limiting factor with regard to scope. This publication is AISC Steel Design Guide Series 11: Vibrations of Steel-Framed Structural Systems Due to Human Activity Second Edition, otherwise referred to as DG11. (page 222)

You are able to define an area on a particular floor level that is to be subject to the vibration response analysis and design. The layout of beams in real multi-story buildings can be of almost any configuration. **The methodology adopted in DG11 is only applicable to regular structures which by and large have to be created from rectilinear grids.** It is your responsibility to make an appropriate selection of the beams etc. that are to be the basic components of the idealized case.

As you proceed through the input making your selections, Tekla Structural Designer will, where it is possible to do so, interrogate the underlying model and retrieve the appropriate data. Once all the data has been assembled, you are then able to perform the check, after which a detailed set of results will be available for review. If you are unhappy with the outcome of your choices you can close the results window and make alternative selections by editing the Floor Vibration Check item properties.

Limitations and Assumptions of DG11 floor vibration

The scope is primarily defined by the DG11 (page 222) but the following additional limitations and assumptions should be noted.

- The design guidance is based on composite floors acting compositely with the steel beams. It is unclear whether the design approach is directly applicable to non-composite construction.
- In DG11, if the slab is attached to the supporting member, the construction can be classed as composite for the purpose of carrying out a vibration analysis even in the absence of shear connectors. Tekla Structural Designer does not define such construction as composite and therefore will only class truly composite construction as composite.
- For simplicity and to avoid the necessity of Tekla Structural Designer having to identify all the beams in the area selected for vibration assessment, the component of the unit mass from the self-weight of the beams is ignored. This will lead to a slight inaccuracy in the participating mass that is conservative (more mass is advantageous). Note, however, that beam self-weight is included in the calculation of beam deflection but only when the self-weight loadcase is included in the load combination.
- Cantilever beams are excluded from the analysis.
- In DG11, if a beam or girder is moment connected to a column, its natural frequency can be enhanced because of the flexural restraint offered by the columns. Tekla Structural Designer does not incorporate this feature.
- Shear deflection in beams and girders is included in the analysis carried out by Tekla Structural Designer.
- Precast slabs are excluded.

Design philosophy of DG11 floor vibration

General

The Engineer ensures the safety of building occupants by satisfying all design criteria at the Ultimate Limit State. Similarly, the health of building occupants is partly taken care of when deflection limits at the Serviceability Limit State are satisfied (although this Limit State does have other purposes than simply the health of occupants).

However, for floors that are subject to cyclic or sudden loading, it is the human perception of motion that could cause the performance of a floor to be found unsatisfactory. Such perception is usually related to acceleration levels. In most practical building structures, the reaction of the occupants to floor acceleration varies between irritation and a feeling of insecurity. This is based on the instinctive human perception that motion in a 'solid' building indicates inadequacy or imminent failure.

The working environment also affects the perception of motion. For busy environments, where the occupant is surrounded by the activity that is producing the vibrations, the perception of motion is reduced. In contrast, for quieter environments (such as laboratories and residential dwellings), where the source of vibration is unseen, the perception of motion is significantly heightened.

The design philosophy to ensure that the potential for such human response is minimized, has a number of facets,

- the **dynamic excitation** causing the vibration i.e. the disturbing force profile, which is force and time dependent. For the sorts of building and occupancy considered here, this is the act of walking.
- the **acceptance criteria**. This depends upon the type of environment. As discussed above this, in turn, depends upon the involvement of the occupant in the generation of the vibration and also on the nature of the occupancy. The latter is important for laboratories carrying out delicate work, or operating theaters, for example.
- the **provided performance**. This is the "resonance response function" and is dependent on the system natural frequency and, more importantly, the participating mass. This function is expressed as a ratio of the floor acceleration to the acceleration of gravity.

Dynamic excitation

In a classical spring-mass system that includes a (viscous) damper, when a simple force is applied to the mass to extend (or contract) the spring, the mass moves up and down (oscillates). This movement is significant at first but eventually reduces to zero due to the resistance offered by the damper. In a floor system in a building,

- the mass is the self-weight of the floor and any other loading that is present for the majority of the time that the occupants could be exposed to vibration effects,
- the spring is the stiffness of the floor system, which will have a number of different component beams (joists and girders) and the floor slab,
- the damper is provided by a number of elements that are able to absorb energy from the free vibration of the system. There will be energy absorbed,
  - within connections, since they behave 'better' than the ideal that is assumed
  - from losses due to the unsymmetrical nature of real buildings e.g. grid layout, and dispersion of loads from furnishings and contents
  - from components such as partitions that are out-of-plane of the vibration and interfere with the 'mode'.

The determination of the contribution of each of these components as they affect real floor systems is given in detail in later sections. These describe the 'response' side of the floor system. In order to establish the required performance of the system the 'input' must also be defined i.e. that event, events or continuum that is the 'dynamic excitation'.

In the simple example described at the start of this section the 'input' was simply a force that caused a displacement to the system and was then released. This might be equivalent to a person jumping off a chair onto the floor. However, in the context of the concerns over the vibration of floors, it is not this sort of input that is of interest. The main concern is the excitation of the floor brought about by walking.

Unlike the simple example, walking produces loading that is cyclic. This loading can be idealized into a series of sine curves of load against time. Each curve is an exact multiple of the walking frequency called harmonics. When one of these harmonics of the cyclic loading coincides with the natural frequency of the floor system then resonance is set up. The consequence of resonance that is detected, and may disturb occupants, is the associated peak acceleration. The peak acceleration due to walking is estimated by selecting the lowest harmonic for which the forcing frequency can match a natural frequency of the floor and is dependent upon the applied force (a constant = 0.29kN [65lb] for floors), the mass of the system (the self-weight of the floor plate plus other loading that could be considered as permanent), and the amount of damping in the system (the damping ratio, β).

Hence, the dynamic excitation of a floor is dependent upon the forcing function due to walking and its relationship to the natural frequency of the floor system. It is the level of the peak acceleration that this generates that is particularly important in determining the performance of the floor.

Acceptance criteria for human comfort

The required performance of a floor system is very dependent upon the potential response of humans. Human response is a very complex subject since there is no such thing as a 'standard human'. The perception of vibration will differ from person to person, their body mass varies significantly and the body's reaction will depend upon age, gender etc. The human response has been studied and the acceptance criterion adopted by DG11 was developed using the acceleration limits as recommended by ISO 2631-2, 1989 adjusted for intended occupancy.

The accelerations acceptable for different use of buildings are described using the 'base' limits. Multiplying factors are used to increase the base acceleration limit according to the intended use of the building. The target acceleration ratio of the floor under consideration is given in DG11 guidance as,

- a_o/g = 0.5% for offices, residences, churches, schools and quiet areas
- a_o/g = 1.5% for shopping malls

You should choose a required acceleration ratio based on both engineering judgement and the advice given in DG11.

A separate acceleration ratio limit for high frequency floors (i.e. those floors with fundamental frequency, f_n, in the range $9 < f_n <= 15$ Hz) also needs to be defined by the Engineer, again based on engineering judgement and with reference to DG11 figure 2-1

Design for walking excitation

The start point is the calculation of the natural frequency of the floor system. The fundamental floor frequency, f_n, is evaluated using the Dunkerley relationship for the combined mode.

In accordance with the guidance given at Section 4.1 of DG11, an advisory Warning is displayed when the fundamental frequency of the floor system, f_n, is < 3.0 Hz.

Next the 'Equivalent Panel Weight' is required. This is dependent upon the physical size of the floor plate selected and an effective width and/or length.

The calculation requires the 'damping ratio' - this is a user input.

The damping ratio, the effective panel weight, the fundamental frequency of the floor and the constant excitation force are used to calculate the peak acceleration ratio, a_p/g.

The design condition is simply,

$a_p/g \le a_o/g$

For floors with high fundamental frequencies, when the calculated value of f_n is in the range 9 Hz < $f_n \le 15$ Hz, then an equivalent sinusoidal peak acceleration, a_{ESPA}, is checked against the separate acceleration ratio limit for high frequency floors.

Design for walking excitation DG11

Floor slab

Slab Properties

For composite slabs the transformed moment of inertia per unit width of the slab, D_s, is calculated from,

D_s	$d_e^3 /(12*n)$	mm^4/mm in metric units
D_s	$12*d_e^3/(12*n)$	ins^4/ft in US Customary units

Where

d_e	=	effective depth of slab taken as slab depth less one half depth of steel decking (mm or inches)
n	=	dynamic modular ratio
	=	E_s /(1.35*E_c)
E_s	=	the steel modulus (N/mm^2 or ksi)
E_c	=	the concrete modulus (N/mm^2 or ksi)

For generic slabs, the transformed moment of inertia per unit width is to be provided by the user.

Beam panel mode

Beam Panel Mode Deflection

The beam panel mode deflection, Δ_j, is the maximum simply supported deflection of the beam or joist calculated using,

Δ_j	=	5 * ((w*b)+w_{swt}) * L_j^4 / (384*E_s *I_{sj})

where

w	=	unit supported weight (psi or N/mm^2)
b	=	beam or joist spacing (in or mm)
w_{swt}	=	beam or joist self weight (zero unless self weight loadcase in combination) (pli or N/mm)
L_j	=	span of beam or joist (in or mm)
E_s	=	steel modulus (psi or N/mm^2)

I_{sj}	=	the inertia of the beam or joist from the database (in^4 or mm^4)

Beam Panel Mode Frequency

The beam panel mode fundamental frequency, f_j, is given by,

f_j	=	$0.18*\sqrt{(g/\Delta_j)}$ Hz
Where		
g	=	the acceleration of gravity
Δ_j	=	the maximum simply supported deflection of the beam or joist calculated as above.

Beam Panel Mode Effective Weight

The effective panel weight for the beam panel mode, W_j, is given by,

W_j	=	$k_j *w*B_j *L_j$
Where		
k_j	=	beam continuity factor
	=	1.0 generally but 1.5 where beams are continuous over supports and an adjacent span is > $0.7*L_j$
w	=	unit supported weight (see Data Derived from Tekla Structural Designer (page 217))
B_j	=	the effective width of the beam panel
	=	$C_j *(D_s /D_j)^{0.25} *L_j$ but ≤ (2/3)*FW
L_j	=	the span of the beam
and		
C_j	=	effective width coefficient for beam

	=	2.0 generally but 1.0 for beams parallel to interior edge
D_s	=	transformed moment of inertia of slab per unit width as above
D_j	=	transformed moment of inertia of beam per unit width
	=	I_j /S
S	=	beam spacing
FW	=	floor width
	=	n_g *L_g
n_g	=	number of bays in the direction of the girder span.
L_g	=	the span of the girder

Girder panel mode

Girder Panel Mode Deflection

The girder panel mode deflection, Δ_g, is the maximum girder deflection derived from the analysis model.

NOTE As Δ_g is taken directly from load analysis there is no need for any adjustment to the girder deflection, as suggested in DG11 section 3.1, when there is only one supported beam.

Girder Panel Mode Frequency

The girder panel mode fundamental frequency, f_g, is given by,

f_g	=	$0.18*\sqrt{(g/\Delta_g)}$ Hz

Where

g	=	the acceleration of gravity
Δ_g	=	the maximum simply supported deflection of the girder derived from the analysis model.

Girder Panel Mode Effective Weight

The effective panel weight for the girder panel mode, W_g, is given by,

W_g	=	k_g * w *B_g * L_g

Where

k_g	=	girder continuity factor
	=	1.0 generally but 1.5 where girders are continuous over supports and an adjacent span is > 0.7*L_g
w	=	unit supported weight (see Data Derived from Tekla Structural Designer (page 217))
B_g	=	the effective width of the girder panel

For the general case,

B_g	=	C_g *(D_j /D_g)$^{0.25}$ *L_g but ≤ (2/3)*FL

If the girder is an interior edge girder as specified by the user,

B_g	=	L_j*(2/3)

Where

L_g	=	the span of the girder
C_g	=	effective width coefficient for beam
	=	1.8 generally but 1.6 for girders supporting joists connected to the girder flange (joist seats)
D_j	=	transformed moment of inertia of beam per unit width as above
D_g	=	transformed moment of inertia of girder per unit width
	=	I_g/L_j generally but
	=	2*(I_g/L_j) for edge girders
FL	=	floor length

	=	$n_j * L_j$
n_j	=	number of bays in the direction of the beam span.

If the girder supports beams with unequal spans, say L_{j1} and L_{j2}, the average beam span length $L_{av} = (L_{j1}+L_{j2})/2$ should replace L_j in the above equation for D_g. The user should confirm the value to be used in such circumstances.

Combined panel mode

There are three possible conditions is to be checked for the combined mode.

If $L_g/B_j > 1.0$, the combined equivalent panel weight, W_{comb}, is given by,

W_{comb}	=	$(\Delta_j/(\Delta_j+\Delta_g))*W_j + (\Delta_g/(\Delta_j+\Delta_g))*W_g$

The floor fundamental frequency, f_{comb}, is given by

f_{comb}	=	$0.18*\sqrt{g/(\Delta_j+\Delta_g)}$

If $0.5 \le L_g/B_j \le 1.0$, the combined equivalent panel weight, W_{comb}, is given by,

W_{comb}	=	$(\Delta_j/(\Delta_j+\Delta_{gred}))*W_j + (\Delta_{gred}/(\Delta_j+\Delta_{gred}))*W_g$

The floor fundamental frequency, f_{comb}, is given by

f_{comb}	=	$0.18*\sqrt{g/(\Delta_j+\Delta_{gred})}$

Where

Δ_{gred}	=	$(L_g/B_j)*\Delta_g$

If $L_g/B_j < 0.5$, the combined equivalent panel weight, W_{comb}, is given by,

W_{comb}	=	$(\Delta_j/(\Delta_j+\Delta_{gred}))*W_j + (\Delta_{gred}/(\Delta_j+\Delta_{gred}))*W_g$

The floor fundamental frequency, f_{comb}, is given by

f_{comb}	=	$0.18*\sqrt{g/(\Delta_j + \Delta_{gred})}$

Where

Δ_{gred}	=	$0.5* \Delta_g$

In addition, if $L_j/L_g < 0.5$, then the peak acceleration ratio is separately checked for the **beam panel mode** and for the **combined panel mode** as above.

Evaluation

The peak acceleration ratio, a_p/g, is evaluated for each f_n in turn (with its associated W), and is given by,

a_p/g	=	MAX[$100*P_0*e^{(-0.35*fn)}/(\beta*W)$] %

where

f_n	=	f_j, f_g, f_{comb}
W	=	the value of W_j, W_g or W_{comb} appropriate to f_n
P_0	=	constant force equal to 0.290 kN [65lb]
β	=	damping ratio

The acceleration limit, a_o/g, is a user input and leads to the final design condition,

a_p/g	≤	a_o/g

NOTE The ‘fundamental frequency of the floor’ output in the results is the f_n associated with the MAX peak acceleration ratio (and not just the MIN f_n).

High Frequency Floors

For floor systems having a natural frequency greater than 9 Hz (but ≤ 15 Hz), the equivalent sinusoidal peak acceleration ratio, a_{ESPA}/g, is given by

a_{ESPA}/g	=	MAX[100*(154 / W)*($f_{step}^{1.43}$/ $f_n^{0.3}$)*{[1-	US-units

$e^{(-4*\pi*h*\beta)}]/(h*\pi*\beta)\}^{0.5}]$ %

	=	$MAX[100*(686 / W)*(f_{step}^{1.43}/f_n^{0.3})*\{[1-e^{(-4*\pi*h*\beta)}]/(h*\pi*\beta)\}^{0.5}]$ %	metric-units

where

W	=	the value of W_j, W_g or W_{comb} appropriate to f_n
f_{step}	=	footstep frequency Hz
f_n	=	fundamental frequency of the floor
h	=	the harmonic matching f_n
	=	5 for 9 Hz < f_n ≤ 11 Hz
	=	6 for 11 Hz < f_n ≤ 13.2 Hz
	=	7 for 13.2 Hz < f_n ≤ 15 Hz
β	=	damping ratio

NOTE The a_{ESPA}/g equation assumes a bodyweight value of 168 lbs [0.75 kN] as indicated in DG11.

The acceleration limit for high frequency floors is a user input and leads to the final design condition,

a_{ESPA}/g	≤	acceleration limit for high frequency floors

Sensitive use analysis DG11

The vibration check calculations can be performed for sensitive equipment & occupancy if required.

These calculations make use of Chapter 6 of DG11 2nd Edition (2016) with revisions and errata of 27 July 2018. They only cover ⅓ octave spectral velocity and acceleration.

For these calculations the mode shape factors ϕ_E and ϕ_W are taken as 1.0. i.e. it is conservatively assumed that the walker and sensitive equipment or sensitive occupant are both at mid-bay.

Use Case - Sensitive Equipment

If walking speed = Very Slow ,

f_{step}	$= 1.25$ Hz	[Table 6-1]
$V_{1/3}$	$= (250 * 10^6) / (\beta * W) * (f_{step}^{2.43} / f_n^{1.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-3a]
$A_{1/3}/g$	$= (100 * 4.2) / (\beta * W) * (f_{step}^{2.43} / f_n^{0.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-8a]

Else if walking speed = Slow , or Moderate or Fast

f_{step} , f_L , f_U and γ as appropriate to selected walking speed	[Table 6-1]

If $f_n \leq f_L$

$V_{1/3}$	$= (175 * 10^6) / (\beta * W * f_n^{0.5}) * (e^{-\gamma*fn})$	[Eqn 6-3b][1]
$A_{1/3}/g$	$= (100 * 6.4) / (\beta * W) * (e^{-\gamma*fn})$	[Eqn 6-8b][1]

If $f_n \geq f_U$

$V_{1/3}$	$= (250 * 10^6) / (\beta * W) * (f_{step}^{2.43} / f_n^{1.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-3b][2]
$A_{1/3}/g$	$= (100 * 4.2) / (\beta * W) * (f_{step}^{2.43} / f_n^{0.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-8b][2]

Else if $f_L < f_n < f_U$

For $V_{1/3}$

1. evaluate [Eqn 6-3b][1] with $f_n = f_L$, then
2. evaluate [Eqn 6-3b][2] with $f_n = f_U$, then
3. linear interpolate for actual f_n

For $A_{1/3}$/ g

1. evaluate [Eqn 6-8b][1] with $f_n = f_L$, then
2. evaluate [Eqn 6-8b][2] with $f_n = f_U$, then
3. linear interpolate for actual f_n

Use Case - Sensitive Occupancy

NOTE DG11 2nd Edn does not provide explicit ⅓ octave spectral Acceleration equations for **Sensitive Occupancy**, so in Tekla Structural Designer these equations have been derived from the ⅓ octave spectral Acceleration equations for **Sensitive Equipment** by ‘modifying’ them by factors 200/250 (for very slow walking) and 120/175 (for other walking speeds).

These modification factors are derived from consideration of the differences between ⅓ octave spectral Velocity equations for Sensitive Equipment and Sensitive Occupancy, which are given explicitly in DG11 2nd Edn.

If walking speed = Very Slow ,

f_{step}	1.25 Hz	[Table 6-1]
$V_{1/3}$	$(200 * 10^6) / (\beta * W) * (f_{step}^{2.43} / f_n^{1.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-9a]
$A_{1/3}/g$	$(200/250)*(100 * 4.2) / (\beta * W) * (f_{step}^{2.43} / f_n^{0.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-8a] modified

Else if walking speed = Slow , or Moderate or Fast

f_{step} , f_L , f_U and γ as appropriate to selected walking speed		[Table 6-1]

If $f_n \leq f_L$

$V_{1/3}$	$(120 * 10^6) / (\beta * W * f_n^{0.5}) * (e^{-\gamma * fn})$	[Eqn 6-9b][1]
$A_{1/3}/g$	$(120/175)*(100 * 6.4) / (\beta * W) * (e^{-\gamma * fn})$	[Eqn 6-8b][1] modified

If $f_n \geq f_U$

$V_{1/3}$	$(200 * 10^6) / (\beta * W) * (f_{step}^{2.43} / f_n^{1.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-9b][2]
$A_{1/3}/g$	$(200/250)*(100 * 4.2) / (\beta * W) * (f_{step}^{2.43} / f_n^{0.8}) * (1 - e^{-2*\pi*\beta*fn / fstep})$	[Eqn 6-8b][2] modified

Else if $f_L < f_n < f_U$

For $V_{1/3}$

1. evaluate [Eqn 6-9b][1] with $f_n = f_L$, then
2. evaluate [Eqn 6-9b][2] with $f_n = f_U$, then
3. linear interpolate for actual f_n

For $A_{1/3}$/ g

1. evaluate [Eqn 6-8b][1] modified with $f_n = f_L$, then
2. evaluate [Eqn 6-8b][2] modified with $_n = f_U$, then
3. linear interpolate for actual f_n

Input requirements for DG11 floor vibration

General

The simplified method for the analysis of the vibration of floors given in the AISC Publication DG11, on which on which the Tekla Structural Designer check is based, is only applicable to regular structures which, by and large, are created from rectilinear grids.

Of course the floor layouts of 'real' multi-storey buildings are rarely uniform and Tekla Structural Designer therefore provides you with the opportunity to select the more irregular floor areas to be assessed with grids that are other than rectilinear.

In so far as the selection of the beams and girders to be used in the analysis is concerned, only beams or girders with Non-Composite, Steel Joist or Composite attributes are valid for selection and, within these confines, the user should be able to:

- select a single beam or girder
- select a girder span as critical plus an adjoining span (in a two or three span configuration)

In all cases, and subject to the above restrictions, which beams and girders from the selected area of floor are chosen is entirely at your discretion and under your judgment, but it is expected that the beams and girders chosen will be those that are typical, common or the worst case. Irrespective,Tekla Structural Designer will take these beams as those that form the idealized floor layout. There is no validation on what the you select (although there is some validation on which beams and girders are selectable i.e. those which have no slab for part of their length, those from angle sections, those with no adjoining span when a 2-span configuration is chosen, and those with no adjoining span at both ends when a 3-span configuration is chosen will not be selectable).

Data Derived from Tekla Structural Designer

Note that, where appropriate, the derived data is for each design combination under SLS loads only.

Unit supported weight

The unit supported weight is used to establish the 'effective panel weight' - that is the weight of the floor and its permanent loading that has to be set in motion during vibration of the floor. It is taken as the slab self-weight (and to be accurate, the beam self-weight), other permanent 'Dead' loads and the proportion of the 'Live' loads (expressed as a percentage) that can be considered as permanent.

The unit supported weight is obtained by summing all the loads (or the appropriate percentage in the case of live loads) that act over or in the selected area. This includes any blanket, area, line and spot loads that are present within the selected area. The component of any of these load types that lie outside of the selected area are ignored. Nodal loads directly on columns are also ignored. The total load is then divided by the area selected.

The slab self-weight will usually be in the Slab Dry loadcase - note that in the case of composite slabs this includes the weight of decking. The beam self-weight is in a separate protected loadcase. For simplicity this component of the unit supported weight is ignored. This leads to a slight inaccuracy in the participating mass that is conservative (more mass is advantageous).

Note that the use of live load reductions has no effect on the floor vibration check.

Slab data

If there are more than one set of slab attributes in the selected area then you have to choose which of these it is appropriate to use. From the designated slab attributes the following information/data is obtained,

- the transformed inertia in mm^4 per mm width [in^4/ft].
- the short-term modular ratio for normal or lightweight concrete as appropriate.

If the designated slab attributes are for a 'generic' slab, then you are asked for the transformed inertia and the dynamic modular ratio.

Beam data

When these are non-composite beams, the inertia is obtained from the sections database. When these beams are of composite construction the inertia is the gross, uncracked composite inertia based on the dynamic modular ratio that is required. Steel joist inertias from the database are assumed to be 'gross' inertias of the chords and are editable. Following guidance contained in AISC Steel Design Guide 11 (page 222), section 3.5, the gross steel joist inertia is factored by quantity C_r and displayed as the 'effective' inertia in the results viewer. C_r is derived from the following formulas,

For joists with angle web members:

C_r	$= 0.90*(1 - e^{-0.28(L/D)})^{2.8}$	valid for 6 ≤ L/D ≤ 0.9

For joists with rod web members:

C_r	$= 0.721 + 0.00725*(L/D)$	valid for 10 ≤ L/D ≤ 0.

Where

L	=	Joist span
D	=	Joist nominal depth

NOTE There is no guidance in DG11 what to do if L/D < 6 for angle web members or L/D < 10 for rod web members. Therefore, in these cases Tekla Structural Designer calculates C_r with L/D = 6 (angle) or L/D = 10 (rod).

The span of the critical/base beam and the adjoining beams is required.

The deflection of the critical beam under the permanent loads is required. To calculate this value, the deflection under the Dead loads and the appropriate percentage of the Live load deflection is summed.

Girder data

The same data is required as that for the beams.

Floor plate data

The dimensions of the floor plate in the idealized cases are defined in one direction by the number of beam bays and in the orthogonal direction by the number of girder bays. In practice, given that the idealized case may not attain, the floorplate dimensions are derived from the slab items you select as participating in the mass.

User Input Data

Secondary Beam Spacing

You must confirm the spacing of the secondary beams - an average value when the spacing is non-uniform.

Permanent Live Loads

You are required to specify the proportion of the permanent live loads that are to be used in the vibration analysis as a percentage of the live load.

Beam Continuity Factor

This has a value of either 1.0 or 1.5 and you are offered the choice when a two or three span beam configuration is selected. The default value is 1.0.

Girder Continuity Factor

This has a value of either 1.0 or 1.5 and you are offered the choice when a two or three span girder window is selected. The default value is 1.0.

This factor is not generally applicable for girders as they usually frame directly into columns and this potential increase in the effective panel weight does not apply under those circumstances.

Beam and Girder location

You are required to specify whether the beam and the girder under consideration is an "internal" location or an "internal edge" location. This information is required to set the constants that are used in calculating the effective widths of the beam and girder panel modes.

Number of bays used to establish effective panel weight

You are required to specify the number of bays in the direction of the beam span, nj, and the number of bays in the direction of the girder span, ng, that are to be used to establish the effective panel weight. The number of bays ranges from 1 to 4 for both directions.

Damping ratio

Floors do not vibrate as a free mass but have some damping i.e. dissipation of the energy in the system. Values of the damping ratio for individual components, β_i are recommended in DG11 as,

Recommended Component Damping Values	
Component	**Ratio of Actual Damping-to-Critical Damping, β_i**
Structural System	0.01
Ceiling and ductwork	0.01
Electronic office fit-out	0.005
Paper office fit out	0.01
Churches, schools and malls	0.0
Lightly furnished quiet spaces	0.005
Full-height dry wall partitions in bay	0.02 to 0.05*
*Depending on the number of partitions in the bay and their location; near the center of the bay provides more damping.	

For example, a floor with ceiling and ductwork supporting an electronic office area has $\beta = \Sigma\beta_i$ = 0.01 (floor) + 0.01 (ceiling and ductwork) + 0.005 (electronic office area) = 0.025, or 2.5% critical damping.

Since an even higher damping ratio might be justified for storage floors for example, a range of up to 10% is offered.

Acceleration limit

You must enter acceleration limits appropriate to the floor under consideration, one limit for floors with frequencies in the range 3-9 Hz and a separate limit for High Frequency floors (in the range 9-15 Hz). These limits will be based on your engineering judgement and the advice given in DG11 - which gives a range of values between 0.5% and 5.0% depending on structural form (for building floors see the table below).

Table 4-1. Recommended Tolerance Limits for Building Floors

Occupancy	Acceleration Limit a_o/g x 100%
Offices, residences, churches, schools and quiet areas	0.5%
Shopping Malls	1.5%

Footstep frequency

You should enter the footstep frequency to be used if a high frequency floor is detected. The range being 1.2 Hz to 2.2 Hz.

Sensitive use

You have to specify the Use case and walking speed if this analysis is performed. See: Sensitive use analysis DG11 (page 214)

Vibration of floors to DG11 references

1. AISC Steel Design Guide Series.*11: Vibrations of Steel-Framed Structural Systems Due to Human Activity Second Edition (2016) with revisions and errata of 27 July 2018.*

Index

A

B

C

S

T

U

V

W