midas nGen

Design Manual

DISCLAIMER

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Section 1 Outline

The design capabilities of midas nGen 7 Foundation include steel design, reinforced concrete design, and basic member design/strength checks. For steel members, the program offers the ability to conduct cross section calculations, which form the basis of the member design strength. For reinforced concrete members, the program offers the ability to conduct cross section and reinforcement calculations.

To conduct member design and strength checks, the analysis of the structure of design interest must be linearized. Design parameters for the computations may include force/moment results output from static or dynamic analysis, as well as the cross section characteristics, material properties, and other member information that is input at the creation of the model.

When creating the analysis model, design criteria for each member may be specified using the design variable input window. If the user does not specify values for certain design variables, the program uses the default values.

After using the “Run Design” option, the member design results may be verified through various options. The program offers the ability to run graphic post-processing on the results, and the outputs can be seen in tabular or list format as well.

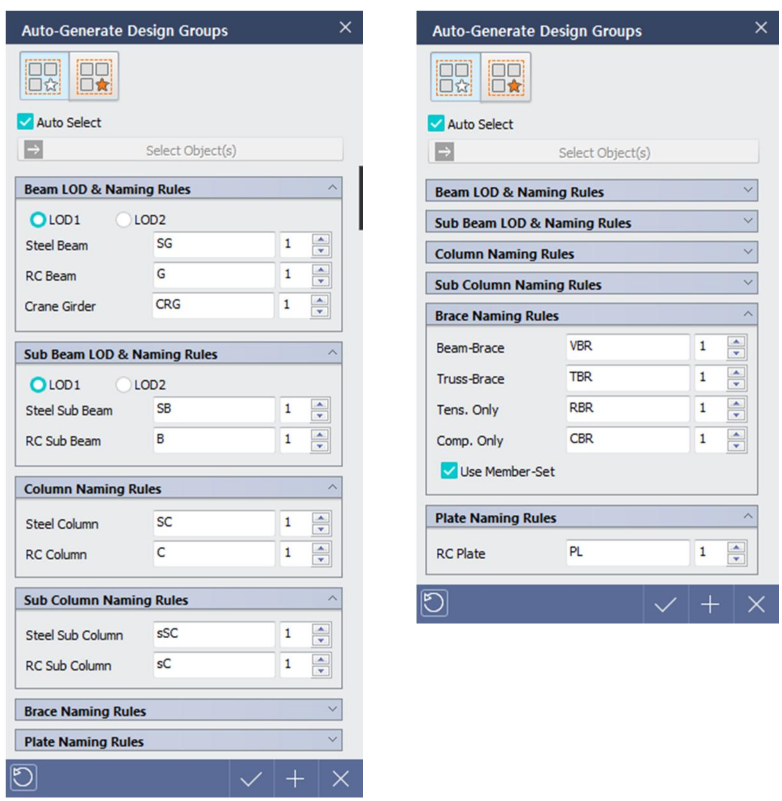
Section 2 Design Capabilities

2.1 Design Groups

Design groups categorize members with the purpose of obtaining common computation results. The following conditions are considered to auto-generate design groups, and the groups may be manually modified by the user.

▸ Members of a group are of the same type (beam, column, brace, wall, slab).
▸ Members of a group have the same characteristics (section, thickness, material).
▸If the reinforcement has been calculated, the reinforced concrete members of a group have the same reinforcement.

Figure 1.2.1 Dialog window for Auto-Generating Design Groups



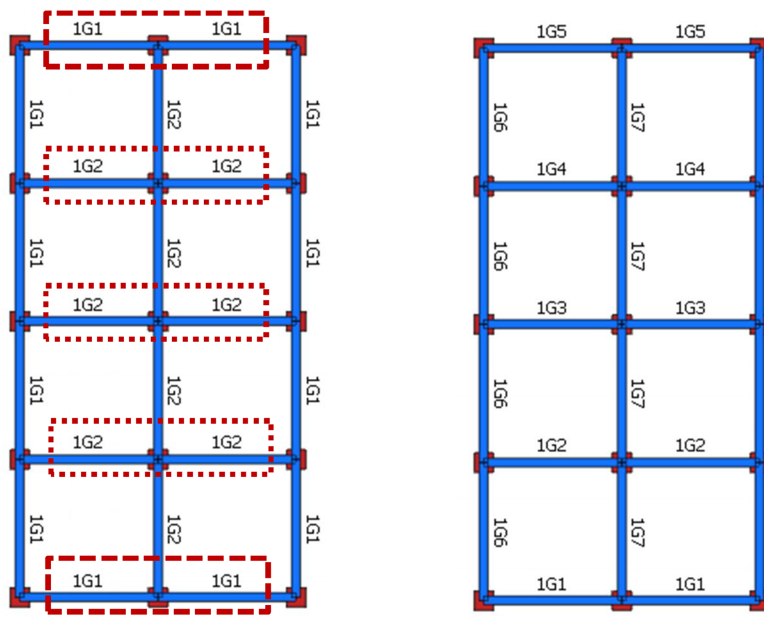
When auto-generating design groups, the naming conventions for the different member types are as follows.

Beams

▸ LOD1 (Level of Detail 1) : Exterior and interior beams are differentiated from one another and are grouped as such. Beams on different levels are categorized into different groups.

▸ LOD2 (Level Of Detail 2) : Beams that are not continuous based on the LOD1 results are categorized in separate groups and thus LOD2 allows the user to obtain a more subdivided level of grouping.

Figure 1.2.2 Beam Naming Convention



LOD1 : Only grouped into interior and exterior beams

LOD2 : Discontinuous beams are grouped separately

Sub Beams

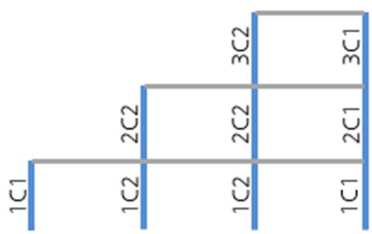
▸ LOD1 (Level of Detail 1) : Sub beams of the same level, measurements, and reinforcement are categorized into the same group.

▸ LOD2 (Level Of Detail 2) : Discontinuous beams based on the LOD1 results are categorized in separate groups and thus LOD2 allows the user to obtain a more subdivided level of grouping.

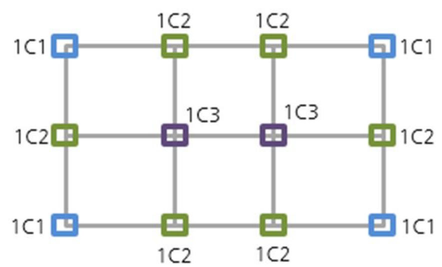
Columns

Groups categorize columns into corner columns, exterior columns, and interior columns.

Figure 1.2.3 Column Naming Convention



Elevation View



Plan View

Sub Columns

Continuous sub columns are grouped together, and all other exceptions are grouped separately.

Braces

Braces are separated into groups based on the analysis characteristics. Braces with the same analysis type, materials, and cross section are grouped together.

▸ Beam-Brace : Braces analyzed as beam elements
▸ Truss-Brace : Braces analyzed as truss elements
▸ Tens. Only : Braces analyzed as tension-only elements
▸ Comp. Only : Braces analyzed as compression-only elements

Plates

Plates are separated into one group per member.

2.2 Design Load Combinations

Load Combination Types

Load combinations for design are decided based on load types and numbers of combinations. Thus, it is important to confirm whether the defined load combination is appropriate for the design purpose, before proceeding with structural member design.

Load combinations for each country and design purpose are shown below.

Table 1.2.1 Load Combination Types depending on Country and Design Purpose

US	Strength	Load combination for strength verification
	Serviceability	Load combination for serviceability verification
	Special	Load combination for strength verification in special earthquake scenarios
	Vertical	Load combination for strength verification in vertical earthquake ground motions - Used for design of members with pre-stressing
	Fatigue	Load combination for fatigue verification
Korean	Strength	Load combination for strength verification
	Serviceability	Load combination for serviceability verification
	Special	Load combination for strength verification in special earthquake scenarios - Used for design of members that may cause a sudden break in earthquake load through instability or collapse.
	Vertical	Load combination for strength verification in vertical earthquake ground motions - Used for design of members with pre-stressing.
Eurocode	Fundamental	Load combination for strength verification (Persistent & Transient)
	Accidental	Load combination for strength verification (Accidental)
	Seismic	Load combination for strength verification (Seismic)
	Characteristic	Load combination for serviceability verification - Used for stress verification of reinforced concrete members
	Frequent	Load combination for serviceability verification - Used for cracking in reinforced concrete members.
	Quasi-Permanent	Load combination for serviceability verification -Used for cracking/stress verification in reinforced concrete members.
	Fatigue	Load combination for fatigue verification
British	Strength	Load combination for strength verification
	Serviceability	Load combination for serviceability verification

Definition and Application of Load Combinations

In this program, pre-defined load combination templates may be used. Alternatively, the user may manually specify load combinations.

The pre-defined, design-based load combination templates are already stored for easy use within the program. The user may modify the template files to apply the appropriate load combination. The file directory for the template is as follows:

Program Application Folder>Design>LoadCombination>Material Folder(RC, □)>Design Standard Folder(ACI 318-11, □)

Load combination template file name extensions are text files, and the main content format is shown below.

```
*VERSION
        1.0.0
*CODE
        AISC360-10(LRFD)          // in ASCE/SEI 7-10.
*DTYP
        STEEL
*LTYP
        D, L, LR, S, R, FP, WX:AL, WY:AL, W:AL, EQX:AL, EQY:AL, EQV:AL,
          EQ:AL, RSX:AL, RSY:AL, RSV:AL, RS:AL
         // A : Alternation (+/-), L : Loop, each load set
*CTYP
        STRN, SERV, SPECSEIS, VERTSEIS, FATG
         // Strength(General), Serviceability(Genera), Strength(Special Seismic),
Strength(Vertical Seismic)
*COMB-STRN
          LCB1, ADD
          1.4*D, 1.4*FP
```

▸ LTYP : This defines the load types that are incorporated into the load combinations. This may be verified from Home > Define > Set > Load Set > Static Load Set. The load types may be selected from the Load Type list that is pre-defined within the program. In particular, the specified keywords must be used in order to apply the desired load types, such as “D” for dead loads and “L” for live loads.

Figure 1.2.4 Static Load Set Dialog Window

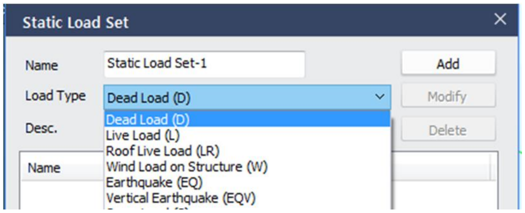


Table 1.2.2 Load Types and Keywords

Load Type	Keyword	Direction
Dead Load	D	
Live Load	L	
Roof Live Load	LR	
Wind Load on Structure	W	May be defined as WX, WY (Combination of both directions)
Earthquake	EQ	May be defined as EQX, EQY (Combination of both directions)
Veritical Earthquake	EQV	
Snow Load	S	
Rain Load	R	
Ice Load	IC	
Earth Pressure	EP	
Horizonal Earth Pressure	EH	
Vertical Earth Pressure	EV	
Ground Water Pressure	WP	
Fluid Pressure	FP	
Buoyancy	B	
Temperature	T	
Live Load Impact	IL	
Collision Load	CO	
User Defined Load	USER	

When defining the load type, two additional options are available to further specify load characteristics. Either or both options may be used.

Table 1.2.3 Additional options for load types

(Load Type):A	This option refers to alternative(+/-) loads. Load combinations for both directions of the applicable load types are created. (Ex.) When adding the option W:A to the load type and combination set LCB1, ADD, 1.0*D, 1.0*W, the following load combinations are created: LCB1_1 : 1.0*D + 1.0*W LCB1_2 : 1.0*D - 1.0*W
(Load Type):L	This option means to "loop each load set". Load combinations for each applicable load type are created. (Ex.) When adding the option L:L to the load set L1, L2, L3 and load definition LCB1, ADD, 1.0*D, 1.0*L, the following load combinations are created: LCB1_1 : 1.0*D + 1.0*L1 LCB1_2 : 1.0*D + 1.0*L2 LCB1_3 : 1.0*D + 1.0*L3

▸ CTYP : This refers to the design load combination. Depending on the design criteria, the load combination type changes, and the type is defined within the tabs in the "Define Load Combination" dialog window.

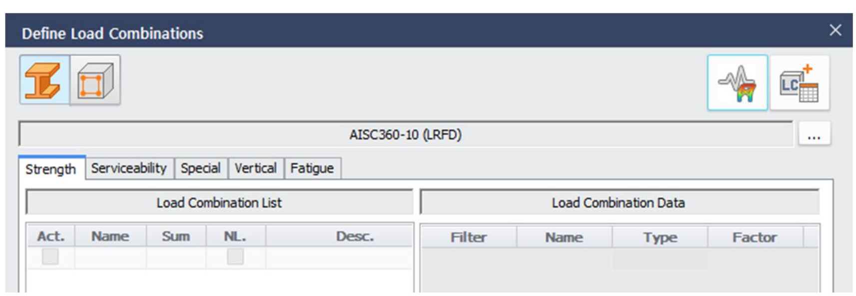


Figure 1.2.5 Define Load Combination Dialog Windows

Table 1.2.4 Load Combination Types and Keywords

Keyword	Load Combination Type
STRN	Strength
SERV	Serviceability
SPECSEIS	Strength > Special Seismic (ASCE, KBC)
VERTSEIS	Strength > Vertical Seismic (ASCE, KBC)
FUND	Strength > Fundamental
ACCD	Strength > Accidental
SEIS	Strength > Seismic
CHAR	Serviceability > Characteristic
FREQ	Serviceability > Frequent
QUAS	Serviceability > Quasi-permanent
FATG	Fatigue
SHRT	Short Term
LONG	Load Term
STRN1	Strength1
STRN2	Strength2
STRN3	Strength3
STRN4	Strength4
STRN5	Strength5
EXTR1	Extreme Event1
EXTR2	Extreme Event2
SERV1	Serviceability1
SERV2	Serviceability2
SERV3	Serviceability 3
SERV4	Serviceability 4
FATG1	Fatigue1
FATG2	Fatigue2

▸ COMB-■■■■ : This defines the load combinations that apply to different load types. The user may input the name of the load combination, the sum type, and the load combination number. The sum types for load combinations are shown below.

Table 1.2.5 Sum types of midas nGen load combinations

Keyword	Sum Type	Application
ADD	Linear Sum	$1.2D+1.6L$
ABS	Absolute Sum	$\lvert 1.2D \rvert + \lvert 1.6L \rvert$
SRSS	Square Root of Sum of Squares	$\sqrt{(1.2D)^2+(1.6L)^2}$
ENVELOP	Envelope	$\max[1.2D, 1,6L]$ $\min[1.2D, 1.6L]$

Nonlinear Load Combinations

When conducting nonlinear analysis, the user must select a load combination for iterative analysis. In nonlinear analysis, only sum type load combinations may be selected.

In iterative analysis, nonlinear elements such as tension-only and compression-only elements experience varying loads and stiffnesses depending on the strain and stress caused by different external loads. Thus, it is not feasible to obtain accurate results through the linear sum of load combinations.

Thus, as shown in Figure 1.2.6, in a structure with tension-only elements, the linear sum of analysis using two different loads is not equal to the result of using both loads at once.

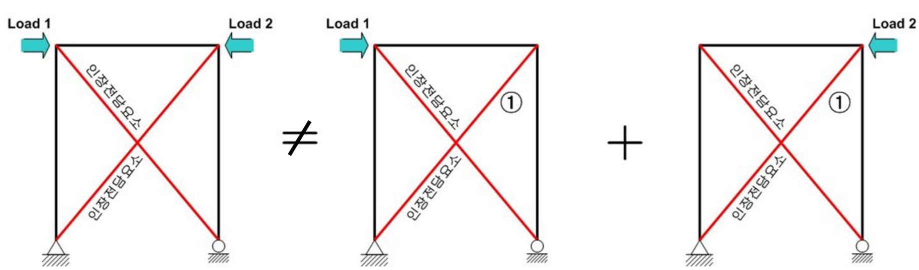


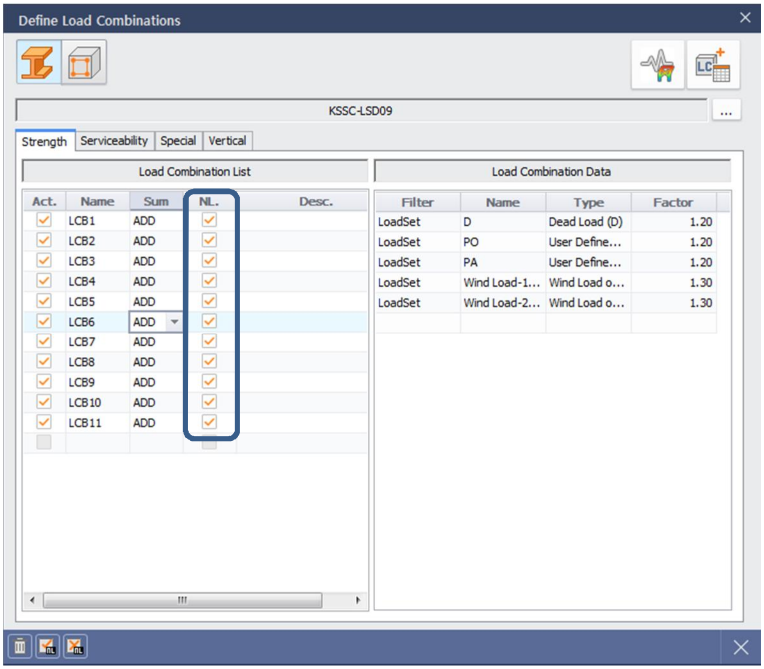
Figure 1.2.6 Comparison of results of a structure with tension-only elements

Axial force (A) in tension-only members due to Load 1: A ≠ 0

Axial force (B) in tension-only members due to Load 2:	B = 0
Linear sum of the axial forces for Load 1 and Load 2 :	A + B ≠ 0
Axial force using a nonlinear load combination (simultaneous combination of load 1 and load 2) :	0 (Zero axial force due to simultaneous loading)

In order to obtain the internal force of such a nonlinear element, unit load criteria must be applied to each load combination. In the program, the load combination information may be interpreted as unit loads when load combinations are defined (the NL Check option shown below in Figure 1.2.7).

Figure 1.2.7 Load Combination Editing Dialog Window



Set loop-option

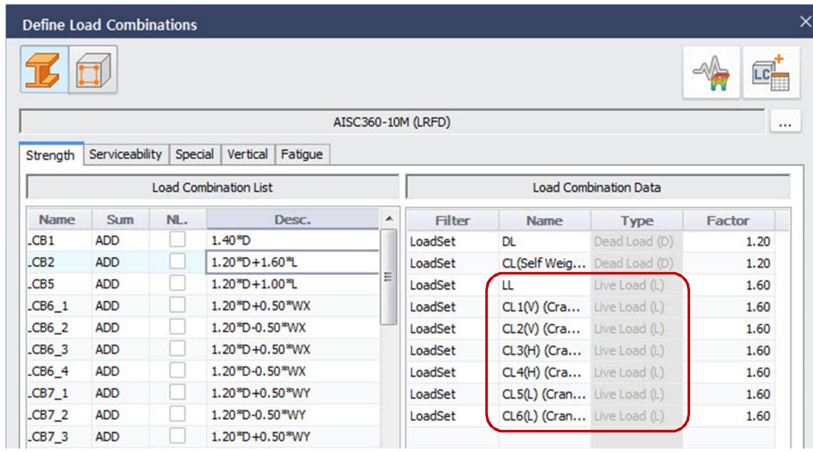
The loop-option is used to create specific load combinations that are created from load sets including loads of the same load type that should not be repeated in the same load combination.

If the Auto Generate Load Combination function is used, then load combinations are created using the sums of loads belonging to the same load types.

When creating load combinations using the results of moving crane analysis, it is important to be aware of certain load results that should not be repeated within the same load combination. In such a case, the loop-option may be set to define the required load combinations.

Consider the example of reaction forces calculated through moving crane analysis. Without using the loop condition and instead using the Auto Generate Load Combination function, all live loads are all included simultaneously within the same load combination.

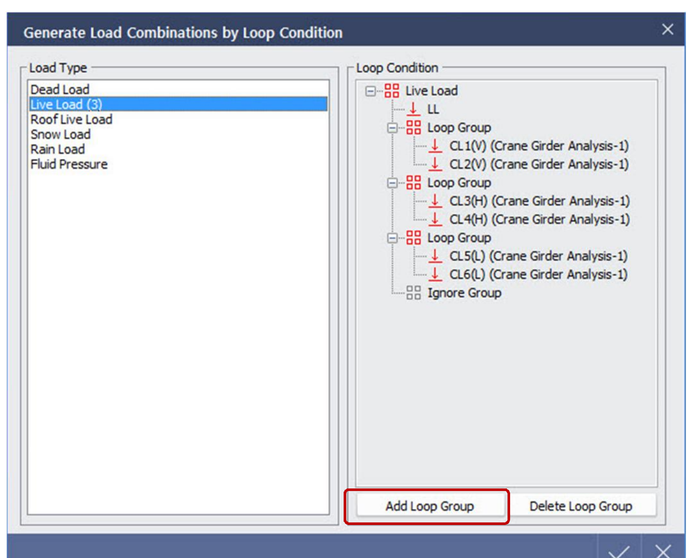
Figure 1.2.8 Load combinations that are defined as a result of using the Auto Generate Load Combination function



Repeating vertical and horizontal loads and axial loads in the same load combination may lead to an over-design of the structure. Each load combination should include one vertical, one horizontal, and one axial load to create an appropriate design. In this case, three Loop Groups are defined and the different loads are categorized into the appropriate Loop Groups.

Define Load Combinations > Generate Load Combinations by Template > Set loop - option

Figure 1.2.9 Load combinations that are defined as a result of using the Loop Option



Using the Loop Condition function, six load combinations for the load combination equation LCB2 (1.2*D+1.6*L) have been created. The automated load combinations using the loop option are shown below.

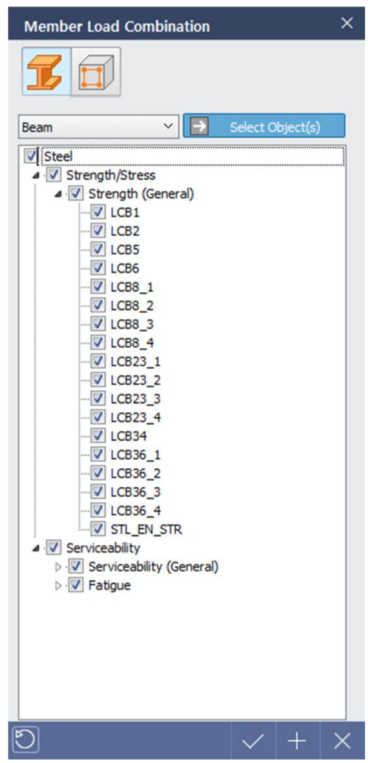
Table 1.2.6 Load combinations automatically created with loop conditions

Conditions	Dead Loads: DL, CL Live Loads: CL1(V), CL2(V), CL3(H), CL4(H), CL5(L), CL6(L) Load Combination Equation: LCB2 = 1.2*D + 1.6*L
Load Combinations without using Loop Conditions	1.2D+1.6*(LL+CL1(V)+CL2(V)+CL3(H)+CL4(H)+CL5(L)+CL6(L))
Load Combinations using Loop Conditions	1.2*D+1.6*(LL+CL1(V)+CL3(H)+CL5(L)) 1.2*D+1.6*(LL+CL2(V)+CL3(H)+CL5(L)) 1.2*D+1.6*(LL+CL1(V)+CL4(H)+CL5(L)) 1.2*D+1.6*(LL+CL2(V)+CL4(H)+CL5(L)) 1.2*D+1.6*(LL+CL1(V)+CL4(H)+CL6(L)) 1.2*D+1.6*(LL+CL2(V)+CL4(H)+CL6(L))

Member Load Combinations

Member-specific load combinations may be selected from the load combinations that have already been created. The user may manually specify load combinations to be applied to a specific member and this load combination may differ from those being applied to other structural members, usually for a specific design purpose.

Figure 1.2.10 Member Load Combination Dialog Window



The load combinations with Check-On status are used in strength verification or reinforcement definition. The load combinations with Check-Off status are not used in strength verification or reinforcement definition.

2.3
Member Parameters

Effective Length Factors

If the axial compression force of a compressive member is small, the column length may be slightly reduced. However, if the compressive force increases and reaches a certain threshold, then the element may suddenly buckle.

The buckling behavior of a column element depends on the cross-section properties, material properties, column length, and boundary conditions. Columns should be designed to avoid buckling behavior.

The effective length factor of a column element should be calculated using an alignment chart or by using a representative effective length factor based on the boundary conditions at the two ends.

Of course, using the alignment chart to determine the effective length factor for all columns of a model is quite burdensome. The program allows for internal, automated calculations of the effective length factors for column elements. The effective length factor is decided based on whether or not side sway is inhibited—this decides whether the structure is composed of a braced frame or unbraced frame.

Braced frames do not permit lateral movement as they have a structural member that resists such movement (e.g. shear walls, braces). Thus, braced frames prevent lateral movement through bracing elements other than the frame.

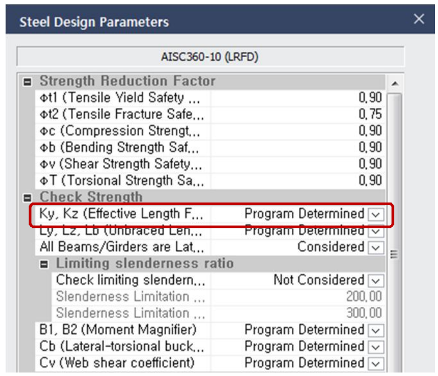
Oftentimes in actual structures, however, braces may exist in only one direction or installed in only one portion of the structure. Thus, it is not often easy to differentiate braced frames from unbraced frames.

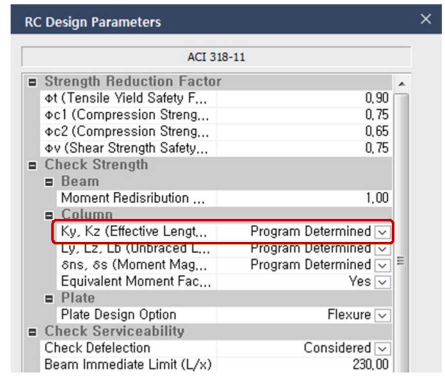
In the automated procedure for calculating the effective length factor, many assumptions are required. Thus, it is important to ensure that the automatically calculated values are realistic.

The automated calculations for the effective length factor of all elements within a model are set up in the Design Parameters dialog window.

Home>Design Settings>General>Design Code

Figure 1.2.11 Design Parameters Dialog Window

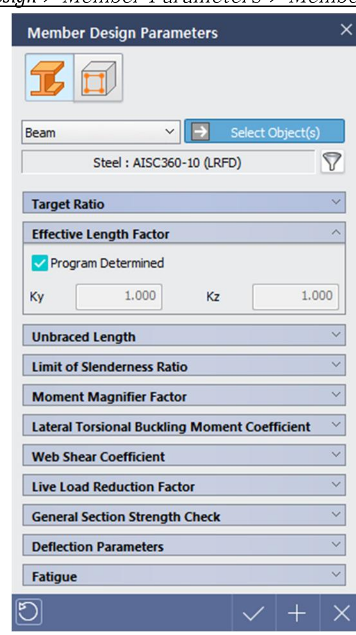




For each member, the user may allow the program to determine the effective length factor or may specify a value for the program to use. This may be done in the Member Design Parameters dialog window.

Analysis & Design > Member Parameters > Member Parameters

Figure 1.2.12 Member Design Parameters Dialog Window



The automatically calculated length factor may be verified in the Design Report and the Design Results (Graphics window).

Result>Design Result>Design Report

Figure 1.2.13 Effective Length Factor within the Design Report

Steel Member Design Summary Report based on AISC360-10 (LRFD) [lbf, in]

A. Member Information

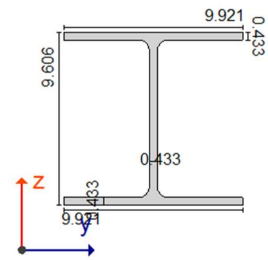
a. Member Name
- 1SC1 - Column-1 [51]

b. Material
- SS400 (KS09) [1]
- F_y = 34.08 ksi

c. Section
- SSD_1SC1_H 244x252x11/11 [20]

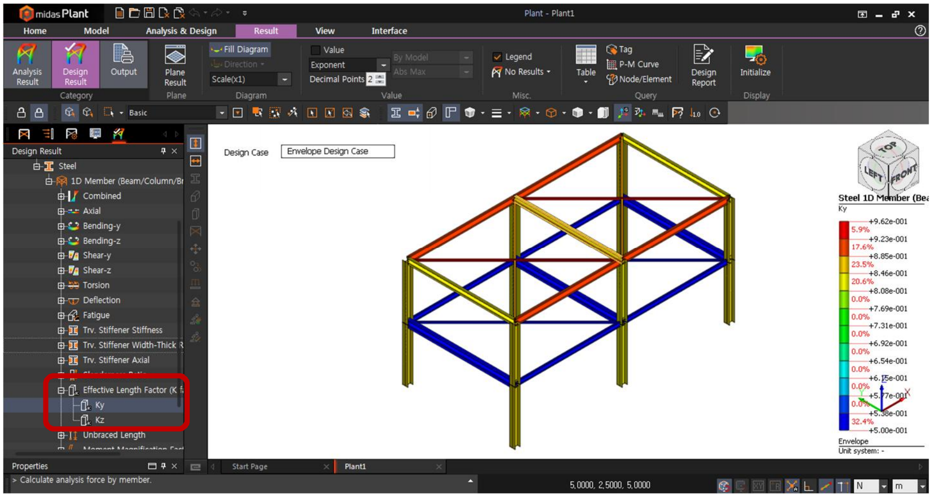
d. Member Length
- L = 16.404 ft
- L_y = 8.202 ft, L_z = 8.202 ft
- L_b = 0.000 ft (at 0.00L, 0.00 ft)

e. Effective Length Factor
- K_y = 0.847, K_z = 0.805



Result>Design Result>Effective Length Factor (K factor)>Ky, Kz

Figure 1.2.14 Effective Length Factor within the Design Results (Graphic Interface)



▸ Algorithm for Determining the Effective Length Factor

K represents the effective length factor, and given the relationship X=π/K, the equations of equilibrium for braced and unbraced frames are as follows.

Braced Frame :

$$F(X) = \frac{G_A G_B}{4} X^2 + \left[\frac{G_A + G_B}{2}\right]\left[1 - \frac{X}{\tan X}\right] + \frac{2}{X}\tan\left[\frac{X}{2}\right] - 1 = 0$$

(1.2.1)

Unbraced Frame : $$F(X) = \frac{G_A G_B X^2 - 36}{6(G_A + G_B)} - \frac{X}{\tan X} = 0$$
(1.2.2)

The following assumptions lead to the above equations of equilibrium.

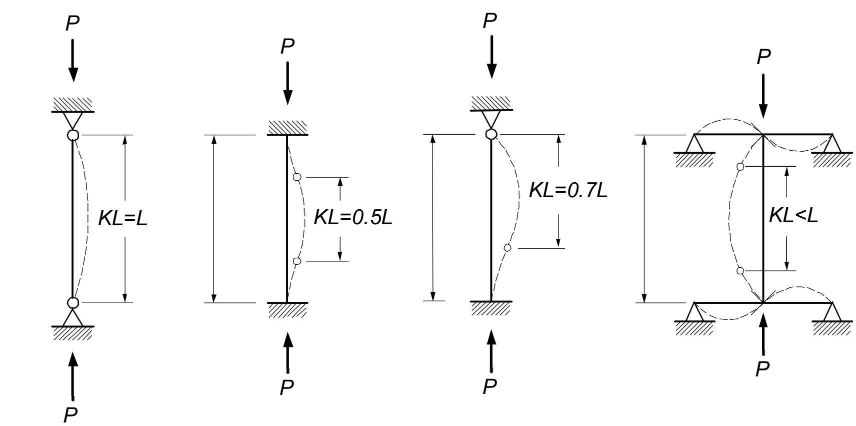
1. All of the motion remains within the elastic region.
2. The members are prismatic.
3. All columns simultaneously experience buckling loads.
4. Structures are symmetrically braced.
5. The restraining moment due to a girder at a node is distributed to the columns based on the stiffness of each column.
6. Girders are elastically restrained at each end with the columns, and when buckling occurs, the rotational displacement of each end of the girder has the equal magnitude and opposite direction.
7. Girders do not support axial loads.

The solution to this nonlinear equation is found through the Newton-Raphson method. The iterative relationship is shown below.

$$X_2 = X_1 - \frac{F(X)}{F'(X)} \tag{1.2.3}$$

The solution to the equation involves tanX and tan(X/2) within $F(X)$ and $F'(X)$, which may become zero or infinite. This program takes this into consideration and ensures that a stable solution is always reached.

Figure 1.2.15 The effective length (KL) of a column with two restrained ends



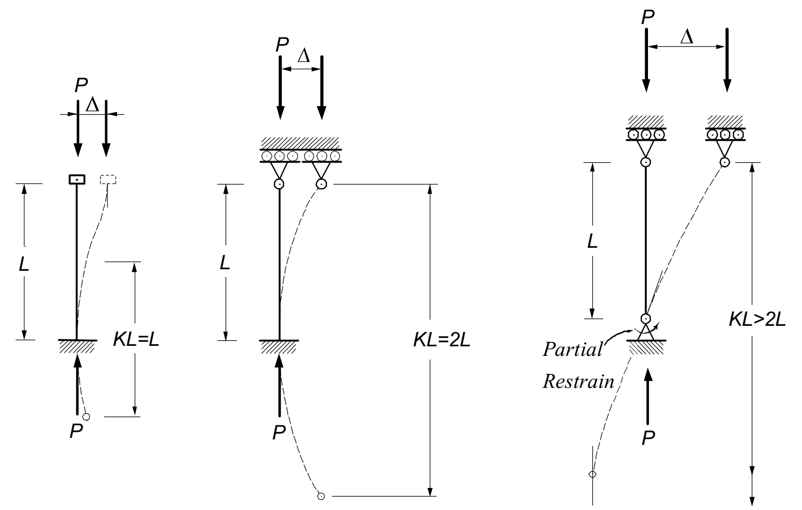
(a) End Rotations Restrained

(b) End Rotations Fully Restrained

(c) One End Restrained,

(d) Partially Restrained

Figure 1.2.16 Effective length (KL) of a column with one free end

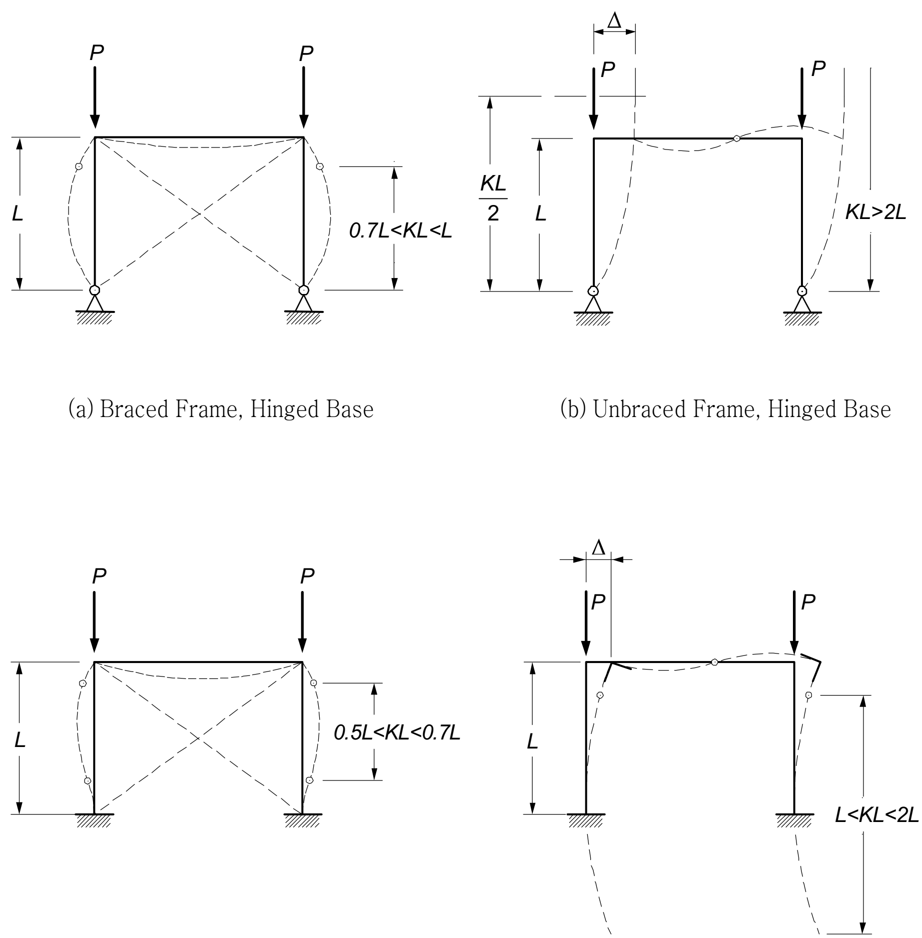


(a) End Rotations

(b) One End Rotation Fully Restrained, Other Unrestrained

(c) One End Rotation Partially Restrained, Other End

Figure 1.2.17 Effective length of braced and unbraced frames



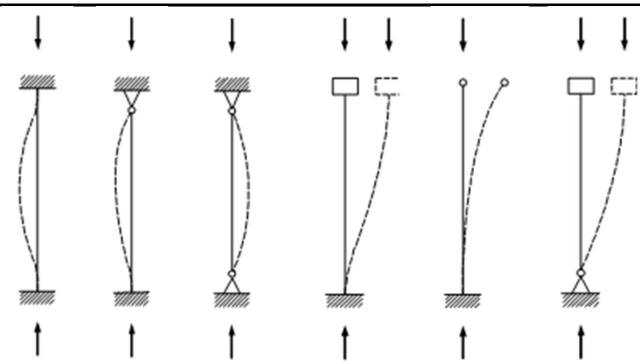
(a) Braced Frame, Hinged Base

(b) Unbraced Frame, Hinged Base

(c) Braced Frame, Fixed Base

(d) Unbraced Frame, Fixed

Table 1.2.7 Effective length factor (K) of columns with different end conditions

	a	b	c	d	e	f
Buckled shape of the column shown by dashed line						
Theoretical value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design values when ideal conditions are approximated	0.65	0.8	1.0	1.2	2.1	2.0
End conditions code		Rotation fixed, Translation fixed				
		Rotation free, Translation fixed				
		Rotation fixed, Translation free				
		Rotation free, Translation free				

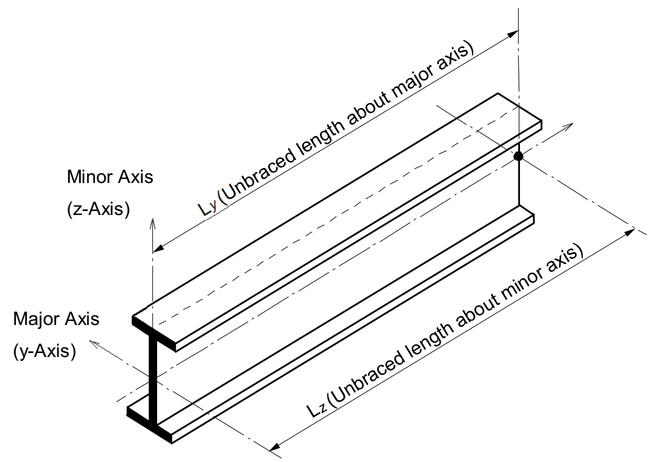
Unbraced Length

The unbraced length of a member is determined with respect to both the y- and z-axis directions (in the element coordinate system).

When a member is subject to axial force or bending moment, the length that experiences bending strain along the element major axis (y-axis) and minor axis (z-axis) in the element coordinate system is called the unbraced length (see Figure 1.2.18).

The unbraced length of a member is used along with the effective length factor. The unbraced length also is incorporated in calculations for the slenderness ratio, which is required in computing the design axial compressive strength or allowable compressive force.

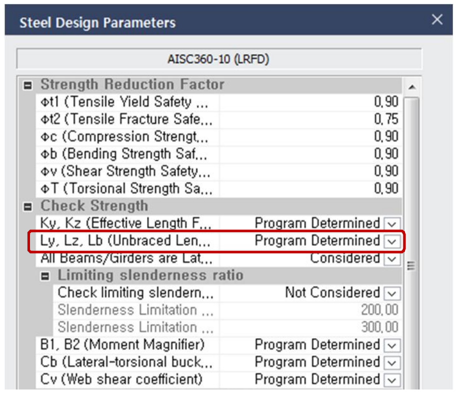
Figure 1.2.18 Unbraced length of a member

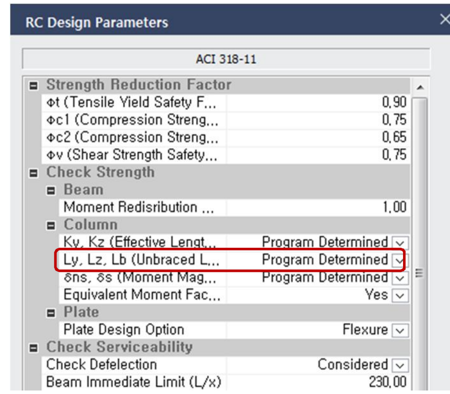


The automatic calculations for the unbraced lengths of all members within a model are set up in the *Design Parameters* section of each Design Code.

Home>Design Settings>General>Design Code

Figure 1.2.19 Design Parameters Dialog Window

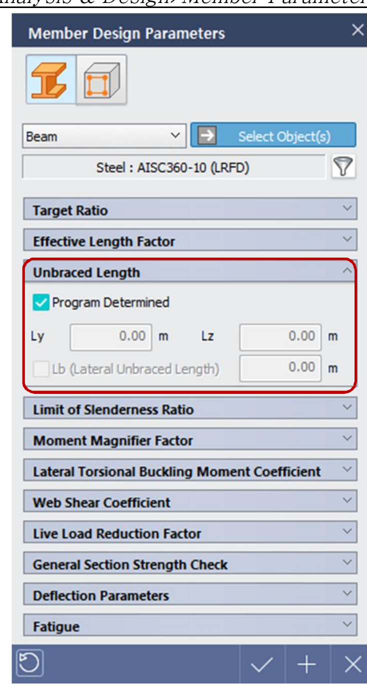




The unbraced length may be left to be determined by the program or the user may specify a value for the program to use. This can be defined within the Member Design Parameters dialog window for each member.

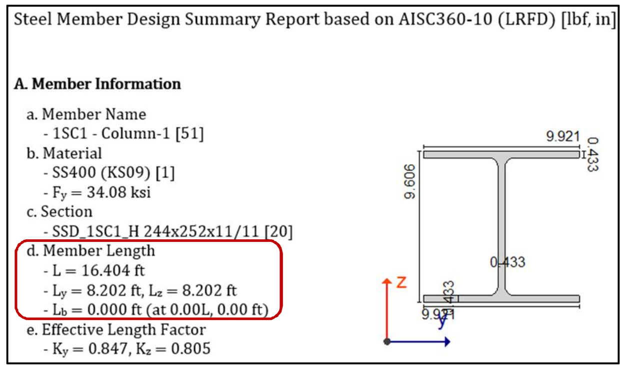
Analysis & Design>Analysis & Design>Member Parameters>Member Parameters

그림 1.2.20 Member Design Parameters Dialog Window



The automatically computed lengths may be seen in the Design Report and the Design Results (graphic interface).

Result>Design Result>Design Report



Steel Member Design Summary Report based on AISC360-10 (LRFD) [lbf, in]

A. Member Information

a. Member Name
- 1SC1 - Column-1 [51]

b. Material
- SS400 (KS09) [1]
- F_y = 34.08 ksi

c. Section
- SSD_1SC1_H 244x252x11/11 [20]

d. Member Length
- L = 16.404 ft
- L_y = 8.202 ft, L_z = 8.202 ft
- L_b = 0.000 ft (at 0.00L, 0.00 ft)

e. Effective Length Factor
- K_y = 0.847, K_z = 0.805

Figure 1.2.21 Unbraced Length shown in the Design Report

Result>Design Result>Unbraced Length>Ly, Lz, Lb

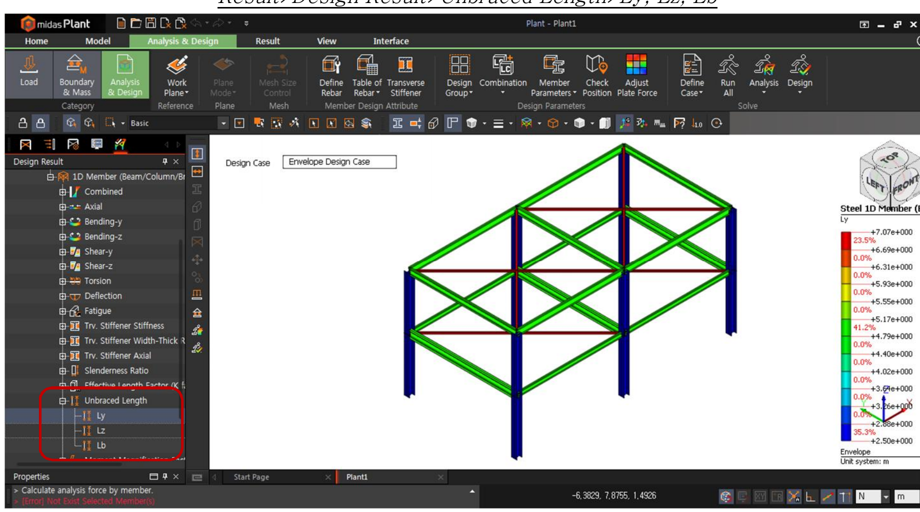
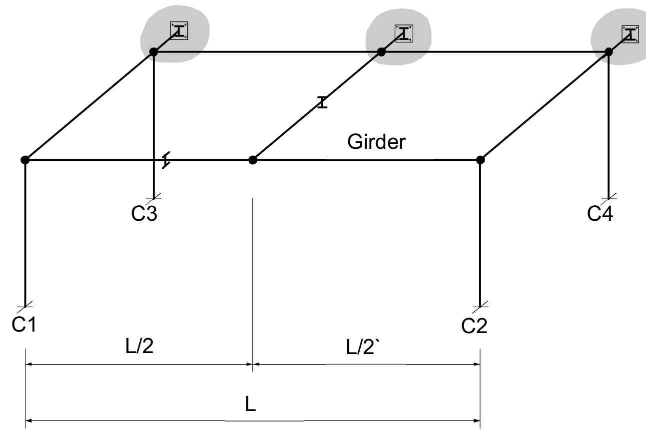


Figure 1.2.22 Unbraced Length shown in the Design Results (graphic interface)

The following example illustrates the calculation of an unbraced length for a typical case.

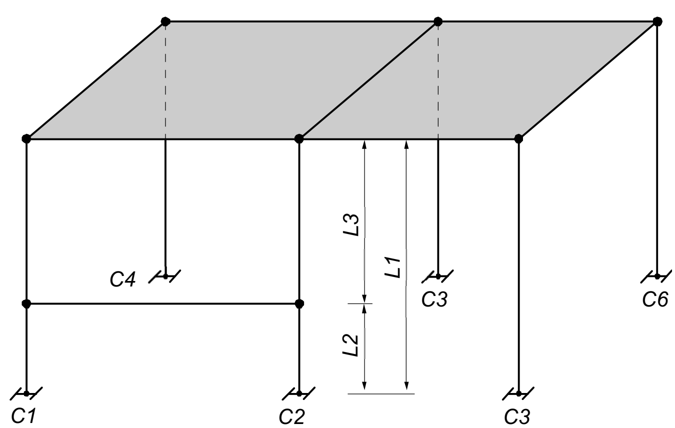
Figure 1.2.23 Unbraced length for major and minor axis of a member, CASE 1



Unbraced length of a girder : Ly = L, Lz = L/2

< CASE 1 >

Figure 1.2.24 Unbraced length for major and minor axis of a member, CASE 2



Unbraced length of a column Ⓐ : $L_y = L2$, $L_z = L1$

Unbraced length of a column Ⓑ : $L_y = L3$, $L_z = L1$

Unbraced length of a column Ⓒ : $L_y = L_z = L1$

< CASE 2 >

Figure 1.2.25 The relationship between the analytical model elements and the unbraced lengths

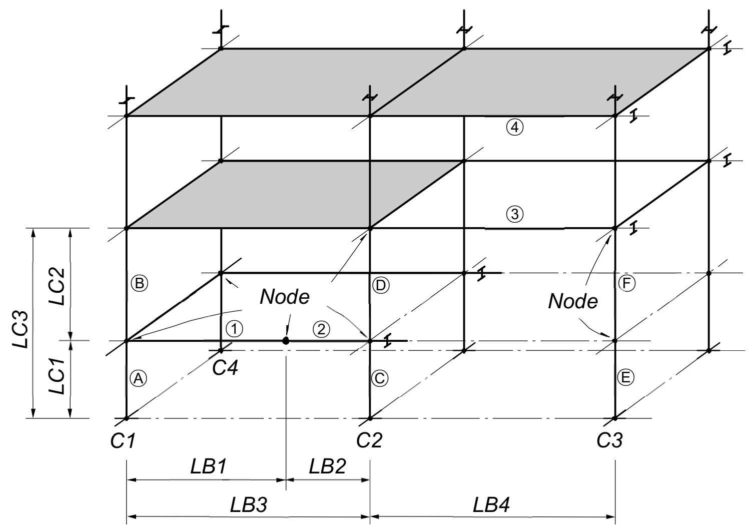


Table 1.2.8 The relationship between the analytical model elements and the unbraced lengths

Element	Unbraced length (Ly) about the element major axis (y-axis)	Unbraced length (Lz) about the element minor axis (z-axis)	Comments
Column Ⓐ	LC1	LC1	-
Column Ⓑ	LC2	LC2	-
Column Ⓒ	LC1	LC3	-
Column	LC2	LC3	-
Column Ⓔ	LC3	LC3	-
Column Ⓕ	LC3	LC3	-
Beam	LB3	LB3	-
Beam ②	LB3	LB3	-
Beam ③④	LB4	LB4	-
Beam	LB4	0	Restrained by slab

Laterally Unbraced Length

If there is a vertical load exerted in the direction parallel to the web of a beam or girder, a vertical deflection occurs on the in-plane side of bending where the moment occurs. As the load increases and eventually passes a certain threshold, the compressive flange of the beam or girder experiences a horizontal displacement outside of the bending plane. As a result, the element may experience rotation and torsion. This phenomenon is called lateral torsional buckling.

When this phenomenon occurs, the member can no longer resist the force being exerted on it and may experience sudden failure. Thus, it is important to design beams and girders to prevent lateral torsional buckling. In typical steel member design criteria, the lateral torsional buckling failure mode is considered and it is required that the designer calculate the allowable bending stress or design bending strength.

The laterally unbraced length is required to calculate the allowable bending stress (design bending stress), which incorporates lateral torsional buckling considerations. This is the distance along the length of the member in which, under lateral loads, the lateral displacement of the compressive flange is restricted.

Figure 1.2.26 Lateral Torsional Buckling

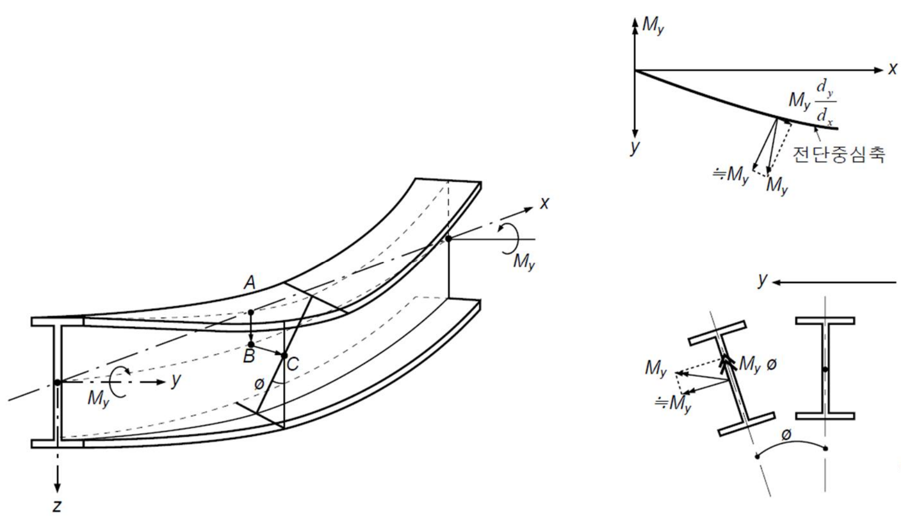
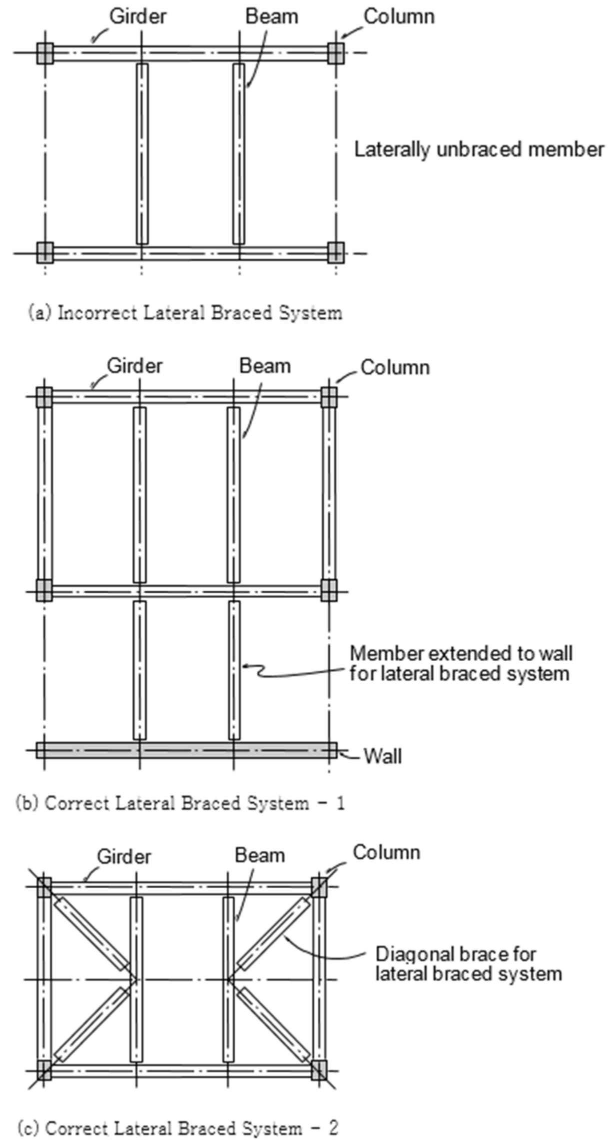


Figure 1.2.27 Example of Laterally Braced Systems



The input for the lateral torsional buckling length and the result of the automated calculation may be verified in the same procedure as the unbraced length.

Live Load Reduction Factor

The live load that is applied to structures are not truly being exerted across the entire floor area. Thus, to achieve reasonable and economical designs, live load reduction factors, shown in Equation 1.2.4, should be used.

$$F = F_D + (LLRF)F_L + F_S \tag{1.2.4}$$

Here,

F	: Axial force, moment or shear force incorporating the live load reduction factor
FD	: Axial force, moment, or shear force due to dead loads or other vertical loads
FL	: Axial force, moment, or shear force due to live loads
FS	: Axial force, moment, or shear force due to lateral loads (wind loads, earthquake loads)
LLRF	: Live load reduction factor

FD, FL, and FS are factored loads (axial force, moment, or shear force).

The live load reduction factor may be calculated as a function of either the tributary area or the number of stories. The calculation procedures for different design codes are shown below.

▸ ASCE7-05 (Calculated based on the Effective Tributary Area)

$$L = L_0(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}) \tag{1.2.5}$$

Table 1.2.9 Live Load Element Factor, K_{LL}

Element	K_{LL}*
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Cornor coumns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified Including: Edge beams with cantilever slabs Cantilever beams One-way slabs	1

Tow-way slabs
Member without provisions for continuous
Shear transfer normal to their span

* In lieu of the preceding values, KLL is permitted to be calculated.

► BS EN 1991-1-1;2002 (Calculated based on the Tributary Area)

$$\alpha_A = \frac{5}{7}\psi_0 + \frac{A_0}{A} \leq 1.0 \tag{1.2.6}$$

With the restriction for categories C and D : $\alpha_A \geq 0.6$
Here,
ψ_0 is the gactor according to EN 1990 Annex A1 Table A1.1
A0 = 10m2
A is the loaded area

► BS EN 1991-1-1;2002 (Calculated based on the number of stories)

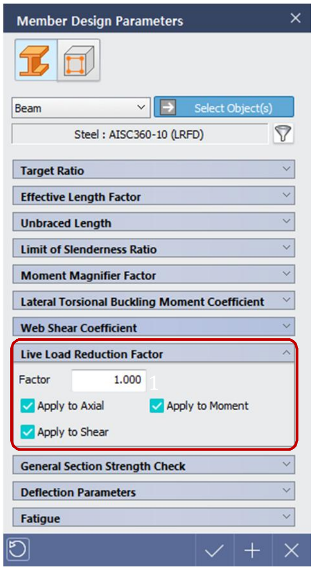
$$\alpha_n = \frac{2+(n-2)\psi_0}{n} \tag{1.2.7}$$

Here,
n : is the nunber of storeys (>2) above the loaded structural elements from the same category.
ψ_0 : is in accordance with EN 1990, Annex A1, Table A1.1

In this program, the live load reduction factor is user-specified, and may be input for each member in the Member Design Parameters dialog window.

Analysis & Design>Analysis & Design>Member Parameters>Member Parameters

Figure 1.2.28 Member Design Parameters Dialog Window



2.4
Check Points

Depending on the check points within a member, the cross-section stiffness and analysis results may change. Thus, the check point locations greatly affect the design results. To obtain more accurate results, check point capabilities have been installed within this program. In midas Gen, five check points were used for 1-dimensional members. In this program, the user may specify check points for the entire structure or for select structural members. Furthermore, as shown in Figure 1.2.29, member performance may be verified to the left or right of the check point locations. This will allow the user to check for any inconsistencies or unrealistic results that may occur due to concentrated loads or member connections.

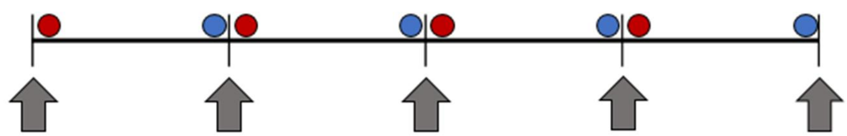


Figure 1.2.29 Check points of midas nGen

Default Check Points

As shown below, check points may be specified based on the member characteristics.

► Steel Beam/Column/Brace: The number of check points may be specified for each type of member, and element performance is verified at the ends of each of the member subdivisions. If the number of check points is five, then the member is divided into four sections.

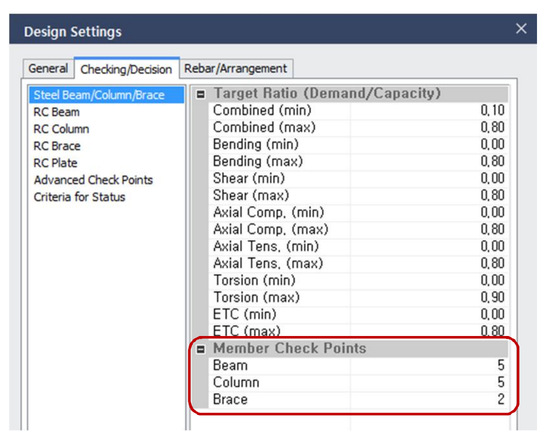


Figure 1.2.30 Check point settings dialog window for steel beam/column/braces

► RC Bow/Column : The locations of the check points for RC members are related to the reinforcement. The locations may be specified with reinforcement considerations. The check point locations and number of check points may be specified individually with respect to each endpoint and midpoint.

Figure 1.2.31 RC Beam/Column Check Points

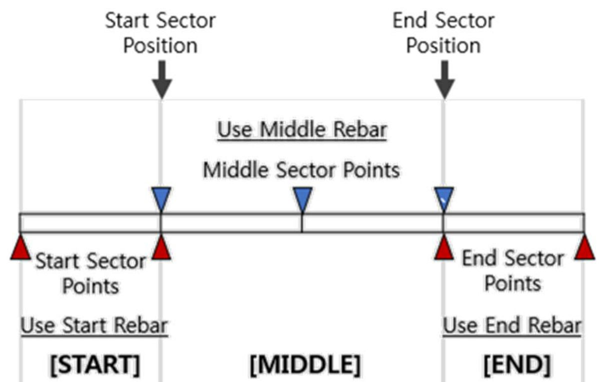
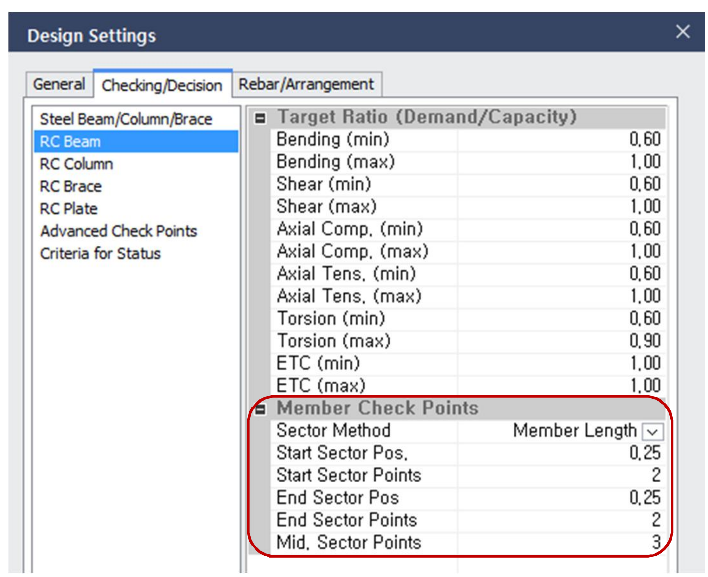
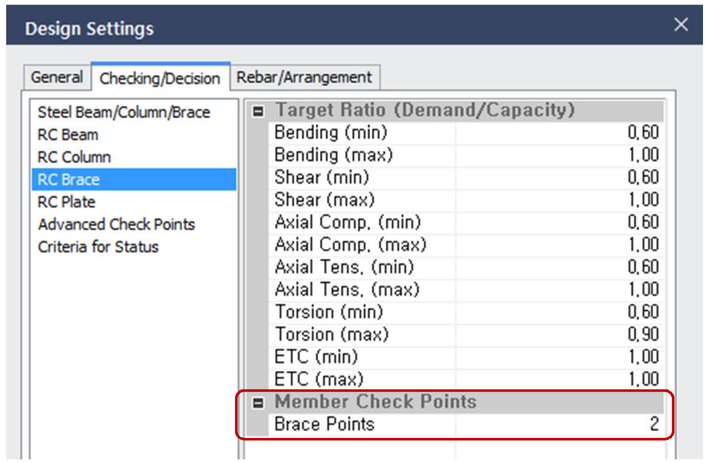


Figure 1.2.32 Check Point settings dialog window for RC Beam/Column



▸ RC Braces : Most braces resist axial forces, and the number of check points are defined regardless of the endpoints and midpoints.

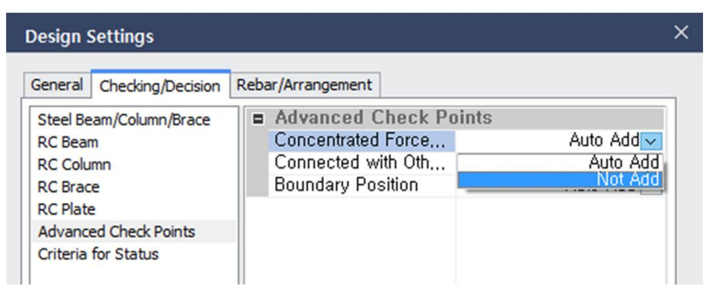
Figure 1.2.33 Check point setting window for RC braces



Advanced Check Points

To achieve a more accurate design, locations that may experience a sudden change in force require more thorough inspection and the user may specify additional criteria. The following figures show scenarios in which sudden changes in force may occur.

Figure 1.2.34 Advanced Check Points Dialog Window



▸ Location of concentrated loads

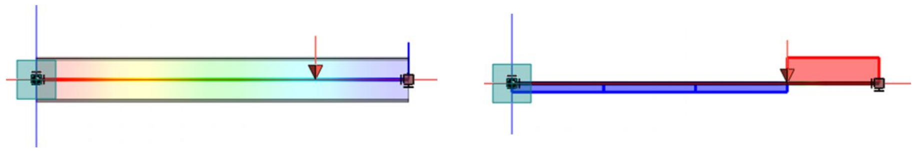


Figure 1.2.35 Location of concentrated loads

▸ Point of connection with other members

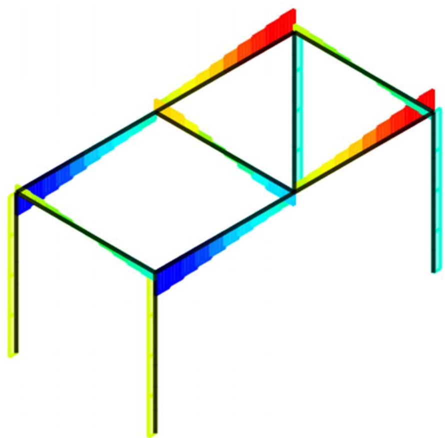


Figure 1.2.36 Point of connection with other members

▸ Location at which boundary conditions are applied

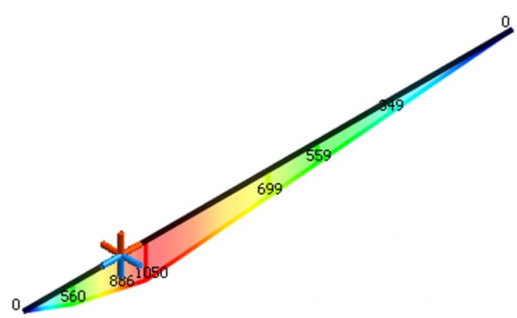


Figure 1.2.37 Boundary conditions

Member Check Positions

Independent of the check point definitions in Design Setting, check point locations may be defined in selected members. The following figure shows how the design length and number of check points may be specified.

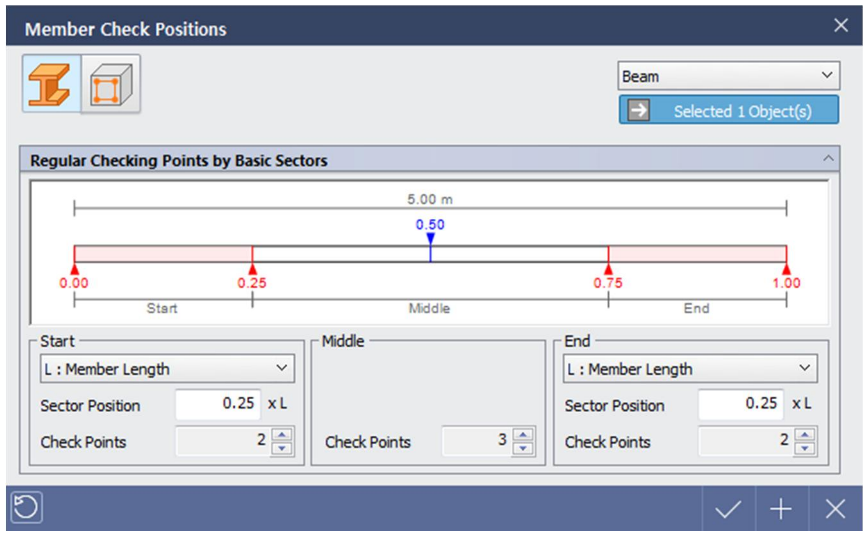


Figure 1.2.38 Member Check Positions set-up dialog window

2.5 Target Ratio and Design Checking/Decision

Target Ratio

In this program, target ratios are defined for each member type, and the user may also incorporate marginal values when designing the cross section or checking the design strength. Typically, design results are deemed to be adequate if the inequality shown in Equation 1.2.8 is satisfied. If the target ratio is incorporated into the design, then the inequality shown in Equation 1.2.9 is used instead.

$$\text{Demand} \leq \text{Capacity} \tag{1.2.8}$$

$$\text{Demand} \leq \text{Target Ratio X Capacity} \tag{1.2.9}$$

Here,

Demand : Demand strength / stress / deflection

Capacity : Strength / force / deflection that the section is capable of handling

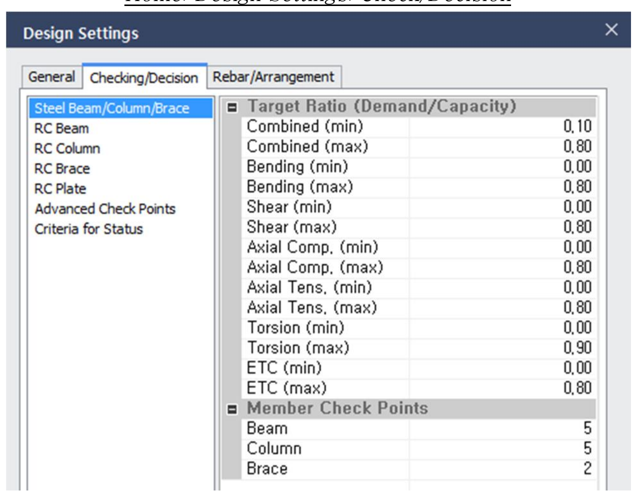
Target ratio is used as a criterion for deciding whether the resulting design for steel or RC member sections is satisfactory. For designing reinforcement in RC members, the target ratio also serves to help decide whether the reinforcement design is satisfactory.

The target ratio may be selected differently depending on the member material properties, shape, or other important values. Thus, it is up to the user to decide on the design criteria and marginal values.

The target ratio for either the entire structure or for select members in the Design Settings dialog window.

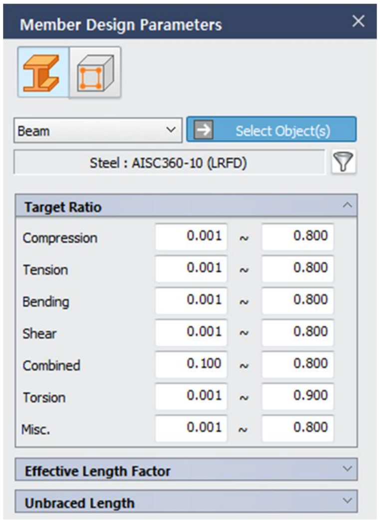
<u>*Home>Design Settings>Check/Decision*</u>

Figure 1.2.39 Target Ratio set up dialog window



The target ratios for various values may be input in the Member Design Parameters dialog window.

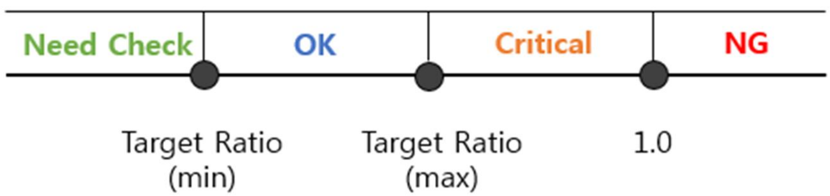
Figure 1.2.40 Target Ratio set up in the Member Design Parameters dialog window


Member Design Parameters
Beam
Select Object(s)
Steel : AISC360-10 (LRFD)
Target Ratio
Compression 0.001 ~ 0.800
Tension 0.001 ~ 0.800
Bending 0.001 ~ 0.800
Shear 0.001 ~ 0.800
Combined 0.100 ~ 0.800
Torsion 0.001 ~ 0.900
Misc. 0.001 ~ 0.800
Effective Length Factor
Unbraced Length

Design Results and Decisions

The program allows the user to check the design results and provides the design checking process in a more detailed manner, as shown in Figure 1.2.4.

Figure 1.2.41 Design Results and Checking



▸ Need Check : This is the case in which the design result is smaller than the minimum target ratio. This often represents overdesign situations, and the user may attempt a more economic design by modifying the materials, cross sections, and design factors.

▸ OK : This is the case in which the minimum target ratio < design result ≤ maximum target ratio.

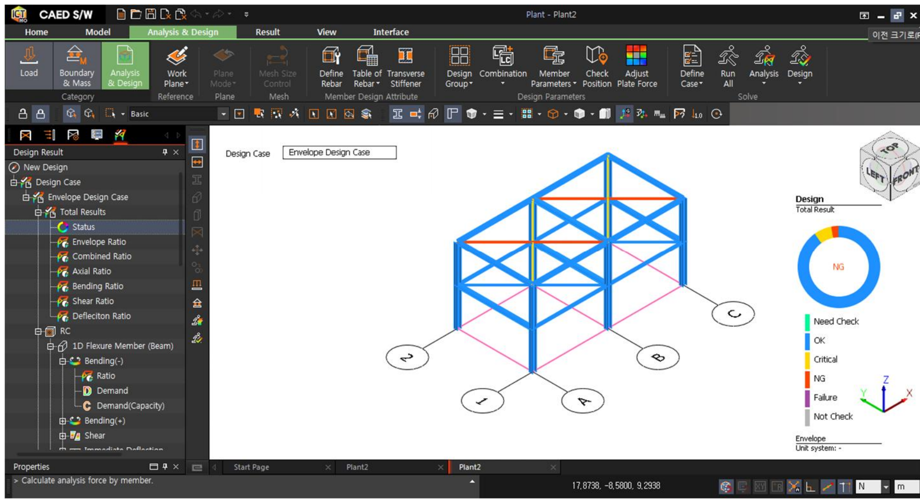
▸ Critical : This is the case in which the maximum target ratio < design result ≤ (1.0). The actual behavior should meet performance criteria, but represents a situation in which some members must be checked as the overall design exceeds the maximum target ratio set by the user.

▸ NG : Design result > (1.0).

▸ Failure : This only applies to RC members and is available so that users may detect the possibility of brittle failure. In such a case, the member does not perform adequately with only augmenting the steel reinforcement, so it is necessary that the user increase the cross section size or material strength.

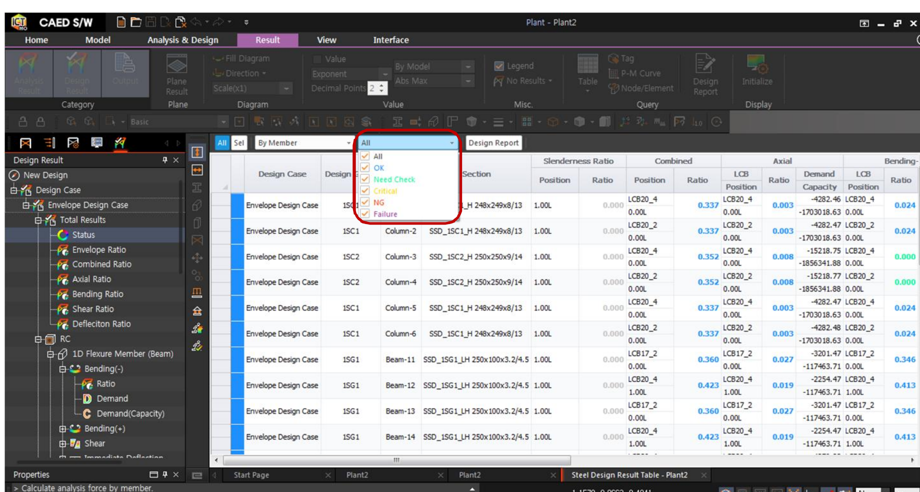
As shown in Figure 1.2.42, through Design Result > Total Result > Status, the member design result may be displayed in a visual manner. The design total result ratio is also provided in the interface, so the user may make a more intuitive decision about the total structural result.

Figure 1.2.42 Structure design result appearance in the software



Additionally, some design results may be filtered in the table of results, or a specific member's performance may be verified within the software. Thus, these options allow for quick access to detailed design results.

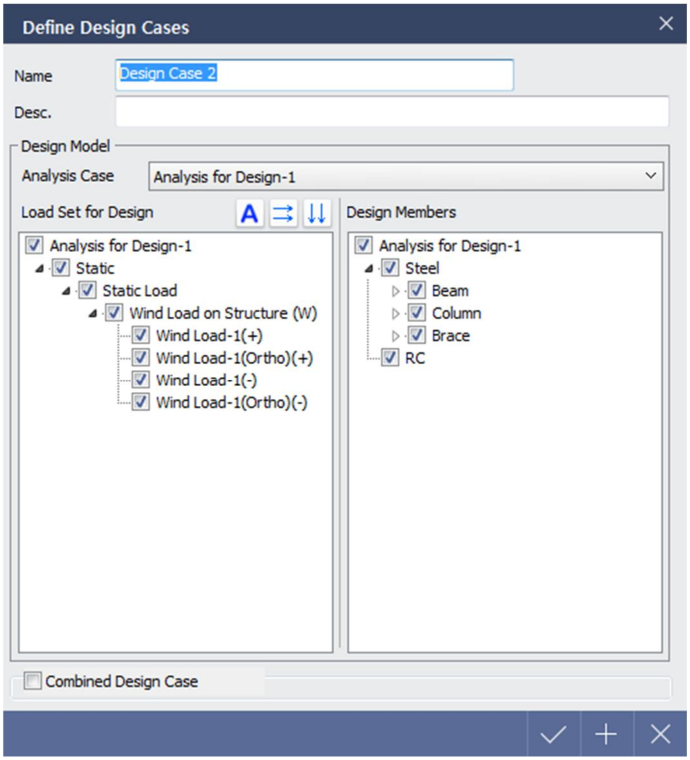
Figure 1.2.43 Structure design result filtering option



2.6
Design Cases

In this program, different analysis cases may be set up with a single model. Each analysis case may be defined to conduct analysis with different target members, loads, and boundary conditions. Thus, even with a single model, a variety of analysis results may be created. Moreover, if the design case is defined with the options included in these analysis cases, then various design results may also be created.

Figure 1.2.44 Design Cases set up dialog window



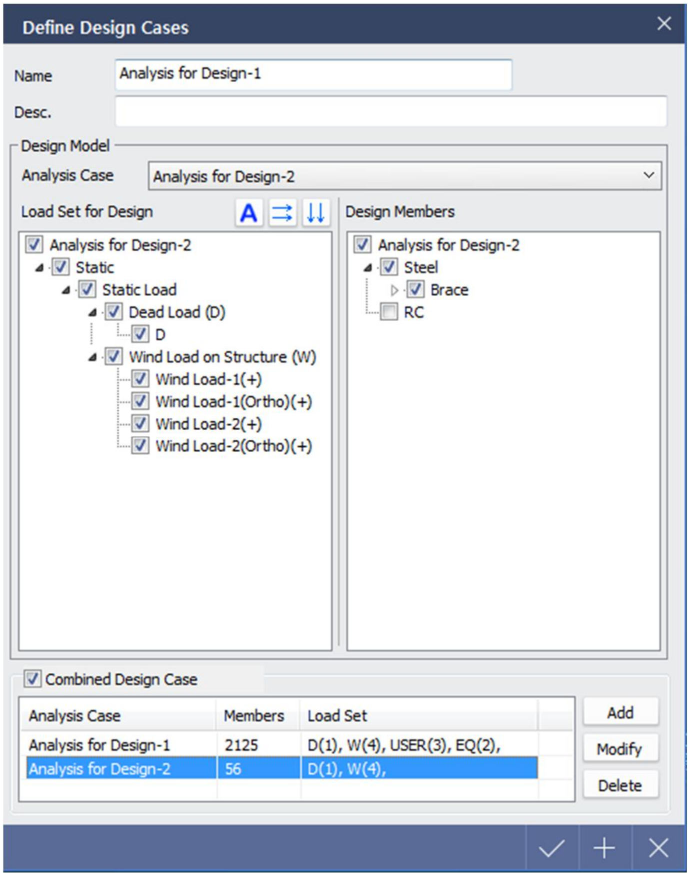
One analysis case is defined within a design case.

The design is conducted using the target members, loads, and boundary conditions defined within the analysis case. After modeling the entire structure and analyzing it, specific members or load conditions may be selected before continuing with design. Using this, various analysis or design cases can be created and separated/merged/connected designs can be conducted and critical members (for each load or type of structural member) can be identified easily.

Various design cases are defined.

Various design cases are defined and then analysis is conducted. Results for each design case can be seen, and the least optimal case is called the Envelope Design Case. The Envelope Design Case result is provided to the user to quickly check the structure's overall appearance. Additionally, when defining design cases, the "Combined Design Case" capability may be used to create combinations of various loading or boundary conditions. Thus, this capability can be used in parametric design and the strength results can be quickly combined to apply the results in structural design.

Figure 1.2.45 Dialog Window for combining design cases



Section 1 Outline

Steel members that are included in the analysis model are checked for adequate strength based on a user-specified design strength or on the entire steel structure.

The program offers the following design codes.

Table 2.1.1 Design codes categorized per country

US	AISC360-10(LRFD)	Load and Resistance Factor Design (US units)
	AISC360-10(ASD)	Allowable Strength Design (US units)
	AISC360-10M(LRFD)	Load and Resistance Factor Design (SI units)
	AISC360-10M(ASD)	Allowable Strength Design (SI 단위계)
	AISC360-05(LRFD)	Load and Resistance Factor Design (US units)
	AISC360-05(ASD)	Allowable Strength Design (US units)
	AISC360-05M(LRFD)	Load and Resistance Factor Design (SI units)
	AISC360-05M(ASD)	Allowable Strength Design (SI units)
	AISC-ASD89	Allowable Strength Design
Eurocode	EN1993-1-1-2005	Limit State Design
	EN1993-1-1-1992	Limit State Design
British	BS5950-1-1990	Limit State Design
Korean	KSSC-LSD09	Load and Resistance Factor Design

	KSSC-ASD03	Allowable Strength Design
	AIK-ASD83	Allowable Strength Design

The program supplies a design summary of the calculations for all the design criteria shown in Table 2.1.1. Detailed calculations for the most widely used design codes (AISC360-10(LRFD) / AISC360-10(ASD) / AISC360-10M(LRFD) / AISC360-10M(ASD) / AISC ASD89 / KSSC-LSD09 / KSSC-ASD03) are provided as well. In the detailed design report, the user may also see specific design code details or basis for the design calculations.

Figure 2.1.1 Member Design Report set up dialog window

Result>Design Result>Design Report

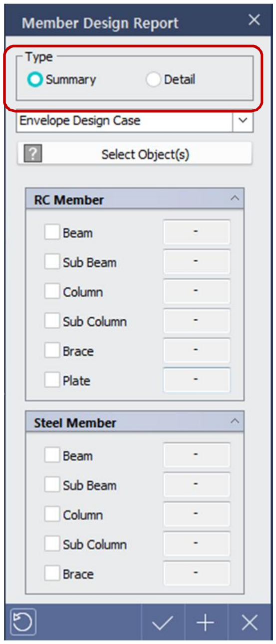


Figure 2.1.2 Sample Design Summary Report

Steel Member Design Summary Report based on AISC360-10 (LRFD) [lbf, in]

A. Member Information

a. Member Name
- Beam-5 [5]

b. Material
- SS400 (KS09) [1]
- F_y = 34.08 ksi

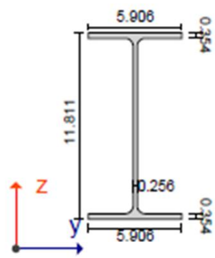
c. Section
- H 300x150x6.5/9 [3]

d. Member Length
- L = 16.404 ft
- L_y = 16.404 ft, L_z = 16.404 ft
- L_b = 0.000 ft (at 0.00L, 0.00 ft)

e. Effective Length Factor
- K_y = 0.766, K_z = 0.500

f. Member Parameters
- B_{1y} = 1.000, B_{1z} = 1.014
- B_{2y} = 1.000, B_{2z} = 1.000
- C_b = 1.472
- C_v = 1.000

g. Seismic Provision: Not Considered



A	7.2509e+000 in^2
A_{sy}	3.4875e+000 in^2
A_{sz}	3.0225e+000 in^2
I_y	1.7322e+002 in^4
I_z	1.2205e+001 in^4
S_y	2.9352e+001 in^3
S_z	4.1313e+000 in^3
Z_y	3.3075e+001 in^3
Z_z	6.4075e+000 in^3
r_y	4.8819e+000 in
r_z	1.2953e+000 in

B. Check Slenderness Ratio

Do not check because checked off Limit of Slenderness check option

C. Check Combined Ratio (at 0.00L, 0.00 ft)

Load Combination: LCB1 (1.40*D)

Axial (Compression)	$P_u/\varphi P_n$	5.04 kips / 167.99 kips = 0.030 < 1.000	Check
Moment	$M_{uy}/\varphi M_{ny}$	33.31 kips-ft / 84.55 kips-ft = 0.394 < 1.000	OK
	$M_{uz}/\varphi M_{nz}$	0.00 kips-ft / 16.38 kips-ft = 0.000 < 1.000	Check
Combined	R_{max}	0.409 < 1.000	OK

D. Check Shear Capacity (at 0.00L, 0.00 ft)

Load Combination: LCB1 (1.40*D)

Shear	$V_{uz}/\varphi V_{nz}$	12.58 kips / 61.81 kips = 0.204 < 1.000	OK

E. Check Deflection (at 0.50L, 8.20 ft)

Load Combination: svLCB1 (1.00*D)

Deflection	δ/δ_{allow}	0.17 in / 0.79 in (L/250) = 0.216 < 1.000	OK

F. Check Fatigue

Do not check because checked off Fatigue check option.

Figure 2.1.3 Sample Design Detail Report

Steel Member Design Detail Report KSSC-LSD09 [kN, m]

A. Design Case: Design Case 1

B. Member Information

a. Member Name
- Beam-75 - [157]

b. Material
- SS400 (KS09) [1]
- F_y = 235.00 MPa

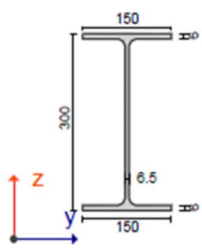
c. Section
- H 300x150x6.5/9 [3]
- Typical rolled section

d. Member Length
- L = 4.000 m
- L_y = 4.000 m, L_z = 2.000 m
- L_b = 2.000 m (at 0.00L, 0.00 m)

e. Effective Length Factor
- K_y = 0.500, K_z = 0.500

f. Member Parameters
- B_{1y} = 1.000, B_{1z} = 1.000
- B_{2y} = 1.000, B_{2z} = 1.000
- C_b = 2.394
- C_v = 1.000

g. Seismic Provision - Not Considered



A	4.678e+003 mm^2
A_{wy}	2.250e+003 mm^2
A_{wz}	1.950e+003 mm^2
I_y	7.210e+007 mm^4
I_z	5.080e+006 mm^4
S_y	4.810e+005 mm^3
S_z	6.770e+004 mm^3
Z_y	5.420e+005 mm^3
Z_z	1.050e+005 mm^3
r_y	1.240e+002 mm
r_z	3.290e+001 mm

C. Position Result

a. Design Result by Check Position

Position		0.00L	0.25L	0.50L	0.75L	1.00L
Combined	LCB	LCB38_2	LCB38_2	LCB19_3	LCB19_3	LCB19_3
	P_u (kN)	1.55	1.55	2.84	2.84	2.84
	M_y (kN·m)	-3.62	-1.60	-0.85	0.93	1.26
	M_z (kN·m)	-0.64	-0.31	-0.12	0.32	0.69
	Ratio	0.06	0.03	0.01	0.02	0.04
Shear-z	LCB	LCB19_4	LCB19_4	LCB19_4	LCB19_4	LCB19_4
	V_z (kN)	-2.96	-2.54	-2.33	-0.69	-0.28
	Ratio	0.01	0.01	0.01	0.00	0.00
Shear-y	LCB	LCB38_2	LCB38_2	LCB19_3	LCB19_3	LCB19_3
	V_y (kN)	-0.33	-0.33	-0.36	-0.36	-0.36
	Ratio	0.00	0.00	0.00	0.00	0.00

b. Description of Load Combinations

LCB19_3: 1.20*D+1.30*W

LCB19_4: 1.20*D+1.30*W

LCB38_2: 0.90*D-1.00*EQY

D. Check Slenderness Ratio

L/r_y = 32.258 < 300.000	L/r_z = 60.790 < 300.000
Major(Y) axis for tension	Minor(Z) axis for tension
4.000 / 0.124 = 32.258	2.000 / 0.033 = 60.790

E. Check Combined Ratio (at 0.00L, 0.00 m)

Load Combination: LCB38_2 (0.90*D-1.00*EQY)

Axial	$P_u/\varphi P_n$	1.550 kN / 989.397 kN = 0.002 < 1.000	Need Check

Check Axial Strength

1) Calculate axial tensile strength (φP_n) 0704.2

φ = 0.900

$P_n = A_g F_y$ = 1099.330 kN

φP_n = 989.397 kN

Moment	$M_{uy}/\varphi M_{ny}$	-3.619 kN·m / -114.633 kN·m = 0.032 < 1.000	Need Check
	$M_{uz}/\varphi M_{nz}$	-0.644 kN·m / -22.208 kN·m = 0.029 < 1.000	Need Check

Check Flexural Strength About Major Axis (Y)

1) Plastic section modulus referred to tension and compression flange

Z_y = 542000.000 mm³

2) Compression flange yielding 0706.2.1

$M_p = F_y Z_y$ = 127.370 kN·m

3) Calculate limiting width-thickness ratio of flange for flexure 0702.4

$\lambda_p = 0.38\sqrt{E_s/F_y}$ = 11.223

$\lambda_r = 1.00\sqrt{E_s/F_y}$ = 29.535

4) Calculate limiting width-thickness ratio of web for flexure 0702.4

$\lambda_p = 3.76\sqrt{E_s/F_y}$ = 111.053

$\lambda_r = 5.70\sqrt{E_s/F_y}$ = 168.352

5) Check width-thickness ratio of flange

λ_f = 8.333 < λ_p = 11.223 - Compact

6) Check depth-thickness ratio of web

λ_w = 39.385 < λ_p = 111.053 - Compact

7) Lateral torsional buckling 0706.2.2

$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$ = 1710.218 mm

$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{J_c}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 (\frac{0.7F_y}{E} \frac{S_x h_o}{Jc})^2}}$ = 5141.934 mm

$M_{n,LTB} = C_b(M_p - (M_p - 0.7F_y S_x(\frac{L_b - L_p}{L_r - L_p})))$ = 295.250 kN·m

8) Calculate flexural strength about major axis (φM_{ny}) 0706.1

φ = 0.900

$\varphi M_{ny} = \varphi min(M_p, M_{n,LTB})$ = 114.633 kN·m

Check Flexural Strength About Minor Axis(Z)

1) Elastic section modulus referred to tension and compression flanges

S_z = 67700.000 mm³

2) Calculate Limiting width-thickness ratio of flange for flexure 0702.4

$\lambda_p = 0.38\sqrt{E_s/F_y}$ = 11.223

$\lambda_r = 1.00\sqrt{E_s/F_y}$ = 29.535

3) Check width-thickness ratio of flange

λ_y = 8.333 < λ_p = 11.223 - Compact

4) Calculate nominal flexural strength for Yielding 0706.6.1

$M_p = min(F_z Z_y, 1.6F_z S_y)$ = 24.675 kN·m

5) Calculate nominal flexural strength for flange local buckling (FLB) 0706.6.2

$\lambda = b/t$ = 8.333 - Compact

Compact Flange - Not Applied

6) Calculate flexural strength about minor axis (φM_{nz}) 0706.1

φ = 0.900

$\varphi M_{nz} = \varphi min(M_p, M_{n,FLB})$ = 22.208 kN·m

Combined	R_{max}	0.061 < 1.000	Need Check

Check Combined Strength

1) Calculate interaction ratio of combined strength 0708.1.1

$P_r/P_c < 0.2$

$\frac{P_r}{2P_c} + (\frac{M_{ry}}{M_{cy}} + \frac{M_{rz}}{M_{cz}})$ = 0.061

F. Check Shear Capacity Minor Axis (at 0.50L, 2.00 m)

Load Combination: LCB19_3 (1.20*D+1.30*W)

Shear	$V_{uy}/\varphi V_{ny}$	-0.362 kN / -342.630 kN = 0.001 < 1.000	Need Check

Check Shear Strength About Minor Axis (Y)

1) Calculate web shear coefficient — 0707.7

k_v = 1.200

$h/t_w = b/2t_f$ = 8.333

$1.10\sqrt{k_v E/F_y}$ = 35.590

$1.37\sqrt{k_v E/F_y}$ = 44.326

C_v = 1.000

2) Calculate shear strength about minor axis — 0707.2.1

A_w = 2700.000 mm²

$V_{ny} = 0.6F_y A_w C_v$ = 380.700 kN

φ = 0.900

φV_{ny} = 342.630 kN

G. Check Shear Capacity Major Axis (at 0.00L, 0.00 m)

Load Combination: LCB19_4 (1.20*D+1.30*W)

Shear	$V_{uz}/\varphi V_{nz}$	-2.960 kN / -274.950 kN = 0.011 < 1.000	Need Check

Check Shear Strength About Major Axis (Z)

1) Calculate web shear coefficient — 0707.2.1

k_v = 5.000

h/t_w = 39.385

$1.10\sqrt{k_v E/F_y}$ = 72.648

$1.37\sqrt{k_v E/F_y}$ = 90.479

C_v = 1.000

2) Calculate shear strength about major axis — 0707.2.1

A_w = 1950.000 mm²

$V_{nz} = 0.6F_y A_w C_v$ = 274.950 kN

φ = 1.000

φV_{nz} = 274.950 kN

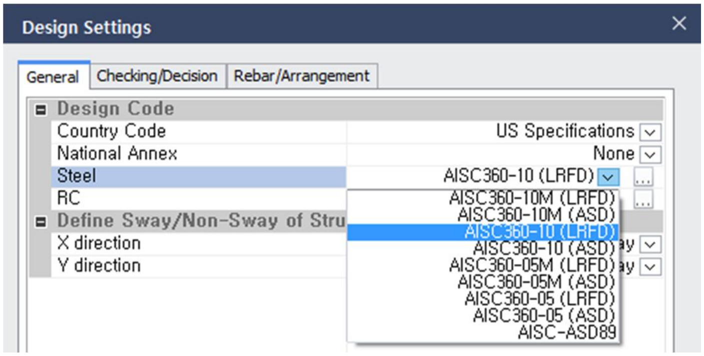
H. Check Deflection (at 0.25L, 1.00 m)

Load Combination: svLCB1 (1.00*D)

Deflection	δ/δ_{allow}	0.073 mm / 11.111 mm = 0.007 < 1.000	OK

The design code may be set in *Home>Design Settings>General>Design Code*. When the country code is selected, the available design codes will be shown.

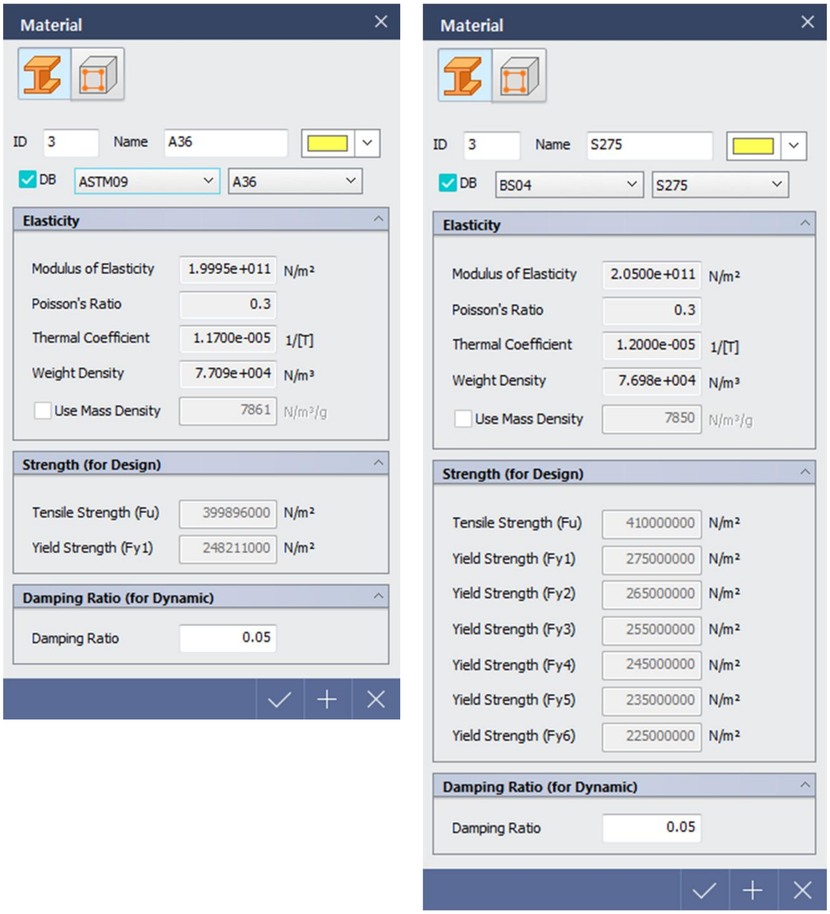
Figure 2.1.4 Design Code



The program also offers a material database for steel, and each database applies different tensile strengths and yield strengths to the design.

ASTM/ASTM09 (미국), BS/BS04 (영국), CNS/CNS06 (대만), CSA (캐나다), DIN (독일)
EN/EN05-PS/EN05-SW/EN05 (유럽), GB/GB03/GB12/JGJ/JTG04/JTJ/TB05 (중국)
GOST-SNIP/GOST-SP (러시아), IS (인도), JIS-Civil/JIS (일본), UNI (이탈리아)
KS-Civil/KS08-Civil/KS08/KS09/KS10-Civil/KSCE-LSD12 (한국)

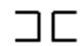
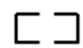
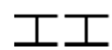
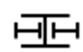
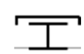
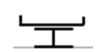
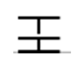
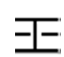
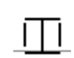
Figure 2.1.5 Material set up dialog window



The design of steel cross sections may be based on a pre-defined cross section template, modified cross section after selecting one from the template database, or a cross-section based on a user-uploaded DWG file. When the cross section comes from a template, the cross section shape that is included in the design code follows said code. If it is a shape that is not included in the selected design code, it is overridden with a shape that is included in the code before proceeding with design computations. Shapes that cannot be overridden may be used in the design by setting the typical cross section material strength reduction factor or effective design reduction factors.

The overriding procedure for cross sections that were selected from a cross section template is shown below.

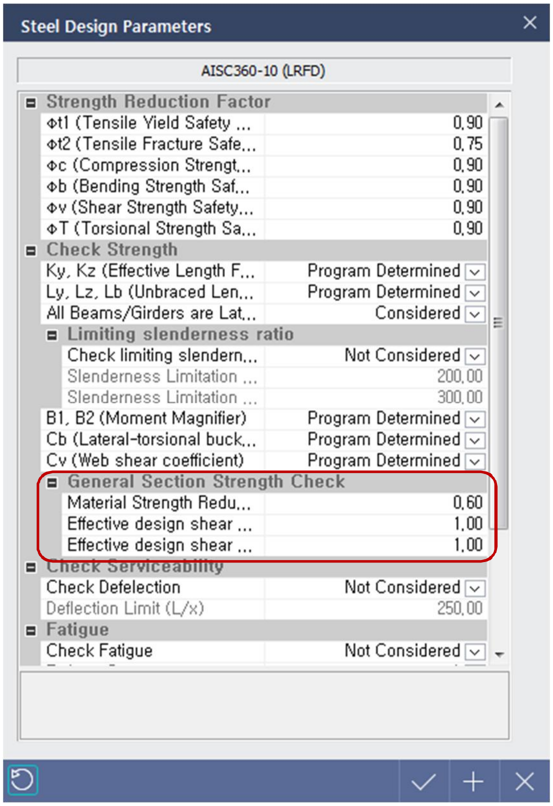
Table 2.1.2 Overriding the shapes of cross sections chosen from template designs

Type	Combine Type	Cross Section Shape	Overriding Design Section
Double Angle	-		Double Angle (T)
Double Channel	Back		Double Channel (H)
	Face		Box
Double H-Section	-		Box
2H Combined Shape	Vertical		H-Shape with Flange Plate
3H Combined Shape	-		Biaxial H-Shape
H-C Combined Shape	Downward		H
	Upward		H
H-T Combined Shape	Vertical		H-Shape with Flange Plate
H-Shape with Flange Plate	-		H-Shape with Flange Plate
H-Shape with Web Plate	-		Box
2T Combined Shape	Face		H
4-Angle	-		Box

General section strength check design factors may be input for either all applicable members or a specific member in the following dialog window.

Home>Design Settings>General>Design Code>Steel>Code-Specific Steel Design Parameters

Figure 2.1.6 Steel Design Parameters dialog window



Analysis & Design>Analysis & Design>Member Parameters

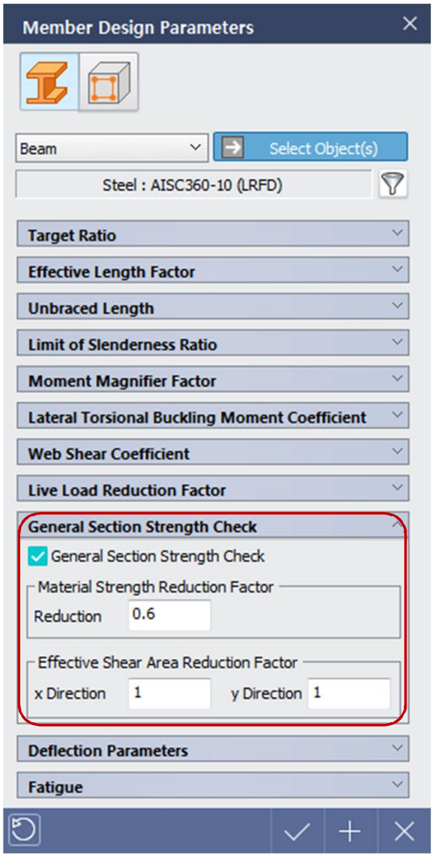


Figure 2.1.7 Member Design Parameters dialog window

Section 2

Design Factors – AISC360-10

Strength Reduction Factor / Safety Factor

This section explains the selection of strength reduction factors and safety factors for tension, compression, bending, shear, and torsion. If the load and resistance factor design (LRFD) method is being used, then strength reduction factors are selected, and if the allowable strength design (ASD) method is being used, then safety factors are selected. Recommended industry values are used as program default values, and the user may modify these values. The factors are selected and applied to the entire model.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

Figure 2.2.1 Strength reduction factor (LRFD) / safety factor (ASD) set up

Strength Reduction Factor	
Φt1 (Tensile Yield Safety ...	0.90
Φt2 (Tensile Fracture Safe...	0.75
Φc (Compression Strengt...	0.90
Φb (Bending Strength Saf...	0.90
Φv (Shear Strength Safety...	0.90
ΦT (Torsional Strength Sa...	0.90

(LRFD)

Safety Factor	
Ωt1 (Tensile Yield Safety ...	1.67
Ωt2 (Tensile Fracture Safe...	2.00
Ωc (Compression Strengt...	1.67
Ωb (Bending Strength Saf...	1.67
Ωv (Shear Strength Safety...	1.67
ΩT (Torsional Strength Sa...	1.67

(ASD)

Setting Lateral Bracing for all Beams and Girders

This section discusses how to select criteria for lateral bracing of beams and girders. After selecting “considered” for lateral bracing, lateral-torsional buckling failure mode is not considered.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

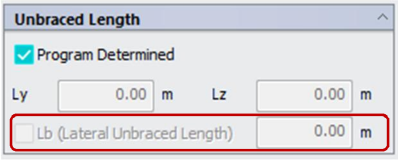
Figure 2.2.2 Beam/Girder Lateral Bracing set up dialog window for the entire model

Check Strength	
Ky, Kz (Effective Length F...	Program Determined
Ly, Lz, Lb (Unbraced Len...	Program Determined
All Beams/Girders are Lat...	Considered

Lateral torsional buckling criteria may be set for each member. To enter the unbraced length, the lateral unbraced length (L_b) should be checked. Then, the user may specify an unbraced length for that member.

Analysis & Design>Analysis & Design>Member Parameters>Unbraced Length

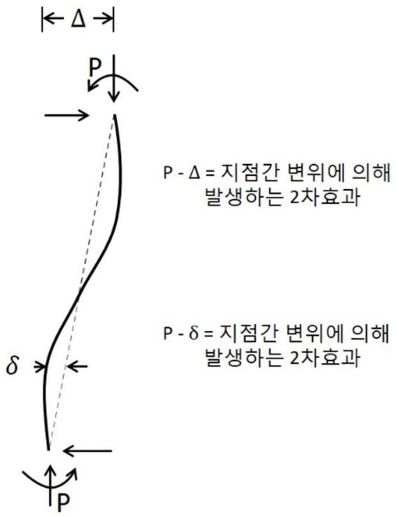
Figure 2.2.3 Member-specific unbraced length set up dialog window



Moment Magnifiers

The entire structure or a specific member may be subject to 2nd order effects (P-Δ, P-δ effects) and to incorporate such effects, second order analysis may be approximated by using first order analysis results and a moment magnifier.

Figure 2.2.4 P-δ Effect



B_1 is a magnification factor that incorporates P-δ effects that may occur due to member strains. This program offers automatic calculation of this magnification factor and is computed as follows:

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \tag{2.2.1}$$

Here,

α = 1.0 (LRFD), 1.6 (ASD)

C_m = Factor assuming no lateral displacement of the frame

With no lateral loads, $C_m = 0.6 - 0.4\frac{M_1}{M_2}$

With lateral loads, $C_m = 1.0$

$$P_{e1} = \frac{\pi^2 EI*}{(K_1 L)^2}$$

$$P_r = P_{nt} + B_2 P_{lt}$$

B_2 is a magnification factor that incorporates P-Δ effects due to displacement along the length between two points on a member, and is computed as follows:

$$B_2 = \frac{1}{1 - \alpha\frac{P_{story}}{P_{e\,story}}} \geq 1 \tag{2.2.2}$$

Here,

α = 1.0 (LRFD), 1.6 (ASD)

P_{story} : Total vertical load carried by the story

$P_{e\,story}$: Elastic buckling strength of the story belonging to the laterally unbraced frame

However, this program does not support automatic computation of the B2 magnification factor. Thus, if the user does not specify a value for B2, a default value of 1.0 is used. To consider more accurate 2nd order effects, results from P-Δ analysis may be used in design.

The moment magnifiers may be automatically computed for the entire model. For specific members, the moment magnifier factors can either be automatically computed or specified by the user.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

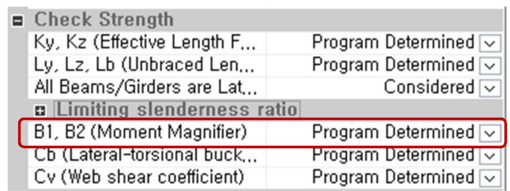


Figure 2.2.5 Dialog window showing moment magnifier automatic computation

Analysis & Design>Analysis & Design>Member Parameters>Moment Magnifier Factor

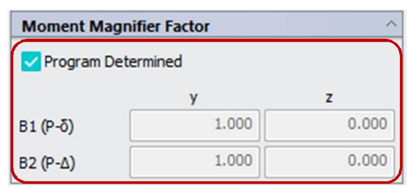


Figure 2.2.6 Dialog window showing moment magnifier selection for a specific

Lateral Torsional Buckling Modification Factor

If the beam is not subject to distributed moment, nominal bending strength is computed as the product of the basic strength and the lateral torsional buckling modification factor C_b. C_b is computed according to Equation 2.2.3, and can be assumed to be 1.0 as a conservative estimate.

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_c} \qquad (2.2.3)$$

Here,

M_{max} = Absolute value of the maximum moment in the unbraced length

M_A = Absolute value of the moment at a quarter length along the unbraced length

M_B = Absolute value of the moment at halfway along the unbraced length

M_C = Absolute value of the moment three quarters length along the unbraced length

The lateral-torsional buckling factor may be computed automatically for the entire model. For specific members, either the automated or user-specified values may be used.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

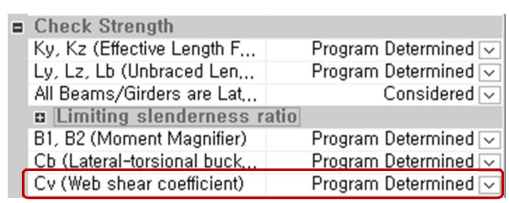


Figure 2.2.7 Dialog window for setting up the lateral-torsional buckling factor for

Analysis & Design>Analysis & Design>Member Parameters>Lateral Torsional Buckling Moment Coefficient

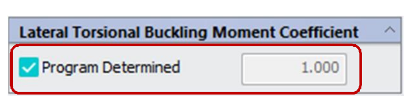


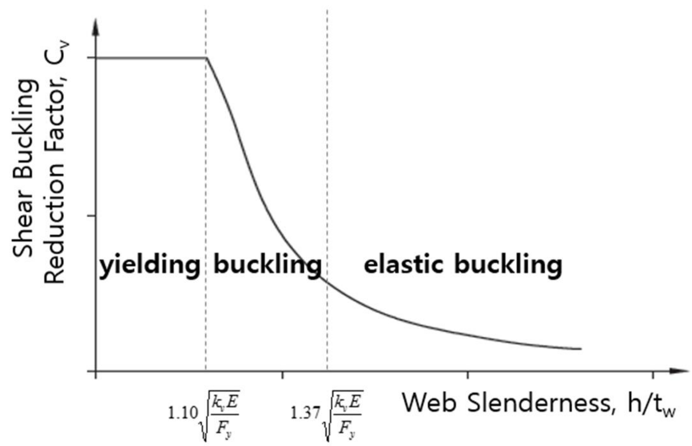
Figure 2.2.8 Dialog window for setting up the lateral torsional buckling factor for a

Web Shear Coefficient

Web shear coefficient or shear buckling reduction factor C_v is used in conjunction with material strength when computing the nominal shear strength, and is computed based on the ratio of the web's depth and thickness.

Except for circular hollow sections, all dual-axis symmetry cross sections, single-axis symmetry cross sections, and C-shape sections are divided into the three categories shown below in Figure 2.29 to compute the shear buckling reduction factor.

Figure 2.2.9 Web Shear Coefficient



In each section, the calculation for C_v is as follows.

Table 2.2.1 C_v equations for each condition

Condition	C_v	V_n based on
$h/t_w \le 1.10\sqrt{k_v E/F_y}$	$C_v = 1.0$	Shear yielding
$1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$	$C_v = \dfrac{1.10\sqrt{k_v E/F_y}}{h/t_w}$	Buckling

$h/t_w > 1.37\sqrt{k_v E / F_y}$	$C_v = \dfrac{1.51 E k_v}{(h/t_w)^2 F_y}$	Elastic buckling

However, for sections with $h/t_w \le 2.24\sqrt{E/F_y}$ or rolled H-shape webs, C_v = 1.0.

For the entire model, the web shear coefficient may be computed automatically. For individual members, either the automated values or user-specified values may be used.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

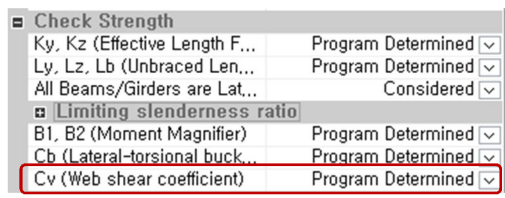


Figure 2.2.10 Dialog window for setting the web shear coefficient for the entire

Analysis & Design>Analysis & Design>Member Parameters>Web Shear Coefficient

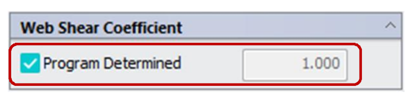


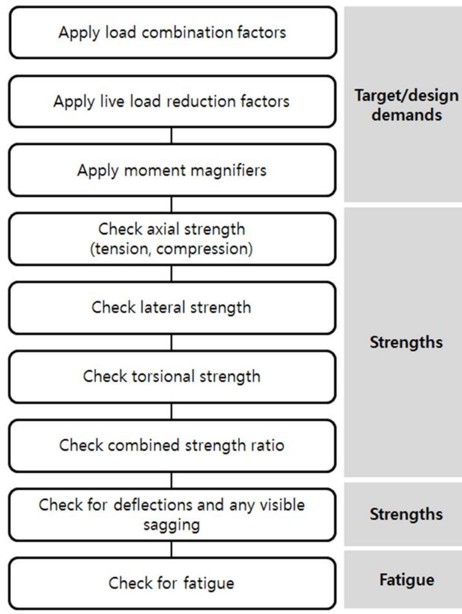
Figure 2.2.11 Dialog window for setting the web shear coefficient for a specific

Section 3

Member Examination Procedure – AISC360-10

The steel member examination procedure following the AISC360-10 design code is as follows.

Load combination or design strength modification factors are applied to the analyzed strengths, and the various strengths are calculated based on the design code and then checked for safety. In particular for beam members, the program will check for sag, and for repeated loads (such as crane girders), fatigue checks will also be conducted.



Calculation of Design Demands

Design demands/forces are computed by applying load combinations, live load reduction factors, and moment magnification factors to the analyzed strengths.

1) Applying load combinations

The analyzed demands at the member check points, load types, and the load combination factors are incorporated in computing the design demands.

2) Applying the live load reduction factor

As explained in the section Design Factors>Live Load Reduction Factors, components that are subject to live loads will have design forces that incorporate the live load reduction factor.

3) Applying the moment magnifier

The following equation describes how to compute the second order moment and axial force incorporating the first order strength and moment magnifier. In the case of axial force, the following equation is only applied for compressive forces.

$$M_r = B_1 M_{nt} + B_2 M_{lt} \tag{2.3.1}$$

$$P_r = P_{nt} + B_2 P_{lt} \tag{2.3.2}$$

Here,

M_{nt} = 1st order moment when there is no lateral deflection of the structure. In this program, this moment is due to dead and live loads.

M_{lt} = 1st order moment when there is lateral deflection of the structure. In this program, this moment is due to all loads except for dead and live loads.

P_{nt} = 1st order axial force when there is no lateral deflection of the structure. In this program, this force is due to dead and live loads.

P_{lt} = 1st order axial force when there is lateral deflection of the structure. In this program, this force is due to all loads except dead and live loads.

Calculation of Design Strengths

The design strength of each member is based on load combinations and must be greater than the calculated required strength.

$$R_u \le \phi R_n \text{ (LRFD)} \tag{2.3.3}$$

$$R_u \le \frac{R_n}{\Omega} \text{ (ASD)} \tag{2.3.4}$$

Here,

R_u : Required strength

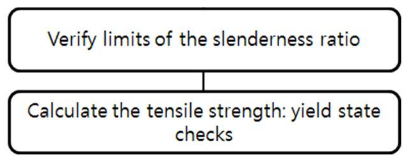
R_n : Nominal strength

Φ : Strength reduction factor

Ω : Safety factor

1) Axial strength: Tension members

Members subject to tension about the central axis must undergo checks regarding slenderness ratio limits and design elongation/tensile strengths.



Slenderness ratio limits for tension members are created to limit sagging or vibration due to self weight. Moreover, if the tension members are too flexible, the member may be twisted severely in the process of transportation or construction. As a result, proper examination is recommended. The default recommended limit is 300, but the user may specify an alternative value.

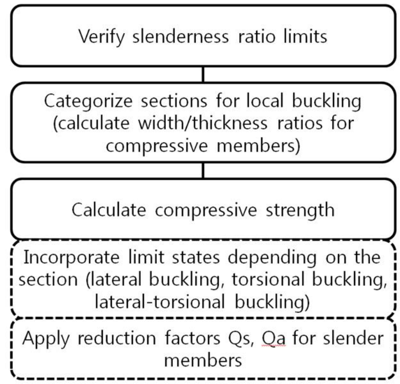
$$\frac{L}{r} \le 300 \tag{2.3.5}$$

The nominal strength of a tension member is typically set to be the minimum of the gross section yielding limit state and the effective cross section yielding limit state. However, this program does not support connection designs and thus the yield limit state for only the gross cross section is calculated.

$$P_n = F_y A_g \tag{2.3.6}$$

2) Axial strength: Compression members

Members subject to compression about the central axis must undergo checks regarding slenderness ratio limits and design compressive strengths.



Unlike tension members, the slenderness ratio limits for compression members incorporate an effective length factor K. The default recommended slenderness ratio limit is 200, but the user may specify an alternative value.

$$\frac{KL}{r} \le 200 \tag{2.3.7}$$

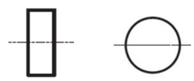
For compression members, long and slender elements incorporate the reduction factor $Q=Q_sQ_a$when calculating the design compressive strength. To do this, the cross section must be properly categorized using the width to thickness ratio.

The nominal strength of a compression member, depending on the cross-section shape, is selected to be the minimum of the flexural buckling (FB) limit state, torsional buckling (TB) limit state, and the flexural-torsional buckling (FTB) limit state.

$$P_n = F_{cr} A_g \tag{2.3.8}$$

Table 2.3.1 Limit states depending on the cross-section shape

Cross-Section Shape	Limit State	
	Compact/Noncompact Section	Slender Section

	FB TB	LB FB TB
	FB FTB	LB FB FTB
	FB	LB FB
	FB FTB	FB FTB
	FB	N/A
Asymmetrical shapes (except for L-shaped sections)	FTB	LB FTB

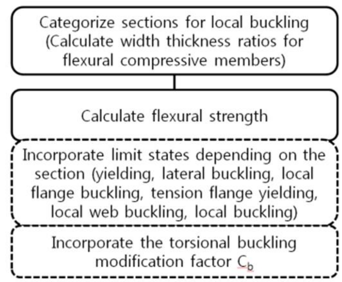
FB=flexural buckling, TB=torsional buckling, FTB=flexural-torsional buckling, LB=local buckling

For a typical cross section, F_{cr} is the product of the steel yield strength F_y and the typical section material strength reduction factor. The nominal compressive strength is calculated as follows.

$$P_n = (Material \quad Strength \quad Factor) \times F_y A_g \tag{2.3.9}$$

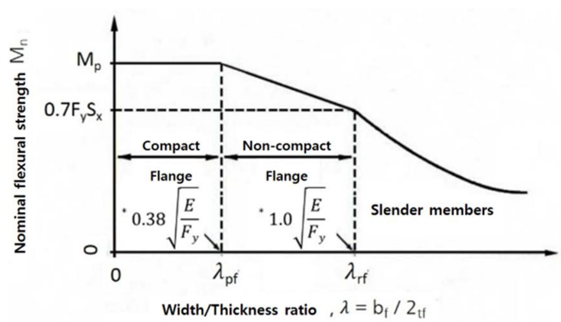
3) Flexural/Bending Strength

Nominal flexural strength is determined based on the member's width to thickness ratio and the laterally unbraced length. Moreover, various limit states depending on the section's shape or flexural-compressive member's cross-section category (compact, noncompact, slender) are also examined, and the minimum value of those limit states is used.



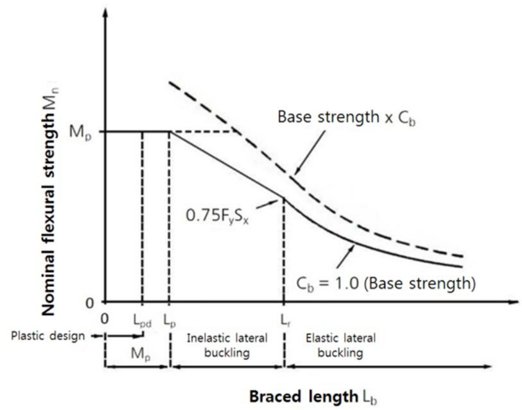
The nominal flexural strength of a member and how it is a function of the width to thickness ratio is shown in Figure 2.3.1. As shown below, the calculation may be split into three areas of compact, noncompact, and slender members. As the width to thickness ratio increases, the nominal flexural strength decreases.

Figure 2.3.1 Nominal flexural strength based on the width to thickness ratio



The nominal flexural strength of a member and how it is a function of the member's laterally unbraced length is shown below in Figure 2.3.2. The computation may be split into four areas of plastic design, full plastic, inelastic lateral buckling, and elastic lateral buckling. As the laterally unbraced length increases, the nominal flexural strength decreases.

Figure 2.3.2 Nominal flexural strength based on the member's laterally unbraced



If the beam is not subject to distributed moment, the nominal flexural strength is computed as the product of the basic strength and the modification factor C_b.

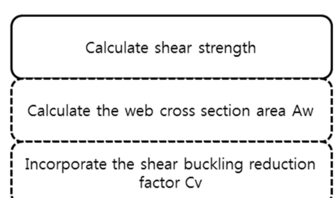
For typical cross-sections, the nominal flexural strength, such as in the case of compression members, the material strength reduction factor of typical cross-sections is incorporated into the calculation.

$$M_n = (Material \quad Strength \quad Factor) \times F_y Z \tag{2.3.10}$$

Here,

Z = Plastic section modulus about the deflection axis.

4) Shear Strength



The nominal shear strength of the web is computed as follows, depending on the cross-section shape.

Table 2.3.2 Nominal shear strength of webs depending on the cross-section shape

Cross-Section Shape	Nominal Shear Strength V_n	Shear Area A_w

Single- or dual-axis symmetry members, channel shapes' webs	$V_n = 0.6F_y A_w C_v$	$A_w = b_w h_w$
L-shape cross-sections	$V_n = 0.6F_y A_w C_v$	$A_w = bt$
Square-shape or box-shape hollow section	$V_n = 0.6F_y A_w C_v$	$A_w = ht$
Circular hollow section	$V_n = F_{cr} A_g / 2$	
Minor axis of single- or dual-axis symmetry members	$V_n = 0.6F_y A_w C_v$	$A_w = b_f h_f$

For typical cross sections, the nominal shear strength incorporates the effective shear area factors and is calculated as follows:

$$V_n = (Material\ \ Strength\ \ Factor\ ) \times F_y \times (Effective\ \ Shear\ \ Area\ \ Factor\ ) \times A_s \qquad (2.3.11)$$

5) Torsional Strength

Steel tube sections can better resist torsion compared to open type cross sections. Thus, the torsional strength is computed with the assumption that the total torsional moment is resisted by the pure torsion shear stress. For other cross-sections, the minimum value is selected from the vertical stress yielding limit state, shear stress yielding limit state, and the buckling limit state. In AISC 360-10, torsional strength equations are provided only for steel tube cross sections, so this program also supports torsional checks for steel tube cross sections.

6) Combination Strength Ratios

In cases where the member is subject to both lateral and axial loads, the following interaction equations must be satisfied:

$$P_r \geq 0.2P_c: \ \frac{P_r}{P_c} + \frac{S}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \tag{2.3.12}$$

$$P_r < 0.2P_c: \ \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \tag{2.3.13}$$

In particular, tensile loads can increase the flexural strength, and thus when calculating M_c, C_b should be multiplied by $\sqrt{1+\frac{P_r}{P_{ey}}}$ and then applied to the interaction equations..

Members subject to torsion, flexure, shear, and axial force simultaneously must satisfy the following interaction equation. According to AISC 360-10, torsional strength equations are only provided for box or pipe shape cross sections, and as a result the program checks for torsional strength for only box or pipe shape cross sections.

$$T_r \leq 0.2T_c \text{ : Neglecting torsional effects} \tag{2.3.14}$$

$$T_r > 0.2T_c: \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \tag{2.3.15}$$

Serviceability Checks

All of the structure, including each specific structural member and connections, must be checked for serviceability. When performing this check, the load factors in load combinations are all 1.0. However, the factor for earthquake loading is 0.7.

1) Sagging Checks

Excessive sagging has negative effects on the structure's appearance and performance. It may also cause damage to nonstructural components, and thus the actual deflection must be smaller than the allowable deflection.

$$\delta_{actual} \leq \delta_{allow} \tag{2.3.16}$$

The actual deflection is calculated as the product of the load combination factor and the analyzed deflection value. The allowable deflection is based on the member length and the user-specified design environment.

Fatigue Checks

If there are repeated loads on a structure, it may experience fatigue and cracks may occur. If cracks become enlarged, the structure may experience collapse. Such fatigue effects are caused by a large number of repeated stresses, and is not typically applied to building structures. Crane girders that are subject to repeated loads or structures that resist machinery or equipment may, however, experience cracks due to fatigue.

Section 4

Design Parameters – EN1993-1-1:2005

Partial Factor

This section explains how to select and apply partial factors for the cross-section ultimate limit value (γ_{M0}), partial factors for instability checks for individual members (γ_{M1}), and partial factors for resistance limit value (for tensile rupture) (γ_{M2}). Industry recommended values are used as default values in the software, but the user may modify these values. The specified partial factors are applied to the entire model.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

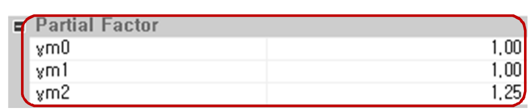


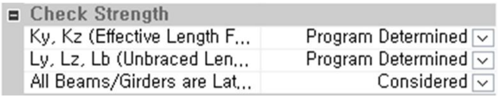
Figure 2.4.1 Dialog window for setting the partial factor

Setting Lateral Bracing for all Beams and Girders

This section explains how to set the lateral bracing conditions for all beams and girders in a model. The user can select “Considered” for “All Beams/Girders are Laterally Braced”, in which case the lateral-torsional buckling strength is not considered.

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters

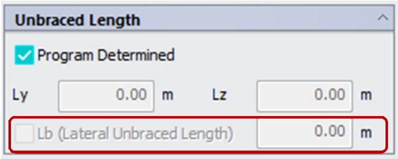
Figure 2.4.2 Dialog window for setting lateral bracing conditions for all beams and



The condition for not considering the lateral-torsional buckling strength can be specified for individual members. In setting the member parameters, when the “Lateral Unbraced Length” option (or L_b) is checked off, the program will consider this member to be laterally unbraced.

Analysis & Design>Analysis & Design>Member Parameters>Unbraced Length

Figure 2.4.3 Dialog window for setting the unbraced length for a specific member



Equivalent Uniform Moment Factor

When calculating the buckling strength of a member subject to both compression and bending, the interaction factors k_{yy}, k_{yz}, k_{zy}, k_{zz}, must first be computed. In doing so, equivalent uniform moment factors are required.

The equivalent uniform moment factors for each direction (either for lateral buckling or lateral-torsional buckling) are computed as follows.

Table 2.4.1 Equivalent uniform moment factors for each direction, based on

$\bar{\lambda}_0 \leq 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N}\right)\left(1-\frac{N_{Ed}}{N}\right)}$	C_{my} C_{mz} C_{mLT}
$\bar{\lambda}_0 > 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N}\right)\left(1-\frac{N_{Ed}}{N}\right)}$	C_{my} $C_{mz} = C_{mi,0} + (1 - C_{mi,0})\frac{\sqrt{\varepsilon_y}a_{LT}}{}$ $C_{mLT} = C_{my}^2 \frac{a_{LT}}{\sqrt{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}}$

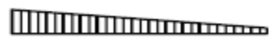
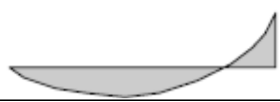
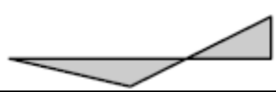
Here,

$\overline{\lambda_0}$: Dimensionless slenderness ratio based on lateral-torsional buckling due to uniform moment

C_1 : This is defined based on the loading and end conditions. It may be calculated as $C_1 = k_c^{-2}$.

k_c is a modification factor and is computed as follows.

Table 2.4.2 Moment distribution shapes and the corresponding modification

Moment distribution	k_c
 $\psi = 1$	1.0
 $-1 \leq \psi \leq 1$	$\dfrac{1}{1.33 - 0.33\psi}$
	0.94
	0.90
	0.91
	0.86
	0.77
	0.82

$N_{cr,y}$, $N_{cr,z}$: Elastic flexural buckling stress for the major and minor axes

$N_{cr,T}$: Elastic torsional buckling stress

ε_y is calculated based on the cross-section class type.

► Class 1, 2, 3 :

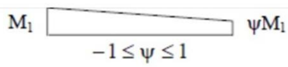
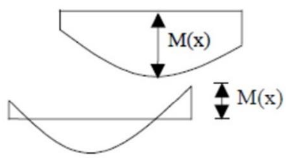
$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}} \frac{A}{W_{el,y}} \tag{2.4.1}$$

► Class 4 :

$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}} \frac{A_{eff}}{W_{eff,y}} \tag{2.4.2}$$

$C_{mi,0}$ is calculated in each direction as shown below.

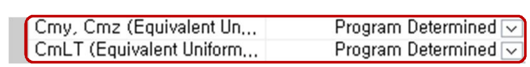
Table 2.4.3 $C_{mi,0}$ in each direction

Moment diagram	$C_{mi,0}$
 M_1 ψM_1 $-1 \le \psi \le 1$	$C_{mi,0} = 0{,}79 + 0{,}21\psi_i + 0{,}36(\psi_i - 0{,}33)\frac{N_{Ed}}{N_{cr,i}}$
 M(x) M(x)	$C_{mi,0} = 1 + \left(\frac{\pi^2 EI_i \lvert\delta_x\rvert}{L^2 \lvert M_{i,Ed}(x)\rvert} - 1\right)\frac{N_{Ed}}{N_{cr,i}}$ $M_{i,Ed}(x)$ is the maximum moment $M_{y,Ed}$ or $M_{z,Ed}$ according to the first order analyses $\lvert\delta_x\rvert$ is the maximum member deflection along the member
	$C_{mi,0} = 1 - 0{,}18\frac{N_{Ed}}{N_{cr,i}}$
	$C_{mi,0} = 1 + 0{,}03\frac{N_{Ed}}{N_{cr,i}}$

The equivalent uniform moment factor can be automatically computed and then applied to the entire model. For individual members, the automated value may be used, or the user may specify an alternative value.

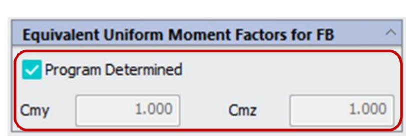
Figure 2.4.4 Dialog window for setting automation for the equivalent uniform moment

Home>Design Settings>General>Design Code>Steel>Design Code-Specific Steel Design Parameters



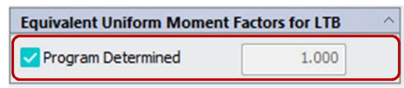
Analysis & Design>Analysis & Design>Member Parameters>Equivalent Uniform Moment Factors for FB

Figure 2.4.5 Dialog window for setting equivalent uniform moment factors for individual



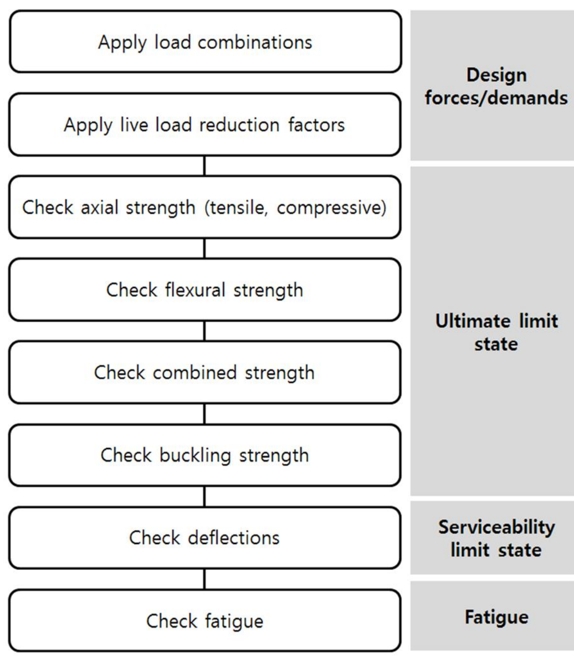
Analysis & Design>Analysis & Design>Member Parameters>Equivalent Uniform Moment Factors for LTB

Figure 2.4.6 Dialog window for setting equivalent uniform moment factors for individual members, for lateral torsional



Section 5

Member Examination Procedure – EN1993-1-1:2005



Calculating Design Strength

The design strength is calculated by incorporating load combinations and live load reduction factors into the analyzed strengths.

1) Load Combination Factors

The design strength is computed by incorporating the analyzed strengths at the member check points, load combination types, and the load combination factors.

2) Live Load Reduction Factors

As mentioned in the Section Design Factors > Live Load Reduction Factors, only the members subject to live loads will incorporate live load reduction factors for computation of design strength.

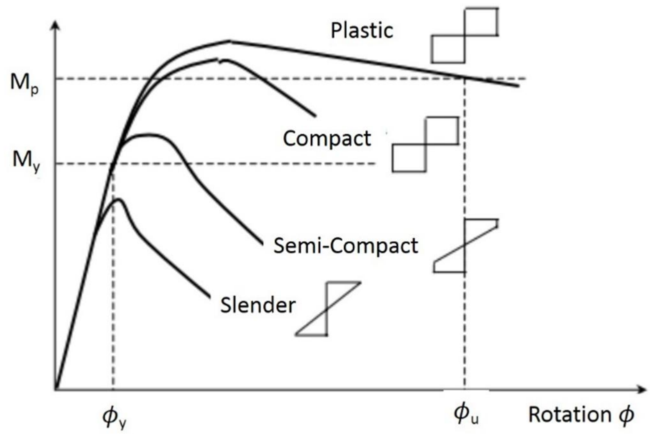
Ultimate Limit State

The design resistance value of the cross-section must be greater than the design load values, and an important factor in calculating the resistance value is the cross-section classification. The Eurocode categorizes cross-sections into four classes and the definitions for each class are shown below.

Table 2.5.1 Cross-sections categorized according to the Eurocode

Class 1	Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
Class 2	Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
Class 3	Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
Class 4	Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Figure 2.5.1 Cross-Section Classification according to Eurocode 3



1) Axial strength: Tension Members

Members subject to uniform tension must satisfy the following limit state:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \tag{2.5.1}$$

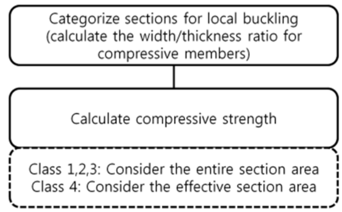
The design strength for tension members is selected to be the minimum of the gross section design plastic resistance strength and the net section design ultimate resistance strength. However, the program only considers the gross section design plastic resistance strength.

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{2.5.2}$$

2) Axial strength: Compression Members

Members subject to uniform compression must satisfy the following limit state:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \tag{2.5.3}$$



The design strength for compression members, $N_{c,Rd}$, is calculated using either the gross section area or the effective section area depending on the section class. The equations are shown below.

► Class 1, 2, 3 :

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{2.5.4}$$

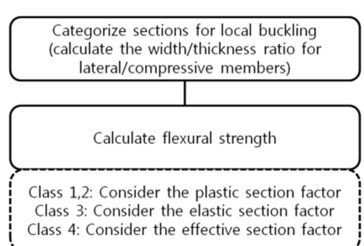
► Class 4 :

$$N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} \tag{2.5.5}$$

3) Lateral Strength

Members subject to pure lateral loads must satisfy the following limit state.

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \tag{2.5.6}$$



► Class 1, 2 :

$$M_{c,Rd} = M \qquad W_{pl}f_y \tag{2.5.7}$$

► Class 3 :

$$M_{c,Rd} = M_{el,Rd} = W_{el,min}f_y \tag{2.5.8}$$

► Class 4 :

$$M_{c,Rd} \frac{W_{eff,min}f_y}{\gamma_{M0}} \tag{2.5.9}$$

4) Shear Strength

Members subject to shear forces must satisfy the following limit state.

$$\frac{V_{Ed}}{V_{c,Rd}} \le 1.0 \tag{2.5.10}$$

Design shear strength $V_{c,Rd}$ is calculated using the following equation using the design plastic shear strength (where torsion does not exist):

$$V_{c,Rd} = V_{pl,Rd} = \frac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{M0}} \tag{2.5.11}$$

In the above equation, design cross-section area A_v is calculated using different equations for different cross-section shapes.

Table 2.5.2 Calculation of design section area A_v depending on the cross-

Rolled I, H shapes	Web	$A_v = A - 2bt_f + (t_w + 2r)t_f \le \eta h_w t_w$
Rolled C shapes	Web	$A_v = A - 2bt_f + (t_w + r)t_f$

Rolled T shapes	Web	$A_v = A - bt_f + (t_w + 2r)\dfrac{t_f}{2}$
Welded T shapes	Web	$A_v = t_w\left(h - \dfrac{t_f}{2}\right)$
Welded I, H shapes, Rectangular sections	Web	$A_v = \eta \sum (h_w t_w)$
	Flange	$A_v = A - \sum (h_w t_w)$
Uniform thickness rectangular hollow section	Section height	$A_v = A\dfrac{h}{b+h}$
	Section depth	$A_v = A\dfrac{b}{b+h}$
Uniform thickness circular hollow section	-	$A_v = \dfrac{2A}{\pi}$

5) Combined Strength

When a member is subject to both lateral and shear forces, the lateral strength may be reduced if the shear force is large. If the shear force is greater than half of the plastic shear strength, then the lateral strength is calculated by reducing the material's yield strength.

$V_{Ed} > \frac{1}{2} V_{pl,Rd}$: When calculating lateral strength, (1- ρ)fy is used instead of fy

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1 \right)^2$$

$V_{Ed} \leq \frac{1}{2} V_{pl,Rd}$: Calculation of lateral strength without any reductions due to shear force

When a member is subject to both lateral and axial forces, the member must satisfy the following equations based on its cross-section class.

► Class 1, 2 : Design plastic lateral strength is reduced due to axial force

$$M_{Ed} \leq M_{N,Rd} \tag{2.5.12}$$

► Class 3 : Consideration of the peak axial stress limit due to lateral and axial forces

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}} \tag{2.5.13}$$

► Class 4 :

$$\frac{N_{Ed}}{A_{eff} f_y / \gamma_{M0}} + \frac{M_{y,Ed} + M_{Ed} e_{Ny}}{W_{eff,y,min} f_y / \gamma_{M0}} + \frac{M_{z,Ed} + N_{Ed} e_{Nz}}{W_{eff,z,min} f_y / \gamma_{M0}} \leq 1 \tag{2.5.14}$$

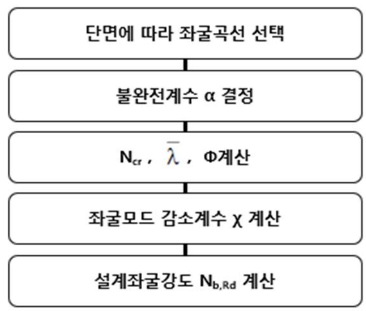
6) Buckling strength verification – axial load

Members subject to axial force must satisfy the following equation for buckling behavior:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \tag{2.5.15}$$

However, if $\bar{\lambda} \leq 0.2$ or $\frac{N_{Ed}}{N_{cr}} \leq 0.04$, buckling effects may be neglected.

Design buckling strength $N_{b,Rd}$ for an axial member is calculated as follows.



► Class 1, 2, 3 :

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \tag{2.5.16}$$

► Class 4 :

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \tag{2.5.17}$$

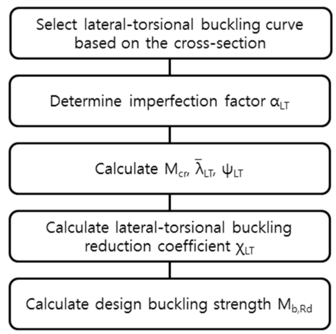
7) Buckling strength verification – lateral load

If a member that is laterally unbraced is subject to bending about the strong axis, the member must satisfy the following equation regarding lateral-torsional buckling:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \tag{2.5.18}$$

However, if the member is a beam and its compressive flange is sufficiently supported, or if the member cross-section is a square or circular hollow section, lateral-torsional buckling is not considered. Furthermore, if $\bar{\lambda}_{LT} \le \bar{\lambda}_{LT,0}$ or $\frac{M_{Ed}}{M_{cr}} \le \bar{\lambda}_{LT,0}{}^2$, lateral-torsional buckling may also be neglected.

Design buckling moment is calculated as shown below.



► Class 1, 2 :

$$M_{b,Rd} = \chi_{LT} W_{pl,y} \frac{f_y}{\gamma_{M1}} \tag{2.5.19}$$

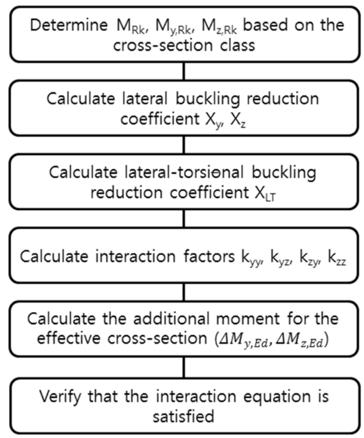
► Class 3 :

$$M_{b,Rd} = \chi_{LT} W_{el,y} \frac{f_y}{\gamma_{M1}} \tag{2.5.20}$$

► Class 4 :

$$M_{b,Rd} = \chi_{LT} W_{eff,y} \frac{f_y}{\gamma_{M1}} \tag{2.5.21}$$

8) Buckling strength of a member subject to both axial and lateral loads



Depending on the cross-section class, N_{Rk}, $M_{y,Rk}$, $M_{z,Rk}$, $\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$ are calculated differently, as shown below.

$$N_{Rk} = f_y A_i \tag{2.5.22}$$

$$M_{i,Rk} = f_y W_i \tag{2.5.23}$$

Table 2.5.3 Calculating N_{Rk}, $M_{y,Rk}$, $M_{z,Rk}$ based on the cross-section class

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y} N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z} N_{Ed}$

Lateral buckling reduction coefficients χ_y, χ_z are calculated with different parameters for each direction.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1.0 \tag{2.5.24}$$

Here,

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$\bar{\lambda}$: dimensionless slenderness ratio

► Class 1, 2, 3:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \tag{2.5.25}$$

► Class 4:

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \tag{2.5.26}$$

α : Imperfection factor for the buckling curve

Table 2.5.4 Imperfection factor α for the buckling curve

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

N_{cr} : Elastic critical force.

The lateral-torsional buckling reduction coefficient χ_{LT} is calculated as follows.

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \leq 1.0 \tag{2.5.27}$$

Here,

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

α_{LT} : Imperfection factors for the lateral-torsional buckling curve

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0.21	0.34	0.49	0.76

M_{cr} : Elastic critical moment

Interaction factors k_{yy}, k_{yz}, k_{zy}, k_{zz} may be calculated using either Annex A or Annex B. midas Plan uses equations of Annex A.

Table 2.5.5 Calculation of interaction factors – Annex A

Interaction factors	Design assumptions	
	Elastic cross-sectional properties class 3, class 4	**Plastic cross-sectional properties class 1, class 2**
k_{yy}	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{yz}}0.6\sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{mz}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\dfrac{1}{C_{zy}}0.6\sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{zz}}$

Members subject to both axial and lateral loads must satisfy the following interaction equations.

$$\frac{N_{Ed}}{\dfrac{\chi_z N_{Rk}}{\gamma_{M1}}}+k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\dfrac{M_{y,Rk}}{\gamma_{M1}}}+k_{yz}\frac{M_{y,Ed}+\Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \tag{2.5.28}$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \tag{2.5.29}$$

Serviceability Limit State

It is important to verify the serviceability of the entire structure, each individual member, connections, and joints. When checking for serviceability, the load factor used in all load combinations is set to be 1.0 (the load factor for earthquake loading is set to be 0.7).

1) Deflection Checks

Excessive deflection negatively affects the structure's appearance and performance. It can also damage the nonstructural components. Thus, the actual deflection must be smaller than the allowable deflection.

$$\delta_{actual} \leq \delta_{allow} \tag{2.5.30}$$

The actual deflection is the product of the analyzed deflection and the load combination factors. The allowable deflection is calculated using the user-specified ratio to be applied to the member length.
The Eurocode checks for the beams' vertical deflection and the columns' horizontal deflection.

Fatigue Checks

If there are repeated loads on a structure, it may experience fatigue and cracks may occur. If cracks become enlarged, the structure may experience collapse. Such fatigue effects are caused by a large number of repeated stresses, and is not typically applied to building structures. Crane girders that are subject to repeated loads or structures that resist machinery or equipment may, however, experience cracks due to fatigue.

Section 6

Cross-Section Computations

In this program, member strength and the user-specified control data is used to create the steel cross-sections. However, the cross-sections must satisfy the criteria shown below.

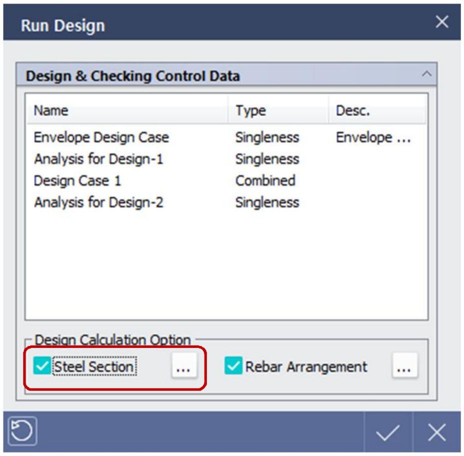
Table 2.6.1 Domain of available steel cross-sections for midas nGen

Cross-Section Shape	Cross-Section DB
H Shape	All DB
C Shape	All DB
L Shape	All DB
T Shape	All DB
Rectangular Hollow Section	All DB
Circular Hollow Section	All DB
2L Shape	AISC2K(US), AISC2K(SI), AISC, CNS91, BS4-93, GB-YB05

Steel Section Calculation Set Up

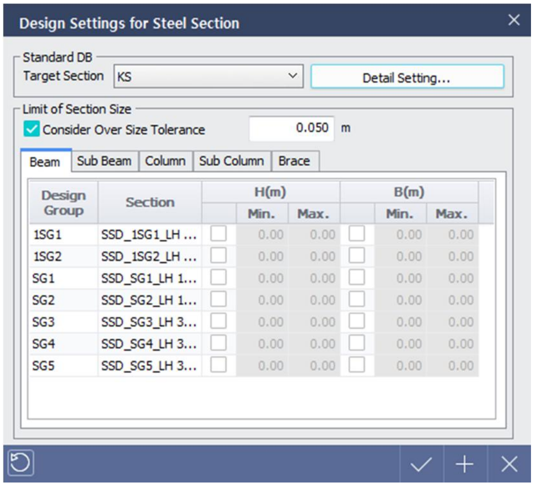
When the design process begins, the dialog window shown below will appear. When *Design Calculation Option>Steel Section* is Checked-On, then the program will find sections that satisfy the member strength and other criteria, which will then be reflected in the model and afterwards the design calculations will be repeated.

Figure 2.6.1 Run Design dialog window



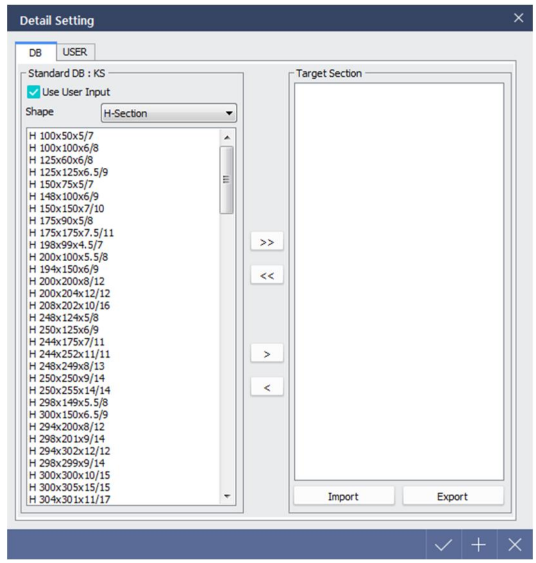
The user may click the […] button to access more detailed design settings to modify section criteria. Depending on the member type design group, the section's depth and height ranges may be specified.

Figure 2.6.2 Design Settings for Steel Section dialog window



In the case of steel members, a more efficient section may be found by browsing the candidate section list. To access this, the user may press the *Detail Setting* button. The user may then select the target candidate sections from either the DB or user-specified sections.

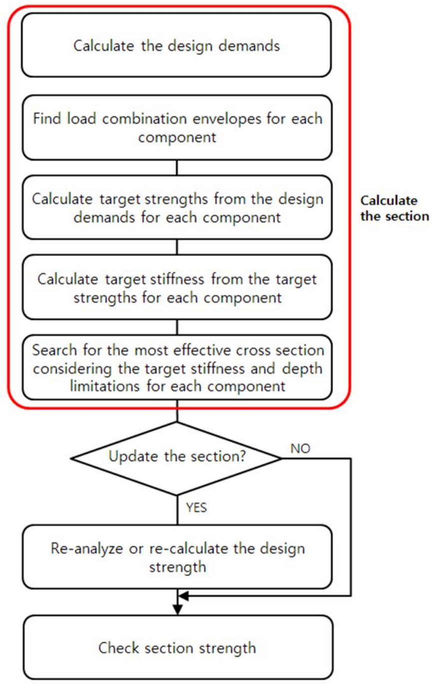
Figure 2.6.3 Detail Setting dialog window



Steel Section Calculation Process

After modeling and analysis, the steel section calculation process is as shown below.

Figure 2.6.4 Steel member section calculation process



1) Calculate the design demands

Calculate the design demands by applying the load combination and design code modification factors.

2) Find load combination envelopes for each component

midas nGen does not go through the section calculation process for all load combinations. The program finds the load combination that yields the maximum and minimum values of the member's axial, shear, torsional, and lateral demands, in order to find the design with the largest possibility of becoming the governing design. Shear and moment demands must also consider both the major and minor axes, and thus a total of 12 load combinations is required for proper section calculation.

P_{max}	$V_{y,max}$	$V_{z,max}$	T_{max}	$M_{y,max}$	$M_{z,max}$
P_{min}	$V_{y,min}$	$V_{z,min}$	T_{min}	$M_{y,min}$	$M_{z,min}$

3) Calculate target strengths from the design strengths for each component

To satisfy the inequality (Design demand) ≤ (Design strength) X (Target ratio), the target strengths are calculated for each component of the internal member forces and design demands.

4) Calculate target stiffnesses from the target strengths for each component

The target stiffnesses are computed from the target strengths for each force component.

Table 2.6.2 Equations for calculating the stiffnesses from target strengths

Force component	Calculation of Stiffness
Axial (Tension)	$A_{tar} = \dfrac{P}{\text{Target Ratio} \times 0.7 \times F_y}$
Axial (Compression)	$A_{tar} = \dfrac{P}{\text{Target Ratio} \times 0.7 \times F_y}$
Shear	$A_{v,tar} = \dfrac{V}{\text{Target Ratio} \times 0.7 \times F_y}$
Moment	$S_{tar} = \dfrac{M}{\text{Target Ratio} \times 0.7 \times F_y}$

5) Search for the most effective cross section considering the target stiffness and depth limitations for each component

The ratio of the calculated target stiffness to the actual stiffness is calculated, and is used to determine whether the section is adequate or not ('OK' or 'NG', respectively). Based on this result, the most effective section is chosen from the sections deemed 'OK' from the user-specified section list. Moreover, the height and depth ranges set in *Design Setting* will be incorporated in the final cross section, and thus the most economic and efficient section will be chosen.

Table 2.6.3 Ratios to check design adequacy for each force component

Force component	Ratio for checking design adequacy
Axial (Tension)	$Ratio = \dfrac{A_{tar}}{0.6A_g}$
Axial (Compression)	$Factor = \begin{cases} \dfrac{0.877}{\lambda_c^2} & (\lambda_c > 1.5) \\ 0.658^{\lambda_c^2} & (\lambda_c \le 1.5) \end{cases}$

	$Ratio = \dfrac{A_{tar}}{Factor \times A_g}$
Shear	$Ratio = \dfrac{A_{v,tar}}{0.8A_v}$
Moment	$Ratio = \dfrac{S_{tar}}{S}$

6) Section Update/Re-Analysis

After completing the steel calculations, a re-analysis is required if the cross-section has changed (as this will change the structure's strength distribution). After updating the section, analysis should be repeated and the cross section should be checked for adequacy before outputting the final design results.

Section 1 Outline

Reinforced concrete (RC) members that are included in the analysis model are checked for adequate strength and rebar arrangements based on user-specified criteria or on the entire RC structure.

The program offers the following design codes.

Table 3.1.1 Design codes categorized per country

US	ACI318-11	Ultimate Strength Design
	ACI318-08	Ultimate Strength Design
	ACI318-05	Ultimate Strength Design
	ACI318-02	Ultimate Strength Design
	ACI318-99	Ultimate Strength Design
	ACI318-95	Ultimate Strength Design
	ACI318-89	Ultimate Strength Design
Eurocode	EN1992-1-1:2004	Limit State Design
	EN1992-1-1:1992	Limit State Design
British	BS8110-1997	Limit State Design
Korean	KCI-USD12	Ultimate Strength Design
	KCI-USD07	Ultimate Strength Design
	KCI-USD03	Ultimate Strength Design
	KCI-USD99	Ultimate Strength Design

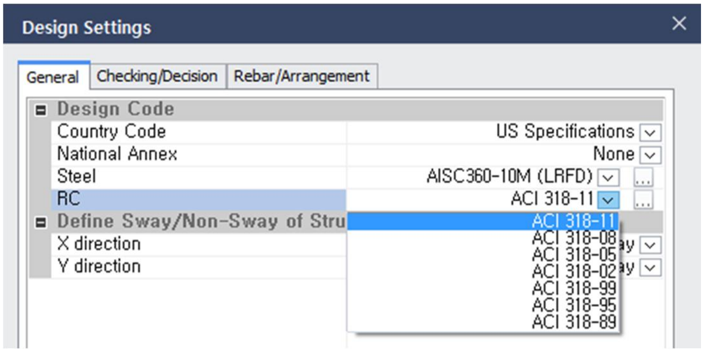
The program supplies a design summary of the calculations for the design criteria shown above. Detailed calculations for the criteria below are also provided.

► ACI318-11

► KCI-USD12

The design code may be set in *Home>Design Settings>General>Design Code*. When the country code is selected, the available design codes will be shown.

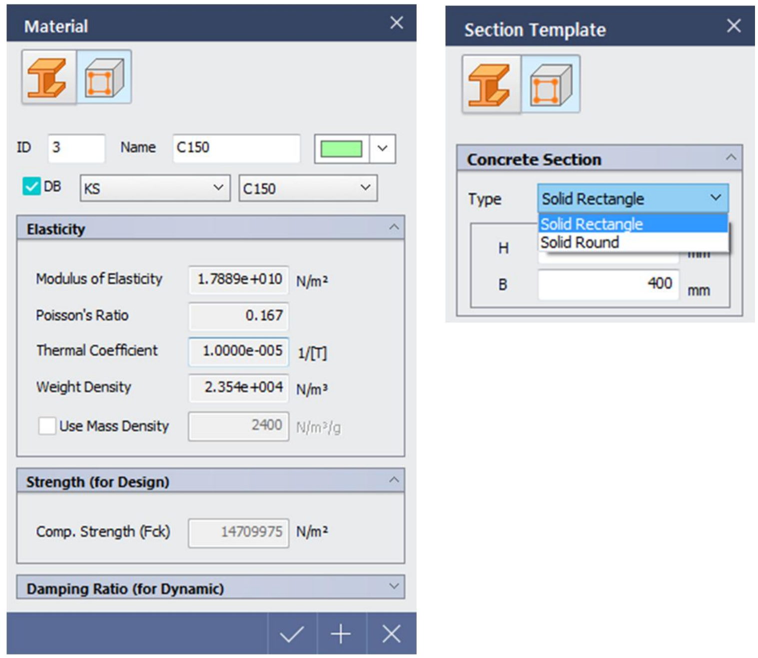
Table 3.1.1 Design Code



The program also offers a material database for concrete, and each database applies different compressive strengths to the design.

ASTM (US), BS (UK), CNS/CNS560 (TW), CSA (CA), EN/EN04 (EU),
GB/GB10/GB-Civil/JTG04/TB05 (CN), GOST-SNIP/GOST-SP (RU),
IS (IN), JIS-Civil/JIS (JP), UNI/NTC08/NTC12 (IT)
KS-Civil/KS/KS01(KCI-2003)/KS01(KCI-2007)/KS01(KCI-2012)/KS01-Civil(KCI-2003)/
KS01-Civil (KCI-2007)/KS01-Civil(KCI-2012)/KSCE-LSD12 (KR)

Figure 3.1.2 Material and section set up dialog window



The RC section may be selected to be either rectangular or round using the section template capability. Beams may only be rectangular, but column/braces may have rectangular or round sections.

Section 2

Rebar/Arrangement

This section explains how to set the rebar/arrangement information, such as rebar type, diameter, spacing, and covering thickness. This can be done in *Home>Design Settings>Rebar/Arrangement*.

Rebar Material

This program offers the following rebar material databases.

ASTM (US), BS (UK), EN/EN04 (EU, UNI (IT)
GB-Civil/GB/GB10 (CN), JIS(Civil)/JIS (JP), KS(MKS)/KS(SI) (KR)

The user may check the corresponding material standard's rebar names, diameters, maximum diameters, section area, unit weight, and strength by clicking on the […] button.

Figure 3.2.1 Rebar Material set up dialog window

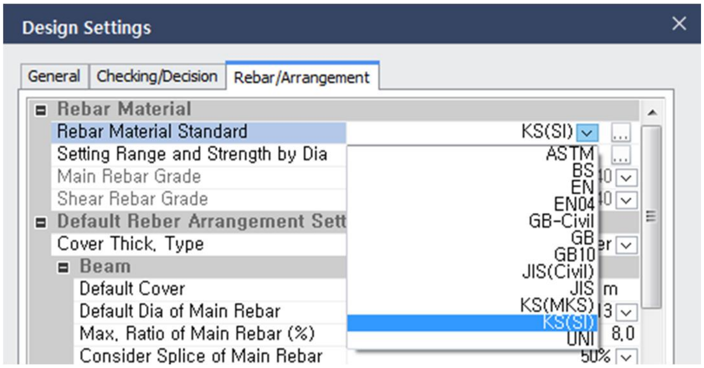
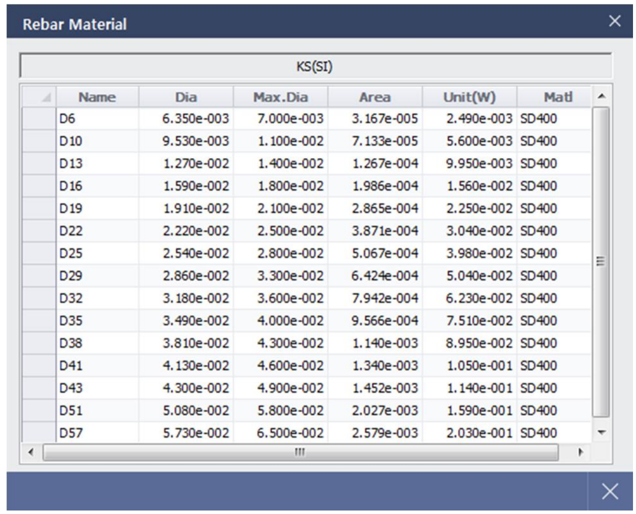


Figure 3.2.2 Dialog window for checking the rebar material standard list



Rebar Material

KS(SI)

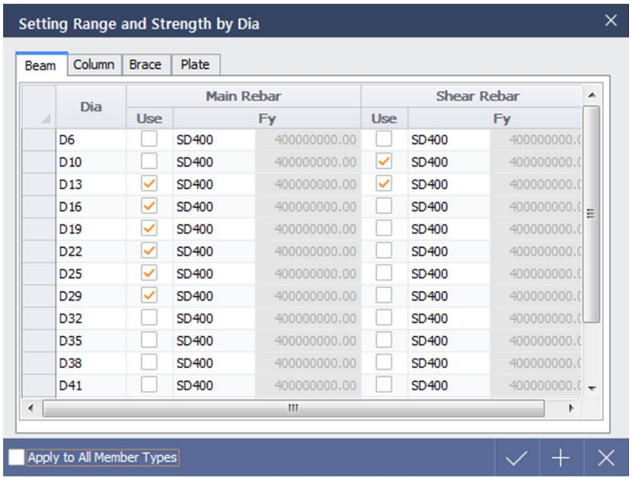
Name	Dia	Max.Dia	Area	Unit(W)	Matl
D6	6.350e-003	7.000e-003	3.167e-005	2.490e-003	SD400
D10	9.530e-003	1.100e-002	7.133e-005	5.600e-003	SD400
D13	1.270e-002	1.400e-002	1.267e-004	9.950e-003	SD400
D16	1.590e-002	1.800e-002	1.986e-004	1.560e-002	SD400
D19	1.910e-002	2.100e-002	2.865e-004	2.250e-002	SD400
D22	2.220e-002	2.500e-002	3.871e-004	3.040e-002	SD400
D25	2.540e-002	2.800e-002	5.067e-004	3.980e-002	SD400
D29	2.860e-002	3.300e-002	6.424e-004	5.040e-002	SD400
D32	3.180e-002	3.600e-002	7.942e-004	6.230e-002	SD400
D35	3.490e-002	4.000e-002	9.566e-004	7.510e-002	SD400
D38	3.810e-002	4.300e-002	1.140e-003	8.950e-002	SD400
D41	4.130e-002	4.600e-002	1.340e-003	1.050e-001	SD400
D43	4.300e-002	4.900e-002	1.452e-003	1.140e-001	SD400
D51	5.080e-002	5.800e-002	2.027e-003	1.590e-001	SD400
D57	5.730e-002	6.500e-002	2.579e-003	2.030e-001	SD400

The user may specify the rebar material strength by using either Batch Setting or Individual Setting.

► Batch Setting : Rebars are divided into main and shear reinforcements. The same strengths are applied regardless of the diameter.

► Individual Setting : Different strengths are applied depending on the rebar diameter. The user may specify the member type (beam, column, brace, plate) and main/shear reinforcement by clicking on the […] button and then specify the strength depending on the diameter.

Figure 3.2.3 Dialog window for setting rebar material strengths


Setting Range and Strength by Dia
Beam
Column
Brace
Plate
Dia
Main Rebar
Shear Rebar
Use
Fy
D6
D10
D13
D16
D19
D22
D25
D29
D32
D35
D38
D41
SD400
Apply to All Member Types

Setting Rebar Default Values

► Cover thickness

Cover thickness decides the starting location for the rebar. The cover thickness may be defined following one of the two processes below.

- Clear Cover: The thickness extends from the concrete edge to the outermost rebar surface
- Con’c Edge~Rebar Center: The thickness extends from the concrete edge to the rebar center

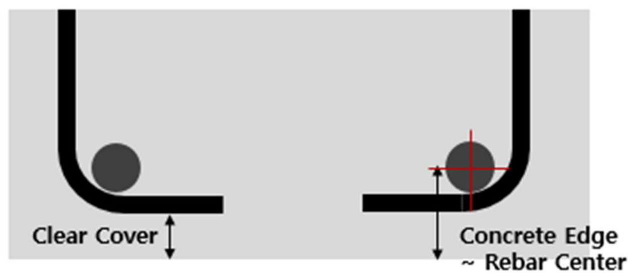


Figure 3.2.4 Cover thickness set up procedure

When calculating the strength of an RC member, steel location is an important parameter. If “Clear Cover” is selected, the first rebar center’s location is calculated using the cover thickness, the shear rebar diameter, and half of the main rebar diameter.

$$d_c = ClearCover + Dia_{shear} + 0.5Dia_{main} \tag{3.2.1}$$

► Basic information regarding rebar calculations for each member type

This section explains the setting of default values for important rebar parameters such as center cover thickness, main reinforcement diameter, largest steel ratio, main reinforcement coupling method, shear reinforcement diameter, etc.

The concrete cover thickness and the steel diameter's default values are considered when first creating the rebar arrangement, and the main reinforcement coupling method is used when calculating the possible number of rebars. For example, the calculation for possible number of rebars for a single layer due to clear cover limits is shown below.

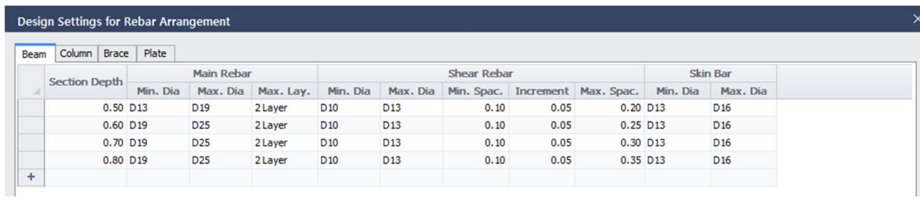
Table 3.2.1 Calculation method for possible number of rebars depending on the

Coupling Method		Calculation for possible number of rebars N
Neglected	N =	(Beam depth – 2X center cover thickness + clear cover rule + main reinforcement diameter) / Steel clear cover rule
50%	N =	(Beam depth – 2X center cover thickness + clear cover rule + main reinforcement diameter) / 0.5X main reinforcement diameter + clear cover rule
100%	N =	(Beam depth – 2X center cover thickness + clear cover rule + main reinforcement diameter) / 1.0X main reinforcement diameter + clear cover rule

► Defining the reinforcement based on the member type

This section explains the process for defining the steel's maximum/minimum diameters and maximum/minimum spacing depending on the member type and section measurements.

For beams, the section height is used as the basis to determine the main reinforcement which will then govern the maximum/minimum diameter for the main reinforcement and the greatest number of rebars. The shear steel reinforcement will determine the maximum/minimum diameter, maximum/minimum spacing, and the spacing increment. The outer steel determines the minimum/maximum diameter. The main steel starts with the minimum diameter, whereas the shear steel starts at the minimum diameter and minimum spacing. Then, the program analyzes the required amount of steel for the expected demand and finds the rebar arrangement that best satisfies spacing requirements and steel ratio requirements.

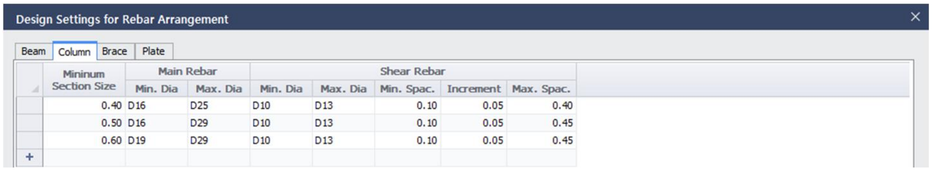


Section Depth	Main Rebar			Shear Rebar					Skin Bar	
	Min. Dia	Max. Dia	Max. Lay.	Min. Dia	Max. Dia	Min. Spac.	Increment	Max. Spac.	Min. Dia	Max. Dia
0.50	D13	D19	2 Layer	D10	D13	0.10	0.05	0.20	D13	D16
0.60	D19	D25	2 Layer	D10	D13	0.10	0.05	0.25	D13	D16
0.70	D19	D25	2 Layer	D10	D13	0.10	0.05	0.30	D13	D16
0.80	D19	D25	2 Layer	D10	D13	0.10	0.05	0.35	D13	D16

Figure 3.2.5 Shear rebar design settings dialog window

For columns and braces, the section measurements' minimum values are used as the basis and the main reinforcement decides the minimum and maximum diameters. The shear reinforcement determines the maximum/minimum diameter, maximum/minimum spacing, and spacing increment. The main reinforcement begins at the minimum diameter, and the shear reinforcement begins at the minimum diameter and minimum spacing. The program then analyzes the required amount of steel for the expected demand and finds the steel rebar arrangement that best satisfies the spacing and steel ratio requirements.

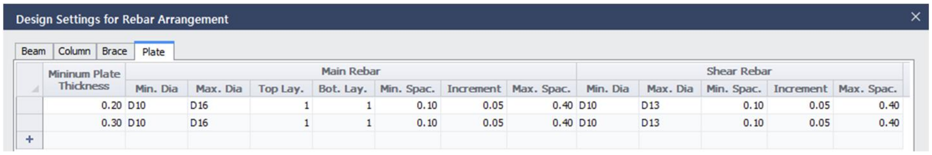
Figure 3.2.6 Column rebar design settings dialog window



Minimum Section Size	Main Rebar Min. Dia	Main Rebar Max. Dia	Shear Rebar Min. Dia	Shear Rebar Max. Dia	Shear Rebar Min. Spac.	Shear Rebar Increment	Shear Rebar Max. Spac.
0.40	D16	D25	D10	D13	0.10	0.05	0.40
0.50	D16	D29	D10	D13	0.10	0.05	0.45
0.60	D19	D29	D10	D13	0.10	0.05	0.45

For plate members, the minimum plate thickness is used as the basis. The main reinforcement dictates the maximum/minimum diameter, upper/lower reinforcement numbers, maximum/minimum spacing, and the spacing increment. The shear reinforcement dictates the maximum/minimum diameter, maximum/minimum spacing, and the spacing increment.

Figure 3.2.7 Plate rebar design settings dialog window



Minimum Plate Thickness	Main Rebar Min. Dia	Main Rebar Max. Dia	Main Rebar Top Lay.	Main Rebar Bot. Lay.	Main Rebar Min. Spac.	Main Rebar Increment	Main Rebar Max. Spac.	Shear Rebar Min. Dia	Shear Rebar Max. Dia	Shear Rebar Min. Spac.	Shear Rebar Increment	Shear Rebar Max. Spac.
0.20	D10	D16	1	1	0.10	0.05	0.40	D10	D13	0.10	0.05	0.40
0.30	D10	D16	1	1	0.10	0.05	0.40	D10	D13	0.10	0.05	0.40

The shear reinforcement makes a list of combinations of the diameter and leg number, and proceeds with the rebar creation in a sequential manner. The diameter list is defined from the minimum diameter to the maximum diameter, and the leg list is defined from two to the maximum number of legs. The maximum number of legs is decided by taking into account the number of shear reinforcement rebars and the clear cover restrictions, and chooses the maximum possible number of legs. The number of legs that takes into account the steel clear cover restriction is calculated as follows.

$$Leg_{space} = \frac{B - 2d_{c,side}}{s_{\max,trans} + d_{shearbar}} \tag{3.2.2}$$

Here,

$d_{c,side}$: Side cover thickness from the concrete edge to the center of steel rebar

$s_{\max,trans}$: Maximum steel spacing for the lateral direction (industry standard value)

$d_{shearbar}$: Diameter of shear rebar

For example, when the shear reinforcement steel diameter range is between D10-D16 and the maximum number of legs is 4, the following list is created and computed in sequential order for shear reinforcement calculations.

Table 3.2.2 Sample list of the shear reinforcement creation list when the maximum number of legs is 4

	D10	D13	D16
2 Leg	① D10 X 2 leg	② D13 X 2 leg	③ D16 X 2 leg
3 Leg	④ D10 X 3 leg	⑤ D13 X 3 leg	⑥ D16 X 3 leg
4 Leg	⑦ D10 X 4 leg	⑧ D13 X 4 leg	⑨ D16 X 4 leg

Section 3

Common Design Considerations

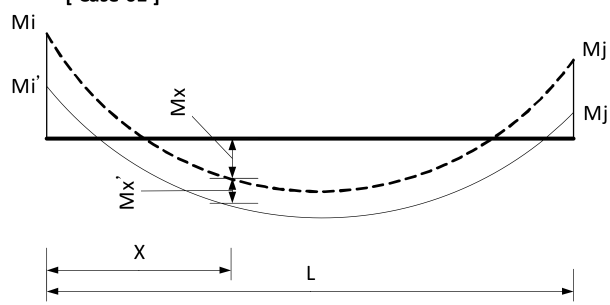
Moment Redistribution Factor [Applicable Member: Beams]

Typically, for statically indeterminate RC structures, a single section failure does not bring structural collapse and there is a significant difference in demand required to bring about the first failure and total collapse. Therefore, simply because an indeterminate beam has reached its ultimate moment does not mean immediate failure. Before reaching the state of failure, the load will increase and create a plastic hinge. This will then affect the moment distribution, and the phenomenon is called the redistribution of moment. That is to say, the state at which failure occurs, the section has plastic resistance. In parts of the member where rotation is allowed, or where the plastic hinge has formed, moment does not change. The moment will instead increase where there is low strength, and this is called moment redistribution.

The moment redistribution factor aims to reflect such phenomena in RC beams, and the design forces that incorporates the moment redistribution are is calculated as follows.

Figure 3.3.1 Design force calculation Case 1 when incorporating the moment redistribution factor

[Case 01]



Put β = Redistribution Factor

- Modified Moment At I-end : Mi' = |Mi| (1-β)
- Modified Moment At J-end : Mj' = |Mj|(1-β)
- Modified Moment at location X :
  Mx' = Mi' + X/L *(Mj'-Mi')
- Redistribution Moment at location X :
  Mxre = Mx + Mx'

[Case 02]

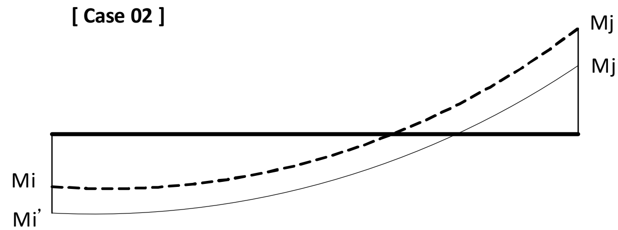


Figure 3.3.2 Case 2 for calculating the design forces while incorporating the moment redistribution factor

Put β = Redistribution Factor
Check Mi = (+) Positive, Mj = (-) Negative.

- Modified Moment At I-end : Mi' = |Mj| (1-β)
- Modified Moment At J-end : Mj' = |Mj| (1-β)
- Modified Moment at location X :
  Mx' = Mi' + X/L *(Mj'-Mi')
- Redistribution Moment at location X :
  Mxre = Mx + Mx'

The moment redistribution factors may be set for either the entire model or for individual members. Factors set for individual members will override the model-wide values if both have been defined.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

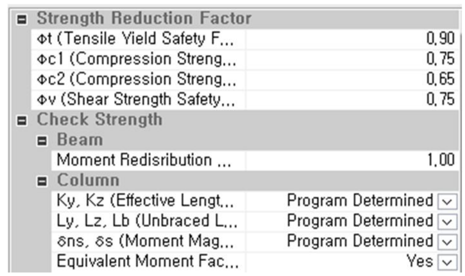


Figure 3.3.3 Dialog window showing how to set the moment redistribution factor for the entire model

Analysis & Design>Analysis & Design>Member Parameters>Moment Redistribution Factor

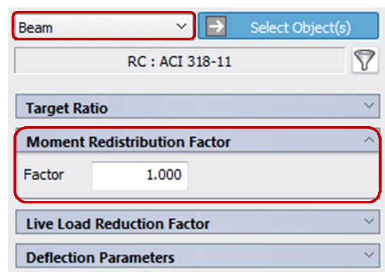


Figure 3.3.4 Dialog window showing how to set the moment redistribution factor for a specific member

Section 4 Design Considerations-ACI318-11

Strength Reduction Factor

Tension-controlled section strength reduction factors, compression-controlled section (hooped reinforcement, etc) strength reduction factors, and shear strength reduction factors can be set. The default values are the industry standard, but the user may specify alternative values. A single set of strength reduction factors is applied to the entire model.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

Strength Reduction Factor	
Φt (Tensile Yield Safety F...	0.90
Φc1 (Compression Streng...	0.75
Φc2 (Compression Streng...	0.65
Φv (Shear Strength Safety...	0.75

Figure 3.4.1 Dialog window for setting the strength reduction factors

Moment Magnifiers δns, δs [Applicable members: columns, braces]

Moment magnifiers are used to approximate second order analysis results using first order analysis results, to incorporate second order effects without conducting a full analysis. Moment magnifiers are automatically computed for laterally braced and unbraced members. If the member's effective length factor k is less than 1.0, then the member is considered to be laterally braced.

δ_{ns} is a moment magnifier that aims to incorporate P-δ effects that occur due to strains developing in the structure, and is supported by automatic computation in this program. It is calculated as follows:

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75P_c}} \tag{3.4.1}$$

Here,

$$C_m = 0.6 + 0.4\frac{M_1}{M_2}$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

$$EI = \frac{0.2E_c I_g + E_s I_{se}}{1+\beta_{ns}}$$

δ_s is a moment magnifier that aims to incorporate P-Δ effects that occur due to local displacements, and is calculated as follows:

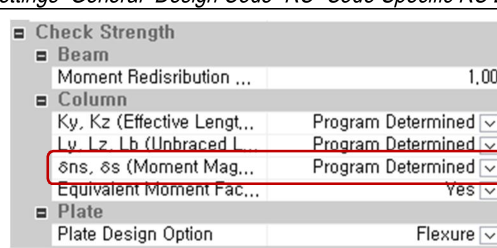
$$\delta_s = \frac{1}{1 - \dfrac{\sum P_u}{0.75\sum P_c}} \geq 1 \qquad (3.4.2)$$

However, δs is not automatically computed in this program. Unless the user specifies a value, 1.0is applied as the default value.

Moment magnifiers may be set to be automatically determined for the entire model, and individual members may be set to take on the automatically computed values or alternative, user-specified values. The program overrides the automatically determined values with any user-specified values for members that have such alternate values defined.

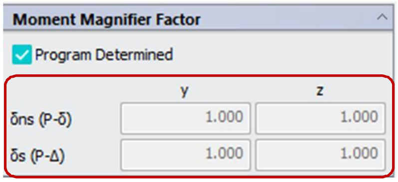
Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

Figure 3.4.2 Dialog window for setting automatic computation of moment magnifiers



Analysis & Design>Analysis & Design>Member Parameters>Moment Magnifier Factor

Figure 3.4.3 Dialog window for setting the moment magnifier factors for individual members



Seismic Design Shear Calculations [Applicable members: beam, column]

When considering seismic design of structures, the design must account for additional shear forces for seismic considerations. The program uses the following methods to calculate the seismic design shear.

Table 3.4.1 Seismic Design Shear calculation methods

Calculation Method	Shear Calculation
Max (V_{e1}, V_{e2})	The maximum of the two values V_{e1}, V_{e2} (which incorporate the additional shear factors a_1, a_2)
Min (V_{e1}, V_{e2})	The minimum of the two values V_{e1}, V_{e2} (which incorporate the additional shear factors a_1, a_2) is used. However, if the seismic design shear is smaller than the analyzed shear forces, then the analyzed shear forces are used instead.
V_{e1}	Shear is added by using the weak shear-strong bending principle. Shear strength is calculated by applying the additional shear factors a_1. a_1 is specified by the user, and can use the industry standard as the default value. If the earthquake resisting system is SMF, $$V_{e1} = V_g + a_1 \frac{\sum M_{pr}}{l}$$ M_{pr} : calculated expected bending strength assuming tensile yield strength of $1.25f_y$ and strength reduction factor of 1.0 If the earthquake resisting system is IMF, $$V_{e1} = V_g + a_1 \frac{\sum M_n}{l}$$

V_{e2}	The shear force due to earthquake loading is increased. Shear strength is calculated by using the additional shear factor a2. a2 is specified by the user, and can use the industry standard as the default value. $$V_{e2} = V_g + a_2 V_{eq}$$

Seismic design criteria are applied to the entire model.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

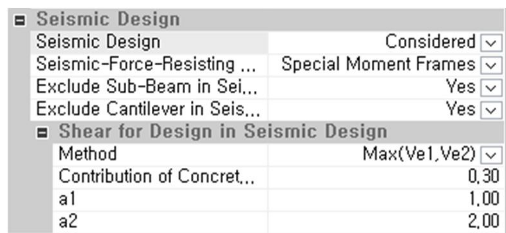


Figure 3.4.4 Dialog window setting seismic design criteria to the entire model

Section 5

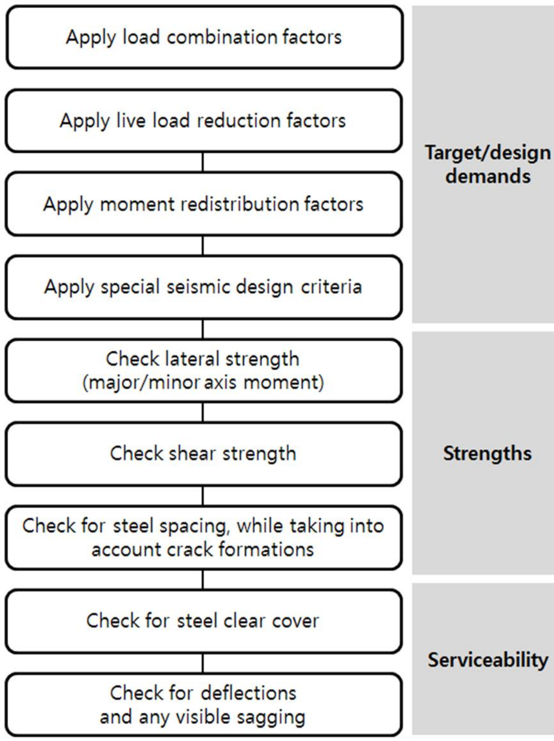
Member Examination Procedure (Beams)-ACI318-11

The member verification procedure for RC beams following the ACI318-11 design code is explained in this section.

The design strength is calculated using the analyzed strengths, load combinations, and design strength modification factors. Industry standard strengths are computed to verify the moment, shear, and main reinforcement spacing, to ensure consistency. For serviceability, immediate deflections, long term deflections, and outer rebar spacing are checked.

Calculation of Design Demands

The design strengths are calculated by applying load combinations, live load reduction factors, moment redistribution factors, and seismic design criteria to the analyzed strengths.


Apply load combination factors
Apply live load reduction factors
Apply moment redistribution factors
Apply special seismic design criteria
Check lateral strength (major/minor axis moment)
Check shear strength
Check for steel spacing, while taking into account crack formations
Check for steel clear cover
Check for deflections and any visible sagging
Target/design demands
Strengths
Serviceability

1) Application of load combinations

The design forces are calculated, factoring in the member check points, load types, and load combination factors.

2) Application of live load reduction factors

As explained in the section Design Factors>Live Load Reduction factors, components that are subject to live loads will have design forces that incorporate the live load reduction factor.

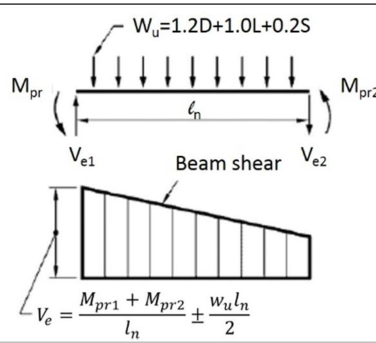
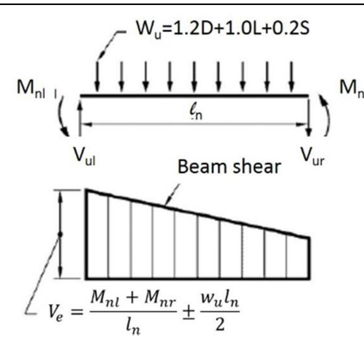
3) Application of Moment Redistribution Factors

Between both ends of a beam in which minor axis moments are present, at least one end is selected for application of a moment redistribution factor less than 1.0 (as explained in Design Factors>Moment Redistribution Factors). Then, the design moment incorporating the moment redistribution factor is calculated.

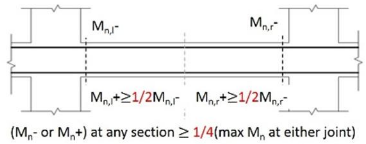
4) Application of Seismic Design Criteria

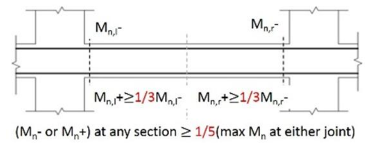
When seismic design criteria are used, the design shear and moment are calculated differently, depending on the earthquake resisting system.

Table 3.5.1 Design shear and moment calculations depending on the earthquake resisting system

	SMF	IMF
Shear		

Moment





As for design shear, one of the two methods is applied—either the strong shear-weak bending principle method or additional shear method—depending on the user's preference. If the earthquake resisting system is a special moment frame, then the design shear as per the strong shear-weak bending principle may be calculated as shown below.

Table 3.5.2 Design shear calculations for a special moment frame, using the strong shear-weak bending principle

$V_{e1,cw,1} = V_g + a_1 \dfrac{M_{pr,i(+)} + M_{pr,j(-)}}{l_n}$	$V_{e1,ccw,1} = V_g + a_1 \dfrac{M_{pr,i(-)} + M_{pr,j(+)}}{l_n}$
$V_{e1,cw,2} = V_g - a_1 \dfrac{M_{pr,i(+)} + M_{pr,j(-)}}{l_n}$	$V_{e1,ccw,2} = V_g - a_1 \dfrac{M_{pr,i(-)} + M_{pr,j(+)}}{l_n}$
$V_{e1,cw} = \max\left[\lvert V_{e1,cw,1}\rvert, \lvert V_{e1,cw,2}\rvert\right]$	$V_{e1,ccw} = \max\left[\lvert V_{e1,ccw,1}\rvert, \lvert V_{e1,ccw,2}\rvert\right]$
$V_{e1} = \max\left[V_{e1,cw}, V_{e1,ccw}\right]$	

The design shear incorporating additional shear due to earthquake loads can be calculated as shown below.

$$V_{e2} = V_g + a_2 V_{eq} \tag{3.5.1}$$

Calculation of Design Strengths

The design strength of each member is based on load combinations and must be greater than the calculated required strength.

$$R_u \le \phi R_n \tag{3.5.2}$$

R_u : Required strength

R_n : Nominal strength

Φ : Strength reduction factor

1) Flexural/Bending strength

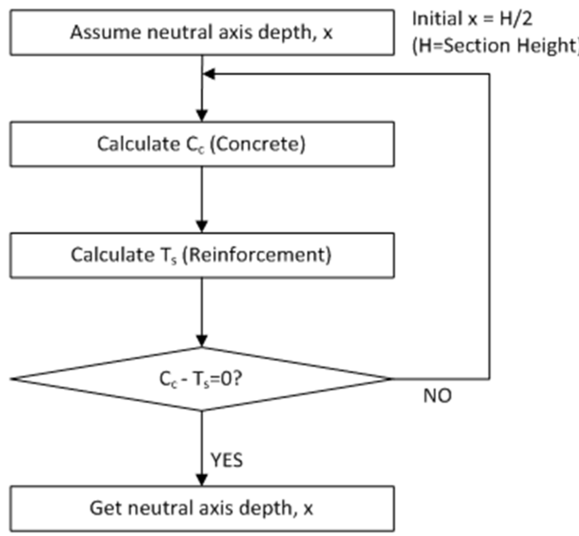
Major and minor bending strengths must be greater than the moment demands expected in the structure. The main reinforcement ratio should satisfy the maximum/minimum steel ratio limits.

$$M_{u(+)} \le \phi M_{n(+)} \tag{3.5.3}$$

$$M_{u(-)} \le \phi M_{n(-)} \tag{3.5.4}$$

$$\rho_{\min} \le \rho \le \rho_{\max} \tag{3.5.5}$$

Neutral axis location is the most important number in calculating the flexural strength. Iterative methods are used to find the solution that satisfies force equilibrium.



Using c as the neutral axis, the compressive force taken on by the concrete C_c is as follows:

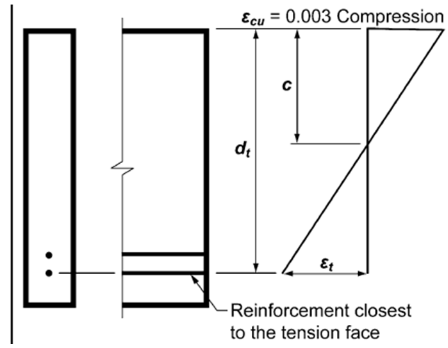
$$C_c = 0.85 f_c ab \tag{3.5.6}$$

Here,

$$a = \beta_1 c$$

$$\beta_1 = \begin{cases} 0.85 & (f_c \le 4000\text{psi}) \\ \max\left[1.05 - 0.05\dfrac{f_c}{1000},\ 0.65\right] & (f_c > 4000\text{psi}) \end{cases}$$

Figure 3.5.1 Neutral axis location for a high strength RC block



C_s is the force taken on by the compressive steel and T_s is the force taken on by the tensile steel:

$$C_s = \sum A_{sci}\left(f_{si} - 0.85 f_c\right) \tag{3.5.7}$$

$$T_s = \sum A_{sti} f_{si} \tag{3.5.8}$$

Here,

$$f_{si} = \min\left[f_y, \varepsilon_{si} E_{si}\right]$$

$$\varepsilon_{si} = \begin{cases} \varepsilon_{cu} \dfrac{d_i - c}{c} & \left(c < d_i\right) \\ \varepsilon_{cu} \dfrac{c - d_i}{c} & \left(c \ge d_i\right) \end{cases}$$

This program uses the bisection method (one of the numerical analysis methods) to find the neutral axis. The principal equation of the bisection method is $C_c + C_s = T_s$. Convergence/stopping criteria are shown below.

Table 3.5.3 Neutral axis calculation methods and corresponding stopping

Stopping Criteria	Description
Convergence	$\left\lvert \dfrac{C_c + C_s}{T_s} - 1.0 \right\rvert \le 0.001 \ \ (tolerance)$

No convergence (Stop computations)	When the number of iterations is larger than 20, it is deemed that convergence will not be reached. The section is either increased or the rebar information is modified (location, number of rebars, spacing, etc).

After locating the neutral axis, the nominal flexural strength is calculated as shown below.

$$M_{ncc} = C_c(c - 0.5a) = C_c(c - 0.5\beta_1 c) \tag{3.5.9}$$

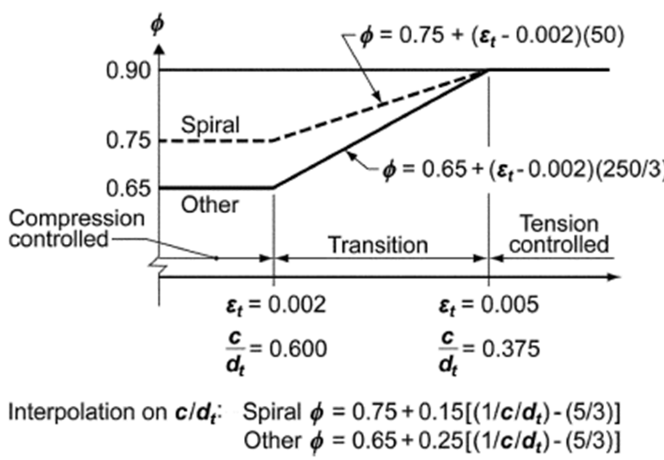
$$M_{nsc} = C_{si}(c - d_i) \tag{3.5.10}$$

$$M_{nst} = T_{si}(d_i - c) \tag{3.5.11}$$

$$M_n = M_{ncc} + M_{nsc} + M_{nst} \tag{3.5.12}$$

The design lateral strength is the product of the nominal strength and the strength reduction factor (whose calculations are shown below and depends on the outermost tensile steel strain ε_t).

Figure 3.5.2 Strength Reduction Factors



The minimum and maximum steel ratios of the main reinforcement is shown below.

Table 3.5.4 Minimum and maximum steel ratios for the main reinforcement

$\rho_{\min}$	$\rho_{\min 1} = \max\left[3.0\dfrac{\sqrt{f_c}}{f_y}, \dfrac{200}{f_y}\right]$ $\rho_{\min 2} = \dfrac{4}{3}\rho_{req}$ $\rho_{\min} = \min\left[\rho_{\min 1}, \rho_{\min 2}\right]$

$\rho_{\max}$	$\rho_{\max} = \rho_b = 0.85\beta_1 \frac{f_c}{f_y} \frac{\varepsilon_{cu}}{\varepsilon_{cu} + 0.004}$ If the earthquake resisting system is SMF and if earthquake design criteria are to be applied, $\rho_{\max} = \min[\rho_b, 0.025]$

2) Shear Strength

The design shear strength must be greater than the expected shear demands. The shear reinforcement spacing must be less than the maximum spacing set by the industry standards.

$$V_u \le \phi V_n \tag{3.5.13}$$

$$s \le s_{\max} \tag{3.5.14}$$

The design shear strength is the product of the strength reduction factor, and the sum of the shear forces taken on by the concrete and shear reinforcement.

$$\phi V_n = \phi (V_c + V_s) \tag{3.5.15}$$

The shear force taken on by the concrete is determined as follows:

$$V_c = 2\sqrt{f_c} bd \tag{3.5.16}$$

However, if the earthquake resisting system is SMF and earthquake design criteria are to be applied, then the user-specified shear contribution of concrete will be multiplied to the above value. The shear contribution of concrete may be set in *Design Settings>General>Design Code>Seismic Design>Shear for Design in Seismic Deisgn*.

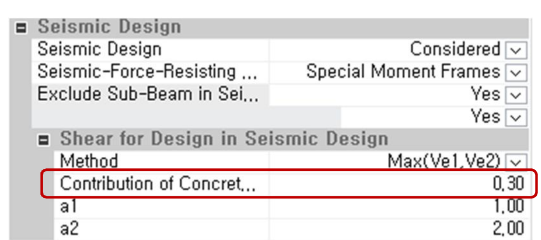


Figure 3.5.3 Dialog window showing the setup of shear contribution of concrete in . . . .

The shear force taken on by the shear reinforcement is calculated as follows:

$$V_s = \frac{nA_{sv1} f_{yt} d}{s} \tag{3.5.17}$$

Here, n : Number of legs of the shear reinforcement

The required shear reinforcement steel amount depends on the shear force, as shown below.

Figure 3.5.4 The required amount of steel depending on the shear force

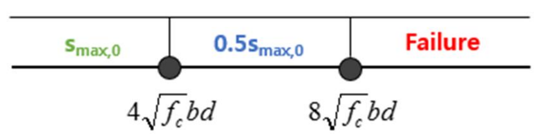
The maximum spacing limits are dependent on the seismic design criteria and whether they are being applied.

Table 3.5.5 Maximum spacing limits based on application of seismic design criteria

No seismic design criteria	$s_{\max,0} = \min\left[\frac{d}{2},\ 24\,\text{in}\right]$
Seismic design criteria applied	▸SMF $s_{\max,0} = \min\left[\frac{d}{4},\ 6\text{in},\ 6D_{main}\right]$ ▸IMF $s_{\max,0} = \min\left[\frac{d}{2},\ 12\,\text{in},\ 8D_{main},\ 24D_{stirrup}\right]$

The maximum spacing limits depending on the shear force is shown below.

Figure 3.5.5 Maximum spacing limits as a function of shear force

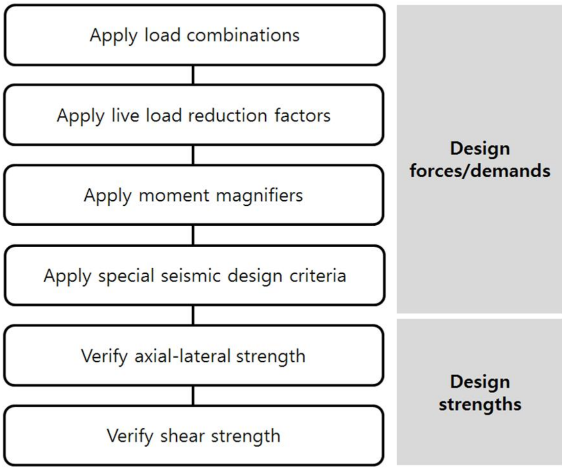


Section 6

Member Examination Procedure (Columns/Braces) ACI318-11

The member examination procedure for RC columns/braces as per the ACI318-11 design code is explained in this section.

The design strength is calculated using the analyzed strengths, load combinations, and design strength modification factors. Industry standard strengths are computed to verify the axial forces, moments, and shear forces.



Calculation of Design Demands

Design demands/forces are computed by applying load combinations, live load reduction factors, moment magnifiers, and special seismic design criteria.

1) Apply Load Combinations

The analyzed demands at the member check points, load types, and the load combination factors are incorporated in computing the design demands.

2) Applying the live load reduction factor

As explained in the section Design Factors>Live Load Reduction Factors, components that are subject to live loads will have design forces that incorporate the live load reduction factor.

3) Applying the moment magnifier

When designing columns/braces, sections are designed differently depending on the slenderness ratio. If the member is a long column, then moment magnifiers are used in calculating the design lateral moment.

The second order moment—incorporating the analytical first order moment and moment magnifiers—are calculated depending on the lateral bracing conditions, as shown below. This program decides the bracing conditions based on the member's effective buckling length factor.

Table 3.6.1 2nd order moments depending on the lateral bracing conditions

Braced	K ≤ 1.0	$M_c = \delta_{ns} M_2$
Unbraced	K > 1.0	$M_1 = M_{1ns} + \delta_s M_{1s}$ $M_2 = M_{2ns} + \delta_s M_{2s}$

Here,

M_2 = The larger of the end lateral moments of the compressive member

M_{1ns} = The end lateral moment calculated using 1st order elastic frame analysis and loads that do not cause lateral strains at the end at which M_1 is applied. This program uses moments due to dead and live loads.

M_{1s} = The end lateral moment of the compressive member, calculated using 1st order elastic frame analysis and loads that cause lateral strains at the end at which M_1 is applied. This program uses moments due to all loads except for dead and live loads.

M_{2ns} = The end lateral moment of the compressive member, calculated using 1st order elastic frame analysis and loads that do not cause lateral strains at the end at which M_2 is applied. This program uses moments due to dead and live loads.

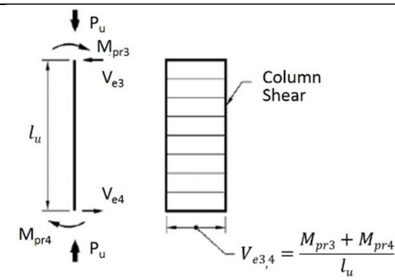
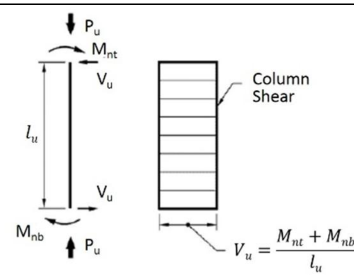
M_{2s} = The end lateral moment of the compressive member, calculated using 1st order elastic frame analysis and loads that cause lateral strains at the end at which M_2 is applied. This program uses moments due to all loads except for dead and live loads.

However, load combinations that include P-Δ analysis criteria (which are part of second order analysis) do not use moment magnifiers.

4) Applying special seismic design criteria

The design shear forces must be calculated differently depending on the resisting system, if seismic design criteria are being applied.

Table 3.6.2 Design shear force calculations for different earthquake resisting systems

	SMF	IMF
Shear		

Design shear forces may be calculated using either the strong shear-weak bending principle method or the additional shear method. The user may specify his or her preference within the program settings. If the earthquake resisting system is a special moment frame, then the design shear is calculated using the strong shear-weak bending principle.

Table 3.6.3 Design shear calculations for special moment frames, using strong shear-weak bending

$V_{e1,cw,1} = V_g + a_1 \dfrac{M_{pr,i(+)} + M_{pr,j(-)}}{l_n}$	$V_{e1,ccw,1} = V_g + a_1 \dfrac{M_{pr,i(-)} + M_{pr,j(+)}}{l_n}$
$V_{e1,cw,2} = V_g - a_1 \dfrac{M_{pr,i(+)} + M_{pr,j(-)}}{l_n}$	$V_{e1,ccw,2} = V_g - a_1 \dfrac{M_{pr,i(-)} + M_{pr,j(+)}}{l_n}$
$V_{e1,cw} = \max\left[\left\|V_{e1,cw,1}\right\|, \left\|V_{e1,cw,2}\right\|\right]$	$V_{e1,ccw} = \max\left[\left\|V_{e1,ccw,1}\right\|, \left\|V_{e1,ccw,2}\right\|\right]$
$V_{e1} = \max\left[V_{e1,cw}, V_{e1,ccw}\right]$	

The design shear force with the additional shear due to earthquake loading is calculated as follows.

$$V_{e2} = V_g + a_2 V_{eq} \tag{3.6.1}$$

Calculation of design strengths

The design strength of each member is based on load combinations and must be greater than the calculated required strength.

$$R_u \le \phi R_n \tag{3.6.2}$$

Here,

R_u : Required strength

R_n : Nominal strength

Φ : Strength reduction factor

1) Axial-lateral strength

To calculate the design strengths of members subject to both axial and lateral loading, the correlation between axial-lateral forces must be incorporated. In this program, the P-M correlations are incorporated into the computation of axial and lateral strengths. The main reinforcement ratio must satisfy the minimum and maximum steel ratio limits.

$$P_u \le \phi P_n \tag{3.6.3}$$

$$M_u \le \phi M_n \tag{3.6.4}$$

$$M_{uy} \le \phi M_{ny} \tag{3.6.5}$$

$$M_{uz} \le \phi M_{nz} \tag{3.6.6}$$

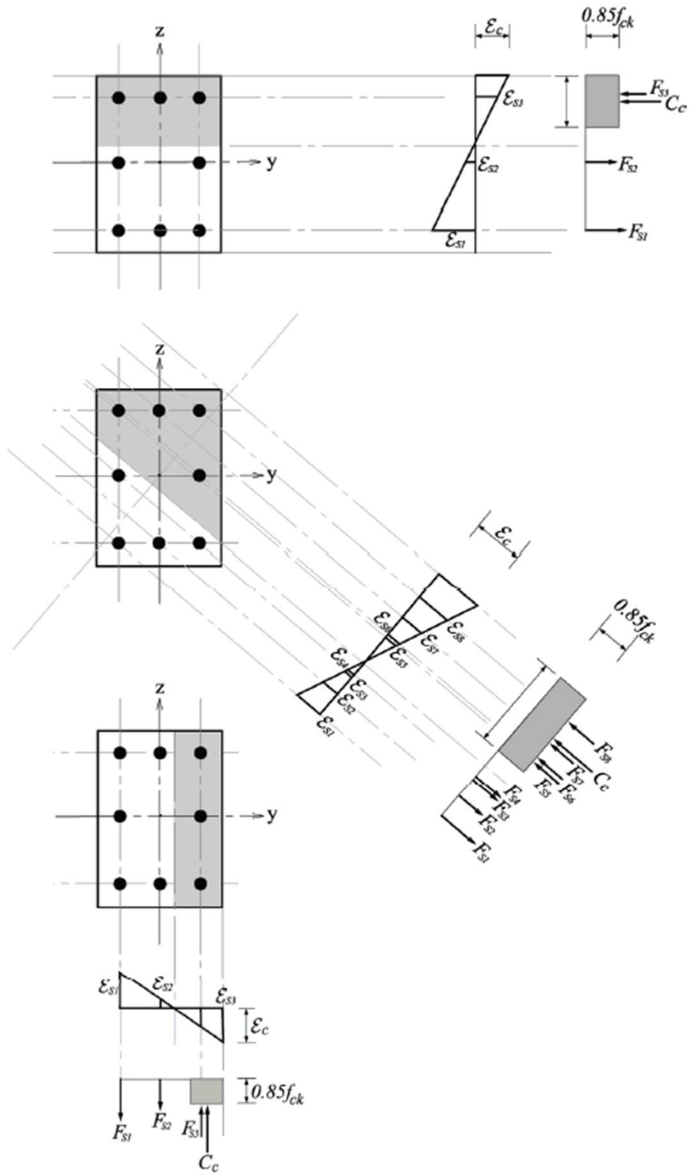
$$\rho_{\min} \le \rho \le \rho_{\max} \tag{3.6.7}$$

Compressive members subject to pure axial force (without eccentricity) have design axial strengths that are calculated as shown below.

Table 3.6.4 Design axial strengths of compressive members, depending on reinforcement type

Spiral reinforcement	$\phi P_{n,\max} = 0.85\phi\left[0.85 f_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$
Hoop reinforcement	$\phi P_{n,\max} = 0.80\phi\left[0.85 f_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$

Column/brace members subject to both axial and lateral loads must satisfy force equilibrium and strain compatibility criteria. Stress-strain relationships for biaxial P-M correlations are shown below.



The axial force and lateral force is calculated using eccentricity. Using the resulting values, the P-M correlation curve is calculated. Through the correlation curve, the design shear corresponding to the desired force may be found.

Figure 3.6.2 Uniaxial P-M correlation (nominal strength)

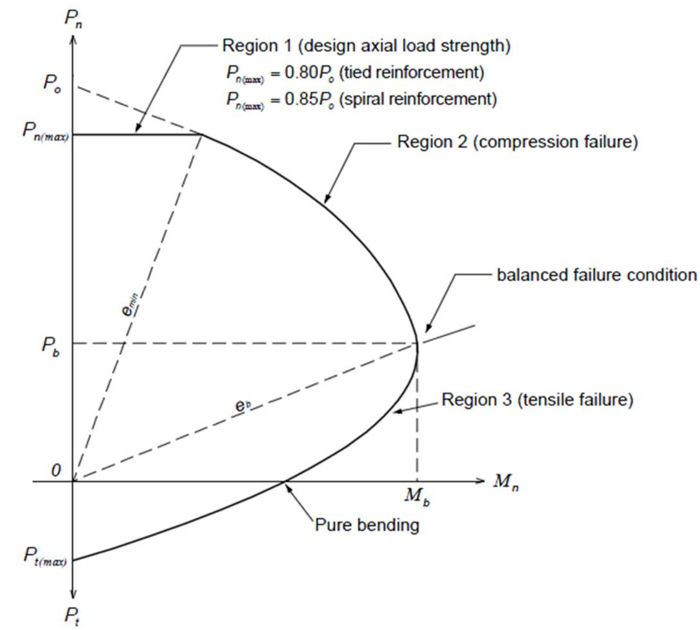


그림 3.6.3 Uniaxial P-M correlation (design strength)

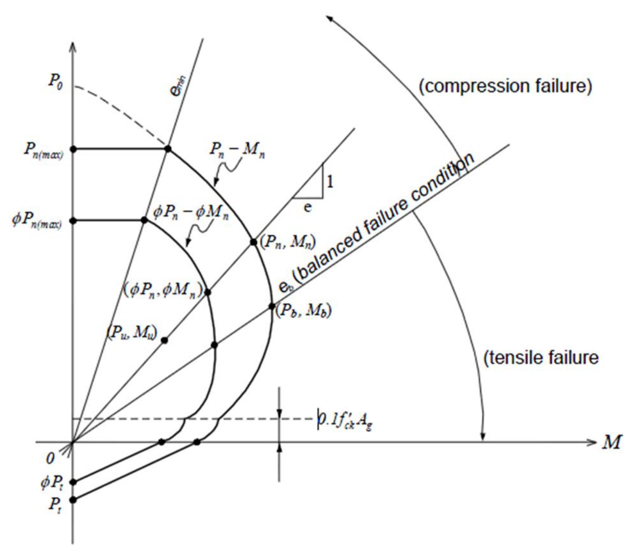


Figure 3.6.4

Biaxial P-M correlation

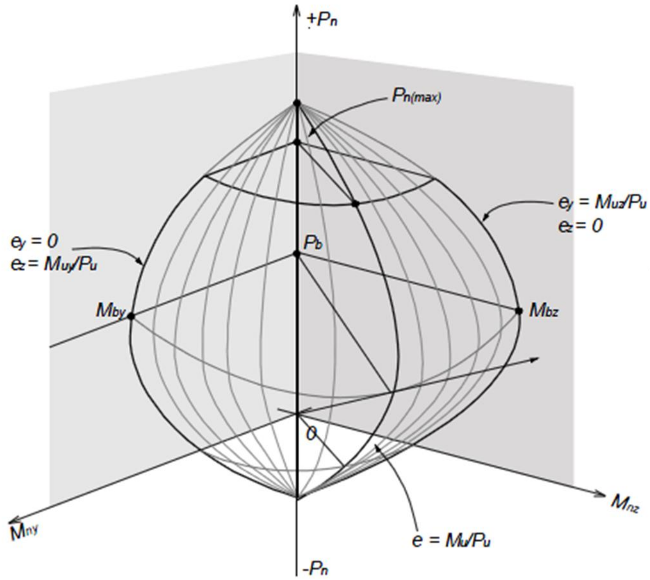
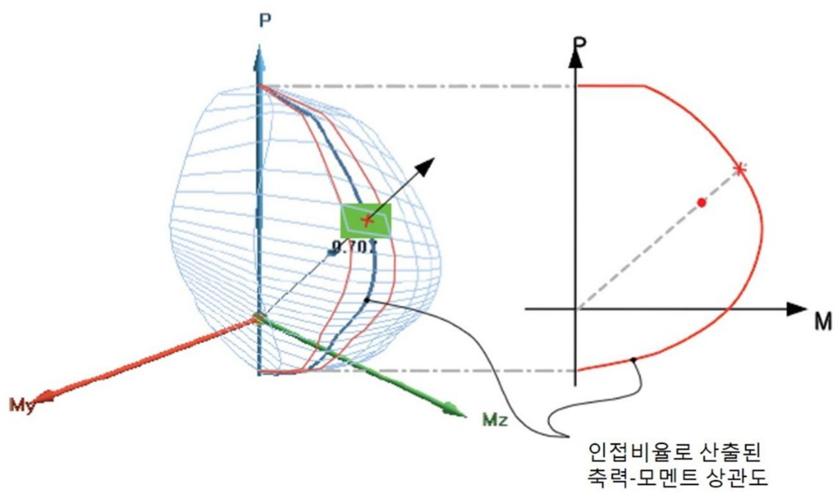


Figure 3.6.5

P-M correlation for a specific load combination



The minimum and maximum steel ratios for the main reinforcement are shown below.

Table 3.6.5 Minimum and maximum steel ratios for the main reinforcement

$\rho_{\min}$	0.01
$\rho_{\max}$	User-specified

2) Shear strength

The design shear strength must be greater than the expected design shear demands. The main reinforcement spacing must be smaller than the maximum spacing limits set by industry standards.

$$V_{uy} \le \phi V_{ny} \tag{3.6.8}$$

$$V_{uz} \le \phi V_{nz} \tag{3.6.9}$$

$$s \le s_{\max} \tag{3.6.10}$$

The design shear strength is the product of the strength reduction factor and the sum of the shear force taken on by the concrete and shear reinforcement.

$$\phi V_n = \phi(V_c + V_s) \tag{3.6.11}$$

The shear force taken on by the concrete is a function of the axial force, as shown below.

Table 3.6.6 Shear forces taken on by the concrete, as a function of the axial force

P = 0	$V_c = 2\sqrt{f_c}bd$
Tensile force	$V_c = 2\left(1+\frac{N_u}{2000A_g}\right)\sqrt{f_c}bd$
Compressive force	$V_c = 2\left(1+\frac{N_u}{500A_g}\right)\sqrt{f_c}bd$

However, if the earthquake resisting system is SMF and seismic design criteria are to be applied, then the user-specified concrete shear contribution actors are to be multiplied.

The shear strength taken on by the shear reinforcement steel is calculated as shown below.

$$V_s = \frac{nA_{sv1} f_{yt} d}{s} \tag{3.6.12}$$

Here, n : number of legs of the main reinforcement

The required shear reinforcement amounts are dependent on the shear force, as shown below:

Figure 3.6.6 Required shear steel reinforcement amounts as a function of shear force

Maximum spacing limits for shear reinforcement are calculated differently depending on the application of seismic design criteria, rebar arrangement (end/interior portions), type of shear reinforcement (hoop/spiral). The maximum spacing limits for rectangular sections are shown below.

Table 3.6.7 Maximum spacing limits for rectangular sections as a function of seismic design criteria

No seismic design criteria	$s_{\max,0} = \min[16\mathrm{D}_{\mathrm{main}},\ 48D_{shear},\ H,\ B]$
Application of seismic design criteria (Ends)	▸ SMF $s_{\max,01} = \min\left[\frac{d}{4},\ 6\mathrm{in},\ 6D_{main}\right]$ $s_{\max,02} = 4 + \max\left[\frac{14 - h_x}{3}, 0\right]$ $s_{\max,03} = \dfrac{A_{sh}}{0.3h_c \dfrac{f_c}{f_{ys}} \left(\dfrac{A_g}{A_c} - 1\right)}$ $s_{\max,04} = \dfrac{A_{sh}}{0.09h_c \dfrac{f_c}{f_{ys}}}$ $s_{\max,0} = \min[s_{\max,01},\ s_{\max,02},\ s_{\max,03}, s_{\max,04}]$ ▸ IMF

	$s_{\max,0} = \min\left[\frac{H}{2}, \frac{B}{2}, 8D_{main}, 24D_{stirrup}, 12\text{in}\right]$
Application of seismic design criteria (Interior)	▸ SMF $s_{\max,01} = \min[16\mathrm{D}_{\mathrm{main}}, 48D_{shear}, H, B]$ $s_{\max,02} = \min[6\mathrm{D}_{\mathrm{main}}, 6\mathrm{in}]$ $s_{\max,0} = \min[s_{\max,01}, s_{\max,02}]$ ▸ IMF $s_{\max,0} = \min[16\mathrm{D}_{\mathrm{main}}, 48D_{shear}, H, B]$

Maximum spacing limits for circular sections are as follows.

Table 3.6.8 Maximum spacing limits for circular sections as a function of seismic design

No seismic design criteria	$s_{\max,0} = \min\left[16\mathrm{D}_{\mathrm{main}},\ 48D_{shear},\ H,\ B\right]$
Application of seismic design criteria (End)	▸ SMF (Hoop Reinforcement) $s_{\max,01} = \min\left[\dfrac{d}{4},\ 6\mathrm{in},\ 6D_{main}\right]$ $s_{\max,03} = \dfrac{\pi D_{shear}^{2}}{0.12H\dfrac{f_c}{f_{ys}}}$ $s_{\max,02} = 4+\max\left[\dfrac{14-h_x}{3},0\right]$ $s_{\max,0} = \min\left[s_{\max,01},\ s_{\max,02},\ s_{\max,03}\right]$ ▸ SMF (Spiral Reinforcement) $\rho_s = \max\left[0.45\left(\dfrac{A_g}{A_c}-1\right)\dfrac{f_c}{f_{ys}},0.12\dfrac{f_c}{f_{ys}}\right]$ $s_{\max,0} = \min\left[\max\left[\dfrac{\pi D_{shear}^{2}}{\rho_s d_c},D_{shear}+1\mathrm{in}\right],D_{shear}+3\mathrm{in}\right]$ ▸ IMF $s_{\max,0} = \min\left[\dfrac{H}{2},8D_{main},\ 24D_{stirrup},\ 12\,\mathrm{in}\right]$
Application of seismic design criteria (Interior)	▸ SMF $\rho_s = 0.12\dfrac{f_c}{f_{ys}}$ $s_{\max,0} = \min\left[\max\left[\dfrac{\pi D_{shear}^{2}}{\rho_s d_c},D_{shear}+1\mathrm{in}\right],D_{shear}+3\mathrm{in}\right]$ ▸ IMF $s_{\max,0} = \min\left[16\mathrm{D}_{\mathrm{main}},\ 48D_{shear},\ H,\ B\right]$

Maximum spacing limits are a function of the shear force, as shown below.

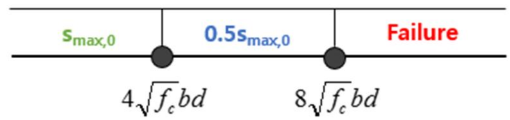


Figure 3.6.7 Maximum spacing limits as a function of shear force

Section 7

Design Parameters EN1992-1-1:2004

Partial Factors

This section explains how to set partial factors ($\gamma_{c,Fundamental}$, $\gamma_{c,Accidental}$) for long-term and short-term loading on concrete and partial factors ($\gamma_{s,Fundamental}$, $\gamma_{s,Accidental}$) for long-term and short-term loading on steel, as well as the long-term loading effective factor (α_{cc}). Industry standards are programmed as default values, but the user may modify these values. Partial factors are applied to the entire model.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

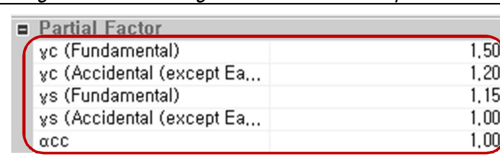


Figure 3.7.1 Dialog window for setting the partial factor for the entire model

Slenderness Limitation [Applicable Members: Columns, Braces]

If the member's slenderness ratio λ is smaller than the slenderness limitation λ_{lim}, then the member's second order effects may be ignored. The slenderness limitation is calculated as shown below.

$$\lambda_{\lim} = \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}} \qquad (3.7.1)$$

Here,

$A = \dfrac{1}{1+0.2\phi_{ef}}$: this is specified by the user in this program.

$B = \sqrt{1+2\omega}$: this is specified by the user in this program.

$C = 1.7 - r_m$

ϕ_{ef} = effective creep ratio.

$$\omega = \frac{A_s f_{yd}}{A_c f_{cd}}$$

$$n = \frac{N_{Ed}}{A_c f_{cd}}$$

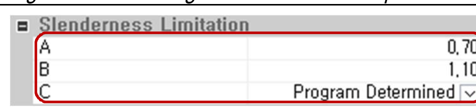
$$r_m = \frac{M_{01}}{M_{02}}$$

M_{01}, M_{02} = 1st order end moments ($|M_{02}| \geq |M_{01}|$)

To calculate the slenderness limitation, parameters A, B, C are input and used for the entire model.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

Figure 3.7.2 Dialog window for setting the slenderness limitation for the entire model



Seismic Design Criteria

In this program, the basic value of the behavior factor q_o is used to calculate the curvature ductility factor, and is calculated as shown below. The user may specify α_u/α_1 and q_o directly.

Table 3.7.1 Behavior factor q0 as a function of the system type

System type	DCM	DCH
Frame system, Dual system, Coupled wall system	$3.0\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$
Uncoupled wall system	3.0	$4.0\alpha_u/\alpha_1$
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

The curvature ductility factor μ_φ is calculated as follows:

$$\mu_\phi = 2q_0 - 1 \ \left(T_1 \geq T_C\right) \tag{3.7.2}$$

$$\mu_\phi = 1 + 2(q_0 - 1)\frac{T_C}{T_1} \; (T_1 < T_C) \tag{3.7.3}$$

γ_{Rd}, which is used to calculate the end moment $M_{i,d}$ for seismic design shear, takes on different values for beams and columns, as shown below.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

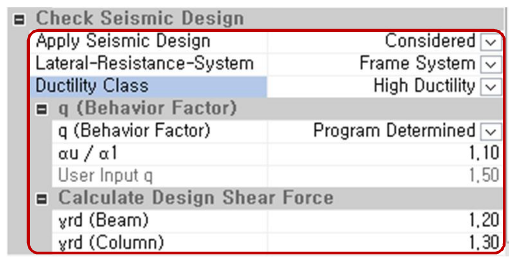


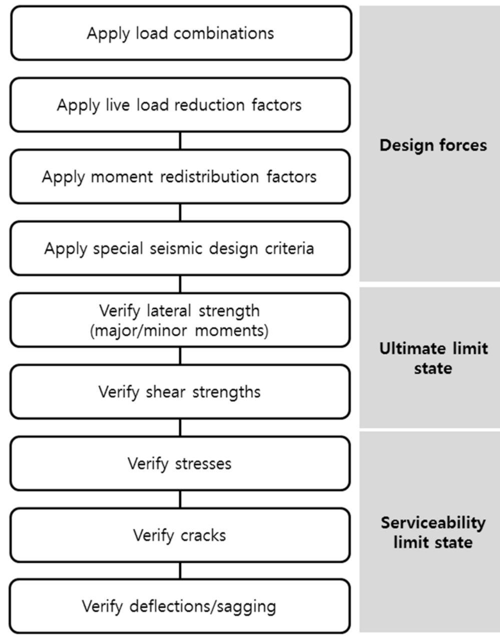
Figure 3.7.3 Dialog window for setting the curvature ductility factor for the entire

Section 8

Member Examination Procedure (Beams) -EN1992-1-1:2004

The member examination procedure for RC beams as per the EN1992-1-1:2004 design code is explained in this section.

The design forces are calculated by applying load combinations and design modification factors. Major/minor moment and shear forces are checked using industry standard values. For serviceability, cracks, stress, and deflections are checked.


Apply load combinations
Apply live load reduction factors
Apply moment redistribution factors
Apply special seismic design criteria
Verify lateral strength (major/minor moments)
Verify shear strengths
Verify stresses
Verify cracks
Verify deflections/sagging
Design forces
Ultimate limit state
Serviceability limit state

Calculation of Design Demands/Forces

Design demands/forces are computed by applying load combinations, live load reduction factors, moment redistribution factors, and seismic design criteria.

1) Applying load combinations

The analyzed demands at the member check points, load types, and the load combination factors are incorporated in computing the design demands.

2) Applying the live load reduction factor

The analyzed demands at the member check points, load types, and the load combination factors are incorporated in computing the design demands.

3) Applying moment redistribution factors

Between both ends of a beam in which minor axis moments are present, at least one end is selected for application of a moment redistribution factor less than 1.0 (as explained in Design Factors>Moment Redistribution Factors). Then, the design moment incorporating the moment redistribution factor is calculated.

4) Applying special seismic design criteria

If seismic design criteria are to be applied, then the strong shear-weak bending is applied to load combinations including earthquake loads and the design shear forces are calculated as shown below.

Table 3.8.1 Calculation of design shear forces using the strong shear-weak bending principle, when applying seismic design criteria

$V_{e1,cw,1} = V_g + \dfrac{M_{i,d,i(+)} + M_{i,d,j(-)}}{l_n}$	$V_{e1,ccw,1} = V_g + \dfrac{M_{i,d,i(-)} + M_{i,d,j(+)}}{l_n}$
$V_{e1,cw,2} = V_g - \dfrac{M_{i,d,i(+)} + M_{i,d,j(-)}}{l_n}$	$V_{e1,ccw,2} = V_g - \dfrac{M_{i,d,i(-)} + M_{i,d,j(+)}}{l_n}$
$V_{e1,cw} = \max\left[\left\|V_{e1,cw,1}\right\|, \left\|V_{e1,cw,2}\right\|\right]$	$V_{e1,ccw} = \max\left[\left\|V_{e1,ccw,1}\right\|, \left\|V_{e1,ccw,2}\right\|\right]$
$V_{e1} = \max\left[V_{e1,cw}, V_{e1,ccw}\right]$	

End moments $M_{i,d}$ are used in computing design shear, and are calculated as follows:

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \min\left[1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right] \tag{3.8.1}$$

Here,

γ_{Rd} = Factor incorporating the increased strength due to strain hardening of steel. Specified by the user

$M_{Rb,i}$ = Design moment for the end of the member

ΣM_{Rc} = Sum of the design moments of column nodes

ΣM_{Rb} = Sum of the design moments of beam nodes

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

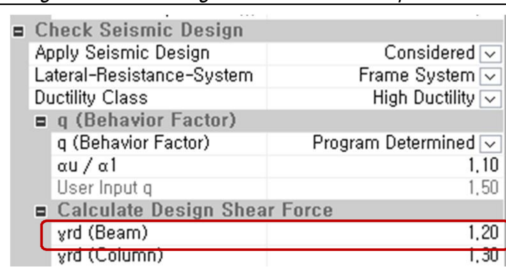


Figure 3.8.1 Dialog window for setting design shear force parameters for the entire

ULS: Ultimate Limit State

The design strength of each structural member must exceed the required strength computed from the load combinations.

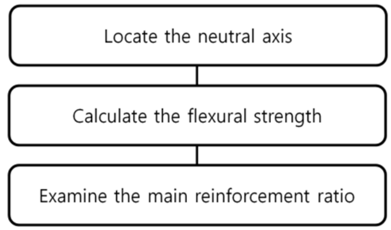
1) Bending/Flexural Strength

Design flexural strength (based on the major/minor moments) must exceed the required flexural strength. The main reinforcement must satisfy the minimum and maximum steel ratios.

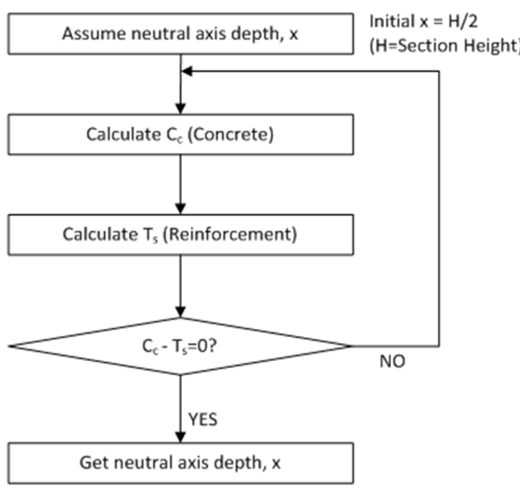
$$M_{Ed(+)} \le M_{Rd(+)} \tag{3.8.2}$$

$$M_{Rd(-)} \le M_{Rd(-)} \tag{3.8.3}$$

$$\rho_{\min} \leq \rho \leq \rho_{\max} \tag{3.8.4}$$



The location of the neutral axis is the most important number in computing the flexural strength of a member. Iterative methods are used to find the solution that satisfies equilibrium.



If x is the neutral axis, then the compressive force taken on by the concrete, C_c, is calculated as:

$$C_c = \eta f_{cd} \int_{dA} \lambda x \tag{3.8.5}$$

Here,

λ : effective height factor of the compressive portion of the concrete

η : effective strength factor

Table 3.8.2 Effective height and strength factors depending on the compressive force in the concrete

Criteria	λ	η
$f_{ck} \le 50MPa$	0.8	1.0
$50 < f_{ck} \le 90MPa$	$0.8-(f_{ck}-50)/400$	$1.0-(f_{ck}-50)/200$
$f_{ck} > 90MPa$	0.7	0.8

x : Depth of the neutral axis

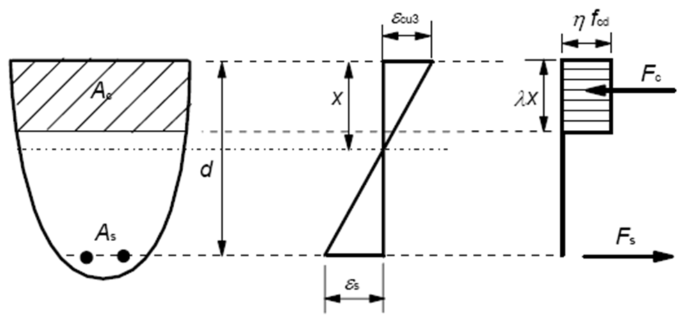


Figure 3.8.2 Height of the neutral axis for a high strength RC block

The compressive force taken on by the steel, C_s, and the tensile force taken on by the steel, T_s, is calculated as follows.

$$C_s = \sum A_{sci}\left(f_{si} - \eta f_{cd}\right) \tag{3.8.6}$$

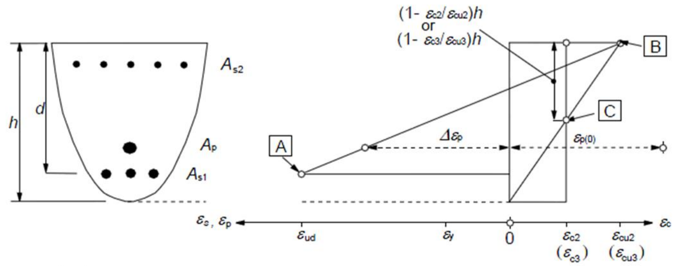
$$T_s = \sum A_{sti} f_{si} \tag{3.8.7}$$

Here,

$$f_{si} = \min\left[f_{yd},\ \varepsilon_{si} E_{si}\right]$$

$$\varepsilon_{si} = \frac{d_i - x}{x}\varepsilon_{cu}$$

Figure 3.8.3 Strain distribution at the ultimate limit state



A - reinforcing steel tension strain limit

B - concrete compression strain limit

C - concrete pure compression strain limit

ε_{cu} : ultimate compressive strain of concrete ($\varepsilon_{cu} = \varepsilon_{cu1}$)

표 3.8.3 Ultimate compressive strain depending on the compressive strength of concrete

Criteria	ε_{cu1}
fck ≤ 50MPa	0.0035
50 < fck ≤ 90MPa	$[2.8+27\{(98-f_{cm})/100\}4]/1000$, $f_{cm}=f_{ck}+8MPa$
fck > 90MPa	0.0028

This program uses the bisection method (one of the numerical analysis methods) to find the neutral axis. Convergence/stopping criteria are shown below.

Table 3.8.4 Convergence/stopping criteria for locating the neutral axis

Stopping Criteria	Description
Convergence	$\left\| \frac{C_c + C_s}{T_s} - 1.0 \right\| \leq 0.001 \ \ (tolerance)$
No convergence (Pause computations)	When the number of iterations is larger than 20, it is deemed that convergence will not be reached. The section is either increased or the rebar information is modified (location, number of rebars, spacing, etc).

After locating the neutral axis, the design bending strength is computed as follows.

$$M_{Rdcc} = C_c(x - 0.5\lambda x) \tag{3.8.8}$$

$$M_{Rdsc} = \sum C_{si}(x - d_i) \tag{3.8.9}$$

$$M_{Rdst} = \sum T_{si}(d_i - x) \tag{3.8.10}$$

$$M_{Rd} = M_{Rdcc} + M_{Rdsc} + M_{Rdst} \tag{3.8.11}$$

The minimum/maximum steel ratios of the main reinforcement are as follows.

Table 3.8.5
Minimum/maximum steel ratios of the main

ρ_{min}	▸ Without application of seismic design criteria $\rho_{min} = \max\left[0.26\frac{f_{ctm}}{f_{yk}},\ 0.0013\right]$ ▸With application of seismic design criteria $\rho_{min} = 0.5\frac{f_{ctm}}{f_{yk}}$
ρ_{max}	User-specified

2) Shear strength

The design shear strength must exceed the required shear strength. The shear reinforcement spacing must be less than the maximum spacing limit set by industry standards.

$$V_{Ed} \le V_{Rd} \tag{3.8.12}$$

$$s \le s_{max} \tag{3.8.13}$$

If concrete takes on the full shear force, steel shear strength may be neglected. However, if the shear force exceeds the resisting force of the concrete, then the shear steel will take on the full shear load. Using these assumptions, the design shear strength can be calculated as follows:

$$V_{Rd} = \begin{cases} V_{Rd,c} & (V_{Ed} \le V_{Rd,c}) \\ V_{Rd,s} & (V_{Ed} > V_{Rd,c}) \end{cases} \tag{3.8.14}$$

The shear force taken on by the concrete is calculated as shown below. Typically, designs consider σ_{cp}, but this program does not consider axial forces. Thus, $\sigma_{cp}=0$ and the shear forces are calculated accordingly.

$$V_{Rd,c1} = C_{Rd,c}k(100\rho_l f_{ck})^{1/3}bd \tag{3.8.15}$$

$$V_{Rd,c2} = v_{\min} bd \tag{3.8.16}$$

$$V_{Rd,c} = \min\left[V_{Rd,c1},\ V_{Rd,c2}\right] \tag{3.8.17}$$

Here,

$$C_{Rd,c} = \frac{0.18}{\gamma_c}$$

$$k = \min\left[1 + \sqrt{\frac{200}{d}}, 2.0\right]$$

$$\rho_l = \min\left[\frac{A_{sl}}{bd},\ 0.02\right]$$

$$k_1 = 0.15$$

$$v_{\min} = 0.035 k^{3/2} f_{ck}^{1/2}$$

Shear force taken on by the shear reinforcement steel is computed as shown below.

$$V_{Rd,s1} = \frac{A_{sw}}{s} z f_{wd} \cot\theta \tag{3.8.18}$$

$$V_{Rd,\max} = \frac{\alpha_{cw} b z v_1 f_{cd}}{\cot\theta + \tan\theta} \tag{3.8.19}$$

$$V_{Rd,s} = \min\left[V_{Rd,s1},\ V_{Rd,\max}\right] \tag{3.8.20}$$

Here,

$z = 0.9d$

α_{cw} = Factor that incorporates the compressive stress state. In beams, axial force is neglected and thus σ_{cp}=0, meaning α_{cw} =1.0.

Table 3.8.6 Recommended values of αcw for non-prestressed structural

Criteria	αcw
$0<\sigma_{cp} \le 0.25f_{cd}$	$1+\sigma_{cp}/f_{cd}$
$0.25\ f_{cd} <\sigma_{cp} \le 0.5f_{cd}$	1.25
$0.5\ f_{cd} <\sigma_{cp} \le 1.0f_{cd}$	$2.5(1-\sigma_{cp}/f_{cd})$

θ = Angle of the compressive concrete struts. Applies user-specified values

Maximum spacing limits for the shear reinforcement is a function of the seismic design criteria and the calculations are shown below.

Table 3.8.7 Maximum spacing limits for shear reinforcement, as a function of seismic design criteria

Without applying seismic design criteria	This program only considers vertical shear reinforcements, and thus, by assuming $\alpha = 90°$, $s_{l,\max} = 0.75d(1+\cot\alpha) = 0.75d$
Applying seismic design criteria	▸ Ends (DCM) $s_{l,\max} = \min[0.25H, 24D_{shear}, 8D_{main}, 225\text{mm}]$ ▸ Ends (DCH) $s_{l,\max} = \min[0.25H, 24D_{shear}, 6D_{main}, 175\text{mm}]$ ▸ Interior $s_{l,\max} = 0.75d(1+\cot\alpha) = 0.75d$

When applying seismic design criteria, shear reinforcement should be more densely arranged in some portions of certain structural members. The length of the member in which a more dense arrangement is required is calculated as follows:

$$l_{cr} = \begin{cases} 1.5h_w & (\text{DCH}) \\ 1.0h_w & (\text{DCM}) \end{cases} \tag{3.8.21}$$

Minimum steel ratio for shear reinforcement is shown below.

$$\rho_{w,\min} = 0.08\frac{\sqrt{f_{ck}}}{f_{yk}} \tag{3.8.22}$$

SLS : Serviceability Limit State

The serviceability limit state of beams is checked with respect to stress limits, crack controls, and deflection controls.

1) Stress limits

Concrete compressive stresses are limited to ensure that longitudinal cracking or other miscellaneous cracking does not occur. The characteristic load combination (one of the serviceability load combinations) are used to check the stress limits.

$$\sigma_c \leq k_1 f_{ck} \tag{3.8.23}$$

Moreover, among the serviceability load combinations, the "Quasi-permanent" load combination is used to compare its concrete stress and the following limits to determine the linearity of creep.

Table 3.8.8 Criteria for determining creep linearity

$\sigma_c \leq k_2 f_{ck}$	Linear creep
$\sigma_c > k_2 f_{ck}$	Nonlinear creep

The tensile stress in the steel is limited to ensure that inelastic strain and excessive cracks/strain do not occur, and is limited using the following equation:

$$\sigma_s \leq k_3 f_{yk} \tag{3.8.24}$$

The required coefficients for stress checks, k_1, k_2, k_3, k_4, may be set by the user, and the default values are the recommended values in the design code.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

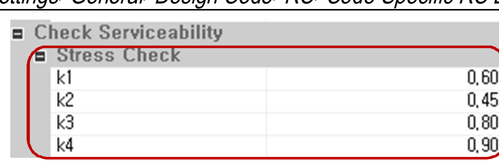


Figure 3.8.4 Dialog window showing set up of Serviceability Limit State

2) Crack control

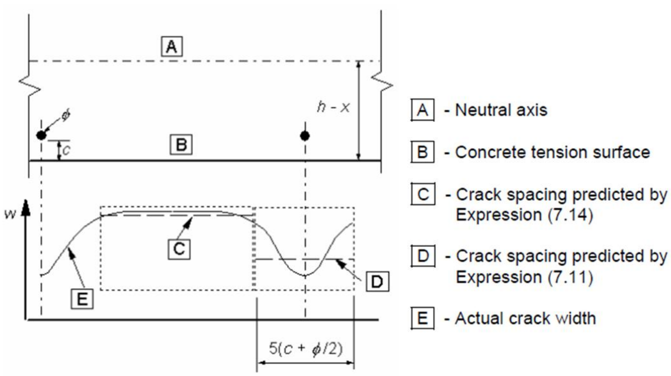
Cracks negatively impact the structure's performance, and must be limited to ensure that appearance is not excessively altered, using the following inequality:

$$w_k \le w_{\max} \tag{3.8.25}$$

Creep width w_k is calculated as shown below.

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{3.8.26}$$

Figure 3.8.5 Creep width w_k, at the same distance from the concrete edge as the rebar spacing



Here,

$$s_{r,\max} = k_3 c + \frac{k_1 k_2 k_4 \phi}{\rho_{p,eff}} \tag{3.8.27}$$

ϕ = Steel diameter. If various steel diameters are used, the equivalent diameter ϕ_{eq} is computed and used instead.

$$\phi_{eq} = \frac{n_1\phi_1^2 + n_2\phi_2^2}{n_1\phi_1 + n_2\phi_2}$$

c = Longitudinal steel cover distance.

k_1 = Factor that incorporates steel's bonding characteristics. This program uses a value of 0.8.

k_2 = Factor that incorporates strain distribution. This program uses a value of 0.5.

$k_3 = 3.4$

$k_4 = 0.425$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}\left(1 + \alpha_e \rho_{p,eff}\right)}{E_s} \geq 0.6 \frac{\sigma_s}{E_s} \tag{3.8.28}$$

σ_s = Tensile stress of the steel, assuming cracked section.

k_t = Factor depending on the load period.

- Short term loading : 0.6

- Long term loading : 0.4

$$f_{ct,eff} = f_{ctm}$$

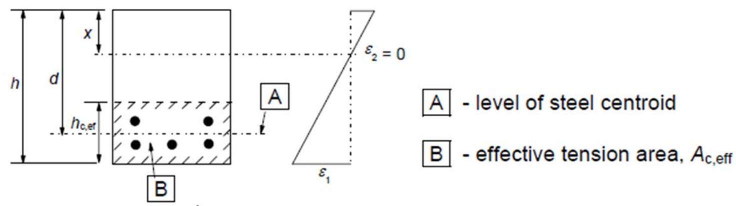
$$\alpha_e = \frac{E_s}{E_{cm}} = \frac{E_s}{22\left(\dfrac{f_{cm}}{10}\right)^{0.3}}$$

$$\rho_{p,eff} = \frac{A_s + \xi_1^2 A_p'}{A_{c,eff}} = \frac{A_s}{A_{c,eff}}$$

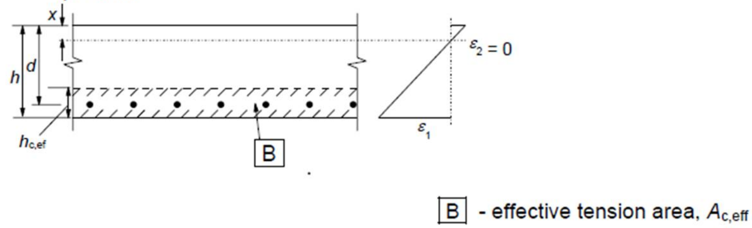
$$A_{c,eff} = b h_{c,ef}$$

$$h_{c,ef} = \min\left[2.5(h-d),\ \frac{h-x}{3},\ \frac{h}{2}\right]$$

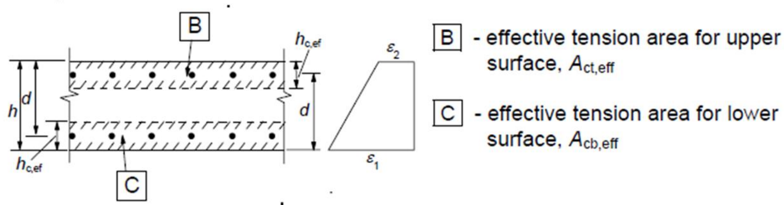
Figure 3.8.6 Effective tension area (typical cases)



a) Beam



b) Slab



c) Member in tension

The crack width limit w_{max} is decided based on the exposure category and applied load combination, and the limits are provided by the design code as shown below. This program distinguishes between the quasi-permanent and frequent load combinations, and uses the user-specified w_{max}.

Table 3.8.9 Crack width limit w_{max} for various exposure categories and applied load

Exposure category	Serviceability Load Combination Type		
	Quasi	Frequent	Characteristic
X0	0.4	User-Specified	Not Checked
XC1			
XC2	0.3		
XC3			
XC4			
XD1	0.3		
XD2			
XD3			
XS1	0.3		
XS2			
XS3			
XF1*	Not Checked		0.2 (Incorporates randomness)
XF2*			
XF3*			
XF4*			
XA1*			
XA2*			
XA3*			

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

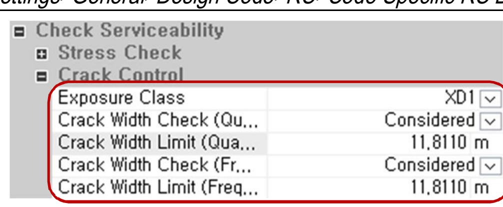


Figure 3.8.7 Dialog window for setting crack width limits

3) Deflection Checks

Excessive deflection negatively impacts the structure's performance and appearance, and can also damage nonstructural components. The actual deflection must be less than the allowable deflection:

$$\delta_{actual} \leq \delta_{allow} \tag{3.8.29}$$

The actual deflection is the product of the analyzed deflection and load combination factors. The allowable deflection applies the user-specified design member length.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

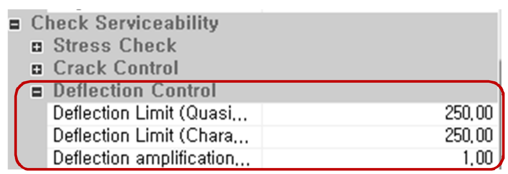


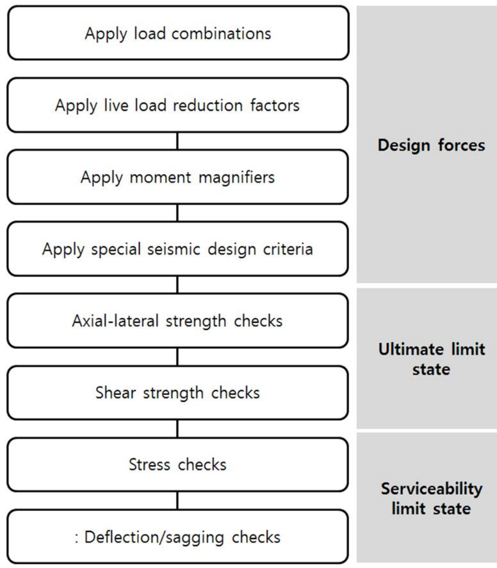
Figure 3.8.8 Dialog window for setting deflection controls

Section 9

Member Examination Procedure (Columns/Braces) -EN1992-1-1:2004

The member verification procedure for RC columns/braces following the EN1992-1-1:2004 design code is explained in this section.

The design strength is calculated using the analyzed strengths, load combinations, and design strength modification factors. Design code strengths are computed to verify the axial strength and flexural strength.


Apply load combinations
Apply live load reduction factors
Apply moment magnifiers
Apply special seismic design criteria
Axial-lateral strength checks
Shear strength checks
Stress checks
: Deflection/sagging checks
Design forces
Ultimate limit state
Serviceability limit state

Calculating design forces/demands

The design forces are calculated, factoring in the live load reduction factors, moment magnifiers, and special seismic design criteria

1) Application of load combinations

2) Application of live load reduction factors

As explained in the section Design Factors>Live Load Reduction Factors, components that are subject to live loads will have design forces that incorporate the live load reduction factor.

3) Application of moment magnifiers

When designing columns and braces, if the slenderness ratio exceeds the slenderness limit, then second order effects must be accounted for and the design moment is calculated as follows.

$$M_{Ed} = M_{0Ed} + M_2 \tag{3.9.1}$$

Here,

$$M_{0e} = \max\left[0.6M_{02} + 0.4M_{01},\ 0.4M_{02}\right]\ \left(\left|M_{02}\right| \ge \left|M_{01}\right|\right)$$

$$M_2 = N_{Ed}e_2 = N_{Ed}\frac{1}{r}\frac{l_0^2}{c}$$

$$\frac{1}{r} = K_r K_\phi \frac{1}{r_0} = K_r K_\phi \frac{\varepsilon_{yd}}{0.45d}$$

$$K_r = \max\left[\frac{n_u - n}{n_u - n_{bal}}, 1.0\right]$$

$$n_u = 1 + \omega$$

$$\omega = \frac{A_s f_{yd}}{A_c f_{cd}}$$

$$n = \frac{N_{Ed}}{A_c f_{cd}}$$

$n_{bal} = 0.4$

Kφ = a factor that incorporates creep effects,

This program does not consider creep, so the value is set to be 1.0.

4) Application of special seismic design criteria

When seismic design criteria are being applied, design shear is calculated using the strong shear-weak bending principle and the load combination that includes earthquake loads, as shown below.

Table 3.9.1 Shear force calculations for seismic design purposes, using the strong shear-weak bending principle

$V_{e1,cw,1}=V_g+\dfrac{M_{i,d,i(+)}+M_{i,d,j(-)}}{l_n}$	$V_{e1,ccw,1}=V_g+\dfrac{M_{i,d,i(-)}+M_{i,d,j(+)}}{l_n}$
$V_{e1,cw,2}=V_g-\dfrac{M_{i,d,i(+)}+M_{i,d,j(-)}}{l_n}$	$V_{e1,ccw,2}=V_g-\dfrac{M_{i,d,i(-)}+M_{i,d,j(+)}}{l_n}$
$V_{e1,cw}=\max\left[\lvert V_{e1,cw,1}\rvert,\lvert V_{e1,cw,2}\rvert\right]$	$V_{e1,ccw}=\max\left[\lvert V_{e1,ccw,1}\rvert,\lvert V_{e1,ccw,2}\rvert\right]$
$V_{e1}=\max\left[V_{e1,cw},V_{e1,ccw}\right]$	

The end moments, Mi,d, required for calculating design shear forces, is computed as follows:

$$M_{i,d}=\gamma_{Rd}M_{Rb,i}\min\left[1,\frac{\sum M_{Rc}}{\sum M_{Rb}}\right] \tag{3.9.2}$$

Here,

γ_{Rd} = Factor that incorporates the increased strength due to steel strain hardening

Specified by the user in *Design Setting*

$M_{Rb,i}$ = Design end moment.

ΣM_{Rc} = Sum of the design moments at the column nodes

ΣM_{Rb} = Sum of the design moments at the beam nodes.

Home>Design Settings>General>Design Code>RC>Code-specific RC Design Parameters

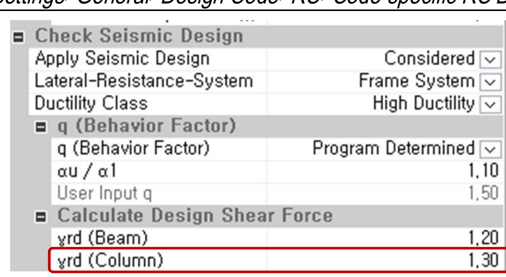


Figure 3.9.1 Dialog window showing the setup of γ_{Rd} for design shear force

ULS: Ultimate Limit State

1) Axial-lateral strength

The design strengths of members subject to both axial and lateral loading must incorporate the P-M correlation. In this program, the P-M correlations are incorporated into the computation of axial and lateral strengths. The main reinforcement ratio must satisfy the minimum and maximum steel ratio limits.

$$N_{Ed} \le N_{Rd} \tag{3.9.3}$$

$$M_{Ed} \le M_{Rd} \tag{3.9.4}$$

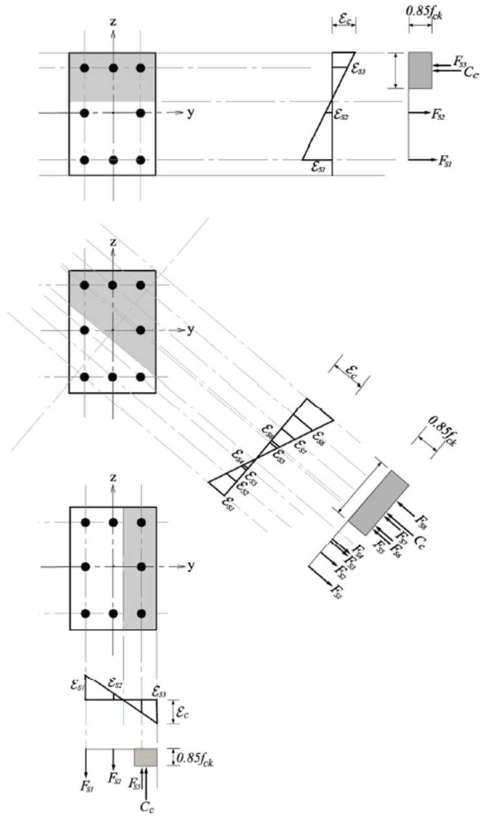
$$M_{Edy} \le M_{Rdy} \tag{3.9.5}$$

$$M_{Edz} \le M_{Rdz} \tag{3.9.6}$$

$$\rho_{\min} \le \rho \le \rho_{\max} \tag{3.9.7}$$

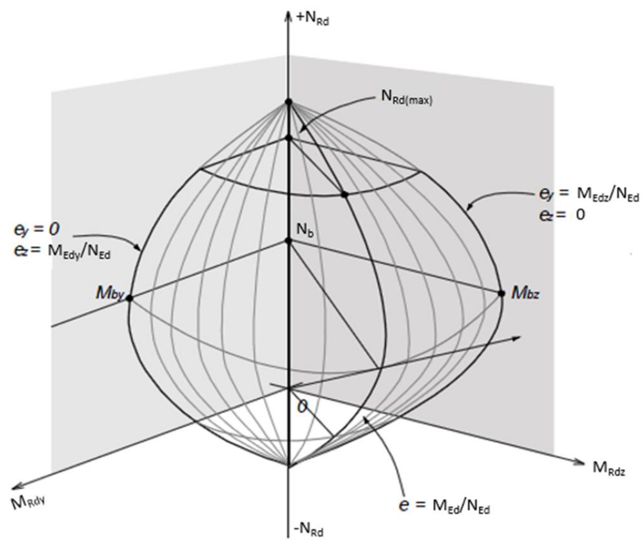
Column/brace members subject to both axial and lateral loads must satisfy force equilibrium and strain compatibility criteria. Stress-strain relationships for biaxial P-M correlations are shown below.

Figure 3.9.2 Stress strain relationships for Biaxial P-M correlation curves



The axial force and lateral moment is calculated using eccentricity. Using the resulting values, the P-M correlation curve is calculated. Through the correlation curve, the design strength corresponding to the desired force may be found.

Figure 3.9.3 Biaxial P-Mcorrelation



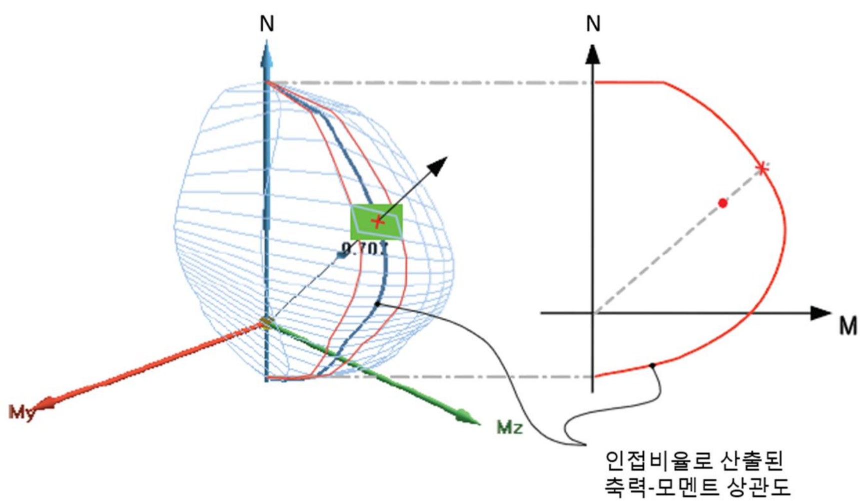


Figure 3.9.4

P-M correlation for a specific load combination

The minimum and maximum steel ratios for the main reinforcement are shown below.

Table 3.9.2 Minimum and maximum steel ratios for the main reinforcement

$\rho_{\min}$	$\max\left[\dfrac{0.10N_{Ed}}{f_{yd}A_c},\ 0.002\right]$
$\rho_{\max}$	User-specified

2) Shear strength

The design shear strength must be greater than the expected design shear demands. The main reinforcement spacing must be smaller than the maximum spacing limits set by the design code.

$$V_{Ed} \le V_{Rd} \tag{3.9.8}$$

$$s \le s_{l,\max} \tag{3.9.9}$$

If concrete takes on the full shear force, steel shear strength may be neglected. However, if the shear force exceeds the resisting force of the concrete, then the shear steel will take on the full shear load. Using these assumptions, the design shear strength can be calculated as follows:

$$V_{Rd} = \begin{cases} V_{Rd,c} & (V_{Ed} \le V_{Rd,c}) \\ V_{Rd,s} & (V_{Ed} > V_{Rd,c}) \end{cases} \tag{3.9.10}$$

The shear force taken on by the concrete is shown below.

$$V_{Rd,c1} = \left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]bd \tag{3.9.11}$$

$$V_{Rd,c2} = (v_{\min} + k_1\sigma_{cp})bd \tag{3.9.12}$$

$$V_{Rd,c} = \min\left[V_{Rd,c1},\ V_{Rd,c2}\right] \tag{3.9.13}$$

Here,

$$C_{Rd,c} = \frac{0.18}{\gamma_c}$$

$$k = \min\left[1 + \sqrt{\frac{200}{d}}, 2.0\right]$$

$$\rho_l = \min\left[\frac{A_{sl}}{bd},\ 0.02\right]$$

$$k_1 = 0.15$$

$$v_{\min} = 0.035k^{3/2} f_{ck}^{1/2}$$

The shear strength taken on by the shear reinforcement is calculated as follows:

$$V_{Rd,s1} = \frac{A_{sw}}{s} z f_{wd} \cot\theta \qquad (3.9.14)$$

$$V_{Rd,\max} = \frac{\alpha_{cw} b z v_1 f_{cd}}{\cot\theta + \tan\theta} \qquad (3.9.15)$$

$$V_{Rd,s} = \min\left[V_{Rd,s1},\ V_{Rd,\max}\right] \qquad (3.9.16)$$

Here,

$$z = 0.9d$$

α_{cw} = Factor that incorporates the compressive stress state

Table 3.9.3 Factor that incorporates the compressive stress state, $\boldsymbol{\alpha_{cw}}$

Criteria	α_{cw}
$0<\sigma_{cp} \le 0.25f_{cd}$	$1+\sigma_{cp}/f_{cd}$
$0.25\ f_{cd} <\sigma_{cp} \le 0.5f_{cd}$	1.25
$0.5\ f_{cd} <\sigma_{cp} \le 1.0f_{cd}$	$2.5(1-\sigma_{cp}/f_{cd})$

v_1 = Strength reduction factor due to cracked concrete section

Table 3.9.4 Strength reduction factor due to cracked concrete section $\boldsymbol{v_1}$

$f_{ywd} \ge 0.8f_{ywk}$	$f_{ywd} < 0.8f_{ywk}$	
	$f_{ck} \le 60$MPa	$f_{ck} > 60$MPa
$v_1 = v = 0.6\left[1 - \frac{f_{ck}}{250}\right]$	$v_1 = 0.6$	$v_1 = \max\left[0.9 - \frac{f_{ck}}{200},\ 0.5\right]$

θ = Angle of the compressive concrete struts. Applies user-specified values

The maximum spacing limits for shear reinforcement is calculated based on the seismic design criteria, as shown below.

Table 3.9.5 Shear reinforcement maximum spacing limits as a function of the seismic design criteria

Without applying seismic design criteria	$s_{l,\max} = \min[20d_L, B, H, 400\text{mm}]$
Applying seismic design criteria (Ends)	▸ DCM $s_{l,\max} = \min\left[\frac{b_0}{2}, 175\text{mm}, 8d_{bL}\right]$ ▸ DCH $s_{l,\max} = \min\left[\frac{b_0}{3}, 125\text{mm}, 6d_{bL}\right]$
Applying seismic design criteria (Interior)	$s_{l,\max} = \min[20d_{bL}, B, H, 400\text{mm}]$

When applying seismic design criteria, there are portions of the member in which shear reinforcement may need to be more densely arranged. The length of this section of the member is calculated as shown below.

$$l_{cr} = \begin{cases} \max\left[1.5h_c, \frac{l_{cl}}{6}, 600\text{ mm}\right] \text{(DCH)} \\ \max\left[1.0h_c, \frac{l_{cl}}{6}, 450\text{ mm}\right] \text{(DCM)} \end{cases} \tag{3.9.17}$$

The minimum steel ratio for shear reinforcement is calculated as follows:

$$\rho_{w,\min} = 0.08\frac{\sqrt{f_{ck}}}{f_{yk}} \tag{3.9.18}$$

SLS : Serviceability Limit State

The serviceability limit state of columns is checked with regards to stress and deflection limits.

1) Stress limits

Concrete compressive stresses are limited to ensure that longitudinal cracking or other miscellaneous cracking does not occur. The characteristic load combination (one of the serviceability load combinations) are used to check the stress limits.

$$\sigma_c \le k_1 f_{ck} \tag{3.9.19}$$

Moreover, among the serviceability load combinations, the "Quasi-permanent" load combination is used to compare its concrete stress and the following limits to determine the linearity of creep.

Table 3.9.6 Criteria for determing creep linearity

$\sigma_c \le k_2 f_{ck}$	Linear creep
$\sigma_c > k_2 f_{ck}$	Nonlinear creep

The tensile stress in the steel is limited to ensure that excessive cracks/strains do not form, and is limited using the following equation:

$$\sigma_s \le k_3 f_{yk} \tag{3.9.20}$$

The required coefficients for stress checks, k_1, k_2, k_3, k_4, may be specified by the user, and the default values are the recommended values in the design code.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

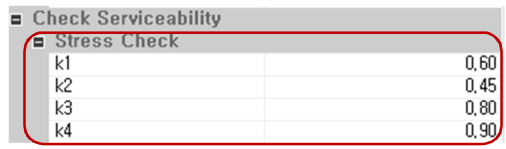


Figure 3.9.5 Serviceability limit state settings window

2) Deflection checks

Excessive deflection negatively impacts the structure's performance and appearance, and can also damage nonstructural components. The actual deflection must be less than the allowable deflection:

$$\delta_{actual} \le \delta_{allow} \tag{3.9.21}$$

The actual deflection is the product of the analyzed deflection and load combination factors. The allowable deflection applies the user-specified design member length.

Home>Design Settings>General>Design Code>RC>Code-Specific RC Design Parameters

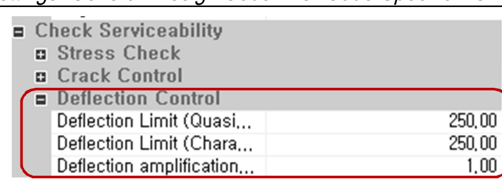


Figure 3.9.6 Dialog window for setting deflection controls

Section 10 Rebar/Arrangement

In this program, the user-specified reinforcement information is used as the basis for RC members and suggests steel rebar arrangements that satisfy strength and steel detail requirements.

Rebar/Arrangement Settings

Conditions for setting rebar/arrangement information for RC members can be set in *Home>Design Setting* or *Design Calculation Option>Rebar Arrangement*.

Home>Design Setting>Rebar/Arrangement

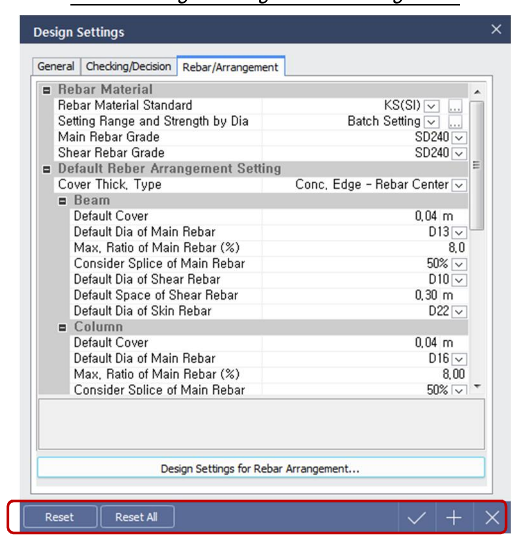
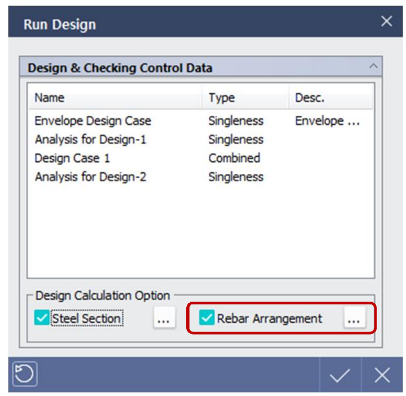


Figure 3.10.1 Design Setting dialog window

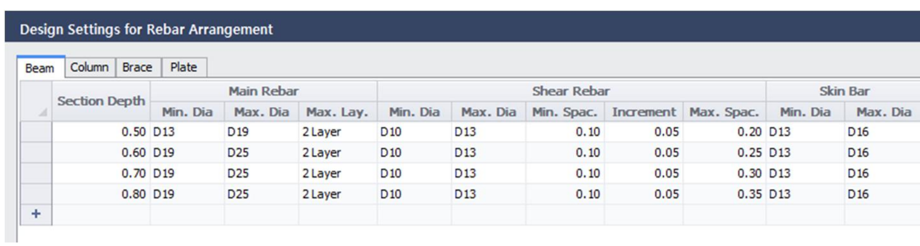
Figure 3.10.2 Rebar Arrangement dialog window

Run Design>Design Calculation Option



When pressing the rebar arrangement button, the following dialog window will pop up in which rebar settings for various member types can be set.

Figure 3.10.3 Dialog window for rebar arrangement setting for various member types



Section Depth	Main Rebar Min. Dia	Main Rebar Max. Dia	Main Rebar Max. Lay.	Shear Rebar Min. Dia	Shear Rebar Max. Dia	Shear Rebar Min. Spac.	Shear Rebar Increment	Shear Rebar Max. Spac.	Skin Bar Min. Dia	Skin Bar Max. Dia
0.50	D13	D19	2 Layer	D10	D13	0.10	0.05	0.20	D13	D16
0.60	D19	D25	2 Layer	D10	D13	0.10	0.05	0.25	D13	D16
0.70	D19	D25	2 Layer	D10	D13	0.10	0.05	0.30	D13	D16
0.80	D19	D25	2 Layer	D10	D13	0.10	0.05	0.35	D13	D16

▸ Beams: Rebar information is specified depending on the section height.

Table 3.10.1 Rebar information for beam members

Main reinforcement	Minimum/maximum diameters and maximum number of rebars are specified. A list of diameters ranging from the minimum to maximum diameter is created and the most efficient rebar/arrangement is computed.
Shear reinforcement	Minimum/maximum diameters, minimum/maximum spacing, and spacing increment are specified. A list of combinations of the diameters, spacing, and spacing increments are created and the most efficient rebar/arrangement is computed.
Outer reinforcement	Minimum/maximum diameters are defined. A list of diameters ranging from the minimum to maximum diameter is created and the most efficient rebar/arrangement is computed.

▸ Columns/Braces: Rebar information is specified depending on the minimum section measurements.

Table 3.10.2 Rebar information for column/brace members

Main reinforcement	Minimum/maximum diameters are specified. A list of diameters ranging from the minimum to maximum diameter is created and the most efficient rebar/arrangement is computed.
Shear reinforcement	Minimum/maximum diameters, minimum/maximum spacing, and spacing increment are specified. A list of combinations of the diameters, spacing, and spacing increments are created and the most efficient rebar/arrangement is computed.

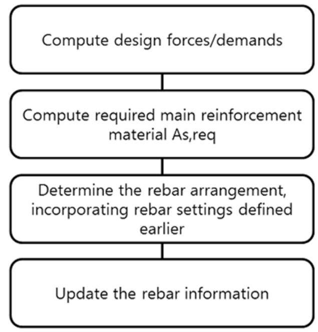
▸ Plate: Rebar information is specified depending on the minimum thickness.

Table 3.10.3 Rebar information for plate members

Main reinforcement	Minimum/maximum diameters, top reinforcement units, lower reinforcement units, minimum/maximum spacing, and spacing increments are specified. A list of combinations of the diameters and spacing is created and the most efficient rebar/arrangement is computed.
Shear reinforcement	Minimum/maximum diameters, minimum/maximum spacing, and spacing increments are specified. A list of combinations of the diameters and spacing is created and the most efficient rebar/arrangement is computed.

RC Beam Main Reinforcement Rebar/Arrangement

The beam's main reinforcement rebar/arrangement computation process is as follows.



The required steel amount for main reinforcement is calculated by assuming a singly reinforced beam and computing the design strength equation accordingly. This equation is used to compute the required steel amounts, and in the case of ACI318-11, the calculations are as follows:

$$C = 0.85 f_{ck} ab$$

(3.10.1)

$$T = A_s f_y$$

(3.10.2)

$C = T$ 에서,

(3.10.3)

$$a = \frac{A_s f_y}{0.85 f_{ck} b}$$

(3.10.4)

$$M_u \le \phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right) = \phi A_s f_y \left(d - \frac{1}{2} \frac{A_s f_y}{0.85 f_{ck} b} \right),$$ Rearranged with respect to A_s (3.10.5)

$$\phi \frac{1}{2} \frac{f_y^2}{0.85 f_{ck} b} A_s^2 - \phi f_y d A_s + M_u = 0$$

(3.10.6)

The required steel amount calculated following the above procedure and the user-specified main reinforcement diameter list are used in conjunction with the design code rebar requirements to create the most efficient main reinforcement rebar arrangement.

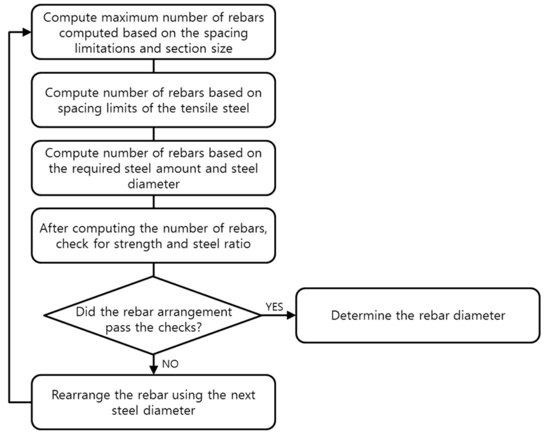
The main reinforcement diameter list begins from the smallest diameter and finds the rebar arrangement that works best for the given section.

The number of rebars in a single layer is calculated based on the rebar arrangement information, required steel ratio, and the spacing limitations. As per ACI318-11, the following items are considered.

Table 3.10.4 Number of rebars arranged for a single layer, as per ACI318-11

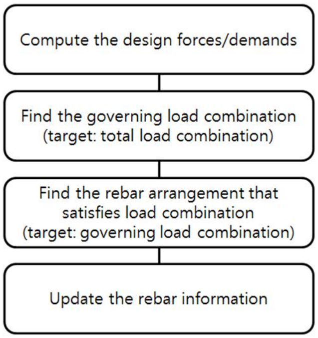
Number of required bars based on the steel clear cove (Maximum number of rebars)	$dist_{clear} = \max\left[d_b, 1\text{ in}, \frac{4}{3}d_{gravel}\right]$ $n_{rebar} = \text{FLOOR}\left[\frac{b-2d_c+dist_{clear}+d_b}{(\text{Splice Ratio})d_b+dist_{clear}}\right]$
Number of bars based on the tensile steel spacing limiations (Crack limitations)	$s_a = \min\left[15\frac{40000}{f_s}-2.5c_c, 12\frac{40000}{f_s}\right]$ $n_{rebar} = \text{CEIL}\left[\frac{b-2d_c}{s_a}\right]+1$
Number of required rebars	$n_{rebar} = \text{CEIL}\left[\frac{A_{s,req}}{A_{s1}}\right]$

The rebar numbers and the steel diameter computed from the above procedure goes through strength and steel ratio checks. If the numbers do not pass these checks, then the rebar arrangement for the next diameter is computed and checked.



RC Column Rebar Arrangement

Main reinforcement rebar arrangement for column members is as follows.



Columns are subject to P-M correlation relationships and thus all load combinations must be considered to obtain the most accurate results. However, for the efficiency of rebar computations, the governing load combination by finding the combination that yields the greatest required steel amount. Additionally, the user-specified main reinforcement diameter list and rebar design code requirements are incorporated in computing the main reinforcement rebar arrangement. The main reinforcement diameter list is searched, starting with the smallest diameter, until the rebar arrangement that satisfies the load demands and the given section is found.

The number of rebars for rectangular columns is calculated by incorporating the spacing limitations and steel ratio limitations. As per ACI318-11, the following items are considered.

Table 3.10.5 Number of rebars for rectangular columns, as per ACI318-11

Number of maximum rebars based on the steel clear cover	$dist_{clear} = \max\left[1.5d_b,\ 1.5\text{ in},\frac{4}{3}d_{gravel}\right]$ $n_{rebar} = \text{FLOOR}\left[\frac{H - 2d_c + (\text{Splice Ratio} - 1)d_b}{dist_{clear} + (\text{Splice Ratio})d_b}\right] + 1$
Number of rebars limited by the maximum steel ratio	$n_{rebar} = \text{FLOOR}\left[\frac{A_g \rho_{\max}}{2A_{s1}}\right]$

The number of rebars and the steel diameter determined following the above procedure goes through strength and steel ratio checks. If the rebar arrangement satisfies the checks, then the rebar arrangement process is complete. However, if the arrangement does not satisfy the checks, then the next diameter is used to compute another possible rebar arrangement.