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www.csiamerica.com

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References
The design of concrete frames is seamlessly integrated within the program. Initiation of the design process, along with control of various design parameters, is accomplished using the Design menu.

It should be noted that two design processes are available in CSiBridge: superstructure design (on the Design/Rating tab) and design of the individual elements comprising the structure (the Advanced > Frame Design commands). This manual addresses the second design process.

Automated design at the object level is available for any one of a number of user-selected design codes, as long as the structures have first been modeled and analyzed by the program. Model and analysis data, such as material properties and member forces, are recovered directly from the model database, and no additional user input is required if the design defaults are acceptable.

The design is based on a set of user-specified loading combinations. However, the program provides default load combinations for each design code supported. If the default load combinations are acceptable, no definition of additional load combinations is required.

In the design of columns, the program calculates the required longitudinal and shear reinforcement. However, the user may specify the longitudinal steel, in
which case a column capacity ratio is reported. The column capacity ratio gives an indication of the stress condition with respect to the capacity of the column.

The biaxial column capacity check is based on the generation of consistent three-dimensional interaction surfaces. It does not use any empirical formulations that extrapolate uniaxial interaction curves to approximate biaxial action.

Interaction surfaces are generated for user-specified column reinforcing configurations. The column configurations may be rectangular, square, or circular, with similar reinforcing patterns. The calculation of moment magnification factors, unsupported lengths, and strength reduction factors is automated in the algorithm.

Every beam member is designed for flexure, shear, and torsion at output stations along the beam span.

All beam-column joints are investigated for existing shear conditions.

For moment resisting frames for seismic zones 2, 3 and 4, the shear design of the columns, beams, and joints is based on the probable moment capacities of the members. Also, the program will produce ratios of the beam moment capacities with respect to the column moment capacities, to investigate weak beam/strong column aspects, including the effects of axial force.

Output data can be presented graphically on the model, in tables for both input and output data, or on the calculation sheet prepared for each member. For each presentation method, the output is in a format that allows the engineer to quickly study the stress conditions that exist in the structure and, in the event the member reinforcing is not adequate, aids the engineer in taking appropriate remedial measures, including altering the design member without rerunning the entire analysis.

### 1.1 Organization

This manual is designed to help you quickly become productive with the concrete frame design options of AASHTO LRFD 2012. Chapter 2 provides detailed descriptions of the Deign Prerequisites used for AASHTO LRFD 2012. Chapter 3 provides detailed descriptions of the code-specific process used for AASHTO LRFD 2012. The appendices provide details on certain topics referenced in this manual.
1.2 Recommended Reading/Practice

It is strongly recommended that you read this manual and review any applicable “Watch & Learn” Series™ tutorials, which are found on our web site, http://www.esiamerica.com, before attempting to design a concrete frame. Additional information can be found in the on-line Help facility available from within the program’s main menu.
Chapter 2
Design Prerequisites

This chapter provides an overview of the basic assumptions, design preconditions, and some of the design parameters that affect the design of concrete frames.

In writing this manual it has been assumed that the user has an engineering background in the general area of structural reinforced concrete design and familiarity with AASHTO LRFD 2012 codes.

2.1 Design Load Combinations

The design load combinations are used for determining the various combinations of the load cases for which the structure needs to be designed/checked. The load combination factors to be used vary with the selected design code. The load combination factors are applied to the forces and moments obtained from the associated load cases and are then summed to obtain the factored design forces and moments for the load combination.

For multi-valued load combinations involving response spectrum, time history, moving loads and multi-valued combinations (of type enveloping, square-root of the sum of the squares or absolute) where any correspondence between interacting quantities is lost, the program automatically produces multiple sub combinations using maxima/minima permutations of interacting quantities. Separate
combinations with negative factors for response spectrum cases are not required because the program automatically takes the minima to be the negative of the maxima for response spectrum cases and the above described permutations generate the required sub combinations.

When a design combination involves only a single multi-valued case of time history or moving load, further options are available. The program has an option to request that time history combinations produce sub combinations for each time step of the time history. Also an option is available to request that moving load combinations produce sub combinations using maxima and minima of each design quantity but with corresponding values of interacting quantities.

For normal loading conditions involving static dead load, live load, wind load, and earthquake load, or dynamic response spectrum earthquake load, the program has built-in default loading combinations for each design code. These are based on the code recommendations and are documented in Chapter 3.

For other loading conditions involving moving load, time history, pattern live loads, separate consideration of roof live load, snow load, and so on, the user must define design loading combinations either in lieu of or in addition to the default design loading combinations.

The default load combinations assume all load cases declared as dead load to be additive. Similarly, all cases declared as live load are assumed additive. However, each load case declared as wind or earthquake, or response spectrum cases, is assumed to be non additive with each other and produces multiple lateral load combinations. Also wind and static earthquake cases produce separate loading combinations with the sense (positive or negative) reversed. If these conditions are not correct, the user must provide the appropriate design combinations.

The default load combinations are included in design if the user requests them to be included or if no other user-defined combination is available for concrete design. If any default combination is included in design, all default combinations will automatically be updated by the program any time the design code is changed or if static or response spectrum load cases are modified.

Live load reduction factors can be applied to the member forces of the live load case on an element-by-element basis to reduce the contribution of the live load to the factored loading.
The user is cautioned that if moving load or time history results are not requested to be recovered in the analysis for some or all of the frame members, the effects of those loads will be assumed to be zero in any combination that includes them.

2.2 Design and Check Stations

For each load combination, each element is designed or checked at a number of locations along the length of the element. The locations are based on equally spaced segments along the clear length of the element. The number of segments in an element is requested by the user before the analysis is performed. The user can refine the design along the length of an element by requesting more segments.

When using the AASHTO LRFD 2012 design code, requirements for joint design at the beam-to-column connections are evaluated at the top most station of each column. The program also performs a joint shear analysis at the same station to determine if special considerations are required in any of the joint panel zones. The ratio of the beam flexural capacities with respect to the column flexural capacities considering axial force effect associated with the weak-beam/strong-column aspect of any beam/column intersection are reported.

2.3 Identifying Beams and Columns

In the program, all beams and columns are represented as frame elements, but design of beams and columns requires separate treatment. Identification for a concrete element is accomplished by specifying the frame section assigned to the element to be of type beam or column. If any brace element exists in the frame, the brace element also would be identified as a beam or a column element, depending on the section assigned to the brace element.

2.4 Design of Beams

In the design of concrete beams, in general, the program calculates and reports the required areas of steel for flexure and shear based on the beam moments, shears, load combination factors, and other criteria, which are described in detail in the code-specific manuals. The reinforcement requirements are calculated at a user-defined number of stations along the beam span.
All the beams are designed for major direction flexure, shear and torsion only. Effects due to any axial forces and minor direction bending that may exist in the beams must be investigated independently by the user.

In designing the flexural reinforcement for the major moment at a particular section of a particular beam, the steps involve the determination of the maximum factored moments and the determination of the reinforcing steel. The beam section is designed for the maximum positive and maximum negative factored moment envelopes obtained from all of the load combinations. Negative beam moments produce top steel. In such cases, the beam is always designed as a Rectangular section. Positive beam moments produce bottom steel. In such cases, the beam may be designed as a Rectangular beam or a T beam. For the design of flexural reinforcement, the beam is first designed as a singly reinforced beam. If the beam section is not adequate, the required compression reinforcement is calculated.

In designing the shear reinforcement for a particular beam for a particular set of loading combinations at a particular station due to the beam major shear, the steps involve the determination of the factored shear force, the determination of the shear force that can be resisted by concrete, and the determination of the reinforcement steel required to carry the balance.

Special considerations for seismic design are incorporated into the program for the AASHTO LRFD 2012 code.

### 2.5 Design of Columns

In the design of the columns, the program calculates the required longitudinal steel, or if the longitudinal steel is specified, the column stress condition is reported in terms of a column capacity ratio, which is a factor that gives an indication of the stress condition of the column with respect to the capacity of the column. The design procedure for the reinforced concrete columns of the structure involves the following steps:

- Generate axial force-biaxial moment interaction surfaces for all of the different concrete section types in the model.
- Check the capacity of each column for the factored axial force and bending moments obtained from each loading combination at each end of the column. This step is also used to calculate the required reinforcement (if none was
specified) that will produce a capacity ratio of 1.0.

The generation of the interaction surface is based on the assumed strain and stress distributions and some other simplifying assumptions. These stress and strain distributions and the assumptions are documented in Chapter 3.

The shear reinforcement design procedure for columns is very similar to that for beams, except that the effect of the axial force on the concrete shear capacity must be considered.

For certain special seismic cases, the design of columns for shear is based on the capacity shear. The capacity shear force in a particular direction is calculated from the moment capacities of the column associated with the factored axial force acting on the column. For each load combination, the factored axial load is calculated using the load cases and the corresponding load combination factors. Then, the moment capacity of the column in a particular direction under the influence of the axial force is calculated, using the uniaxial interaction diagram in the corresponding direction as documented in Chapter 3.

2.6 P-Delta Effects

The program design process requires that the analysis results include P-delta effects. The P-delta effects are considered differently for “braced” or “non-sway” and “unbraced” or “sway” components of moments in columns or frames. For the braced moments in columns, the effect of P-delta is limited to “individual member stability.” For unbraced components, “lateral drift effects” should be considered in addition to individual member stability effect. The program assumes that “braced” or “nonsway” moments are contributed from the “dead” or “live” loads, whereas, “unbraced” or “sway” moments are contributed from all other types of loads.

For the individual member stability effects, the moments are magnified with moment magnification factors, as documented in Chapter 3 of this manual.

For lateral drift effects, the program assumes that the P-delta analysis is performed and that the amplification is already included in the results. The moments and forces obtained from P-delta analysis are further amplified for individual column stability effect if required by the code.
Users of the program should be aware that the default analysis option is turned OFF for P-delta effect. The user can turn the P-delta analysis ON and set the maximum number of iterations for the analysis. The default number of iterations for P-delta analysis is 1. Further details about P-delta analysis are provided in Appendix A of this design manual.

2.7 Element Unsupported Lengths

To account for column slenderness effects, the column unsupported lengths are required. The two unsupported lengths are \( l_{33} \) and \( l_{22} \). These are the lengths between support points of the element in the corresponding directions. The length \( l_{33} \) corresponds to instability about the 3-3 axis (major axis), and \( l_{22} \) corresponds to instability about the 2-2 axis (minor axis).

Normally, the unsupported element length is equal to the length of the element, i.e., the distance between END-I and END-J of the element. The program, however, allows users to assign several elements to be treated as a single member for design. This can be accomplished differently for major and minor bending, as documented in Appendix B of this design manual.

The user has options to specify the unsupported lengths of the elements on an element-by-element basis.

2.8 Choice of Input Units

English as well as SI and MKS metric units can be used for input. The codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. For example, the AASHTO code is published in inch-pound-second units. By default, all equations and descriptions presented in the “Design Process” chapter correspond to inch-kip-second units. However, any system of units can be used to define and design a structure in the program.
Chapter 3
Design Process

This chapter provides a detailed description of the code-specific algorithms used in the design of concrete frames when the AASHTO LRFD 2012 codes have been selected. The implementation of AAASHTO LRFD 2012 code includes the 6th Edition of AASHTO LRFD Bridge Design Specification and 2013 Interim Revisions. CSiBridge provides options to design or check moment resisting frames of Zones 1 (low seismic activity), 2, 3, and 4 (high seismic activity) as required for seismic design provisions. The details of the design criteria used for the different seismic zones are described in the following sections. For simplicity, all equations and descriptions presented in this chapter correspond to inch-kips-second units unless otherwise noted.

3.1 Notation

The various notations used in this chapter are described herein:

- $A_{cp}$: Area enclosed by outside perimeter of concrete cross-section, in$^2$
- $A_{cv}$: Area of concrete used to determine shear stress, in$^2$
- $A_g$: Gross area of concrete, in$^2$
- $A_l$: Area of longitudinal torsion reinforcement, in$^2$
$A_o$  Gross area enclosed by shear flow path, $\text{in}^2$

$A_{oh}$  Area enclosed by centerline of the outermost closed transverse torsional reinforcement, $\text{in}^2$

$A_s$  Area of tension reinforcement, $\text{in}^2$

$A'_s$  Area of compression reinforcement, $\text{in}^2$

$A_{s(\text{required})}$  Area of steel required for tension reinforcement, $\text{in}^2$

$A_{st}$  Total area of column longitudinal reinforcement, $\text{in}^2$

$A_{v/s}$  Area of transverse torsion reinforcement (closed stirrups) per unit length of the member, $\text{in}^2/\text{in}$

$A_v$  Area of shear reinforcement, $\text{in}^2$

$A_{v/s}$  Area of shear reinforcement per unit length of the member, $\text{in}^2/\text{in}$

$C_m$  Moment gradient coefficient, dependent upon column curvature, used to calculate moment magnification factor

$E_c$  Modulus of elasticity of concrete, ksi

$E_s$  Modulus of elasticity of reinforcement, assumed as 29,000 ksi

$I_g$  Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, $\text{in}^4$

$I_{se}$  Moment of inertia of reinforcement about centroidal axis of member cross-section, $\text{in}^4$

$K$  Effective length factor

$L$  Clear unsupported length, in

$M_1$  Smaller factored end moment in a column, kip-in

$M_2$  Larger factored end moment in a column, kip-in

$M_b$  Non-sway component of factored end moment, kip-in

$M_c$  Factored moment to be used in design, kip-in

$M_s$  Sway component of factored end moment, kip-in

$M_u$  Factored moment at a section, kip-in
\( M_{u2} \)  Factored moment at a section about 2 axis, kip-in
\( M_{u3} \)  Factored moment at a section about 3 axis, kip-in
\( P_b \)  Axial load capacity at balanced strain conditions, kip
\( P_e \)  Euler buckling load of column, kip
\( P_{max} \)  Maximum axial load strength allowed, kip
\( P_0 \)  Axial load capacity at zero eccentricity, kip
\( P_u \)  Factored axial load at a section, kip
\( V_c \)  Shear force resisted by concrete, kip
\( V_E \)  Shear force caused by earthquake loads, kip
\( V_{D+L} \)  Shear force from span loading, kip
\( V_{max} \)  Maximum permitted total factored shear force at a section, kip
\( V_p \)  Shear force computed from probable moment capacity, kip
\( V_s \)  Shear force resisted by steel, kip
\( V_u \)  Factored shear force at a section, kip
\( a \)  Depth of compression block, in
\( a_b \)  Depth of compression block at balanced condition, in
\( a_{max} \)  Maximum allowed depth of compression block, in
\( b \)  Width of member, in
\( b_f \)  Effective width of flange (T-beam section), in
\( b_w \)  Width of web (T-beam section), in
\( c \)  Depth to neutral axis, in
\( c_b \)  Depth to neutral axis at balanced conditions, in
\( d \)  Distance from compression face to tension reinforcement, in
\( d' \)  Concrete cover to center of reinforcing, in
\( d_s \)  Thickness of slab (T-beam section), in
Concrete Frame Design AASHTO LRFD 2012

- $f'_c$: Specified compressive strength of concrete, ksi
- $f_y$: Specified yield strength of flexural reinforcement, ksi.
- $f_{ys}$: Specified yield strength of shear reinforcement, ksi.
- $h$: Overall depth of a column section, in
- $p_{cp}$: Outside perimeter of the concrete cross-section, in
- $p_h$: Perimeter of centerline of outermost closed transverse torsional reinforcement, in
- $r$: Radius of gyration of column section, in
- $\beta$: Factor indicating the ability of diagonally cracked concrete to transmit tension and shear
- $\beta_l$: Factor for obtaining depth of compression block in concrete
- $\beta_d$: Absolute value of ratio of the maximum factored dead load moment to the maximum factored total load moment
- $\theta$: Angle of inclination of diagonal compressive stresses with the longitudinal axis of beams or columns, degrees
- $\delta_b$: Moment magnification factor for non-sway moments
- $\delta_s$: Moment magnification factor for sway moments
- $\varepsilon_c$: Strain in concrete
- $\varepsilon_{c,\text{max}}$: Maximum usable compression strain allowed in extreme concrete fiber (0.003 in/in)
- $\varepsilon_s$: Strain in reinforcing steel
- $\varepsilon_{s,\text{min}}$: Minimum tensile strain allowed in steel rebar at nominal strength for tension controlled behavior (0.005 in/in)
- $\phi$: Strength reduction factor

### 3.2 Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure is to be checked. There are more types of loads specified in the code than are considered in the current implementation of the
default load combinations. However, the user has full control of the definition of loads and load combinations.

There are six types of dead loads: dead load of structural components and non-structural attachments (DC), downdrag (DD), dead load of wearing surface and utilities (DW), horizontal earth pressure load (EH), vertical earth pressure load (EV), and earth surcharge load (ES). Each type of dead load case requires a separate load factor.

There are six types of live loads: vehicular live load (LL), vehicular dynamic load allowance (IM), vehicular centrifugal force (CE), vehicular braking force (BR), pedestrian live load (PL), and live load surcharge (LS). All of these load cases require the same factor and do not need to be treated separately.

If the structure is subjected to structural dead load (DL), live load (LL), wind load (WL), and earthquake loads (EL), and considering that wind and earthquake forces are reversible, the following default load combinations have been considered for Strength and Extreme Event limit states (AASHTO 3.4.1):

\[
1.50DL \\
1.25DL + 1.75LL \\
1.25DL + 1.75(0.75PLL)
\]

\[
0.9DL + 1.4WL \\
1.25DL + 1.4WL \\
1.25DL + 1.35LL + 0.4WL
\]

\[
0.9DL + 1.0EL \\
1.2DL + 0.5LL + 1.0EL
\]

These are also the default design load combinations in the program whenever the AASHTO LRFD 2012 code is used. The user is expected to define the other load combinations as necessary.

PLL is the live load multiplied by the Pattern Live Load Factor. The Pattern Live Load Factor can be specified in the Preferences.
Live load reduction factors can be applied to the member forces of the live load analysis on a member-by-member basis to reduce the contribution of the live load to the factored loading.

When using the AASHTO LRFD 2012 code, the program design assumes that a P-delta analysis has been performed.

### 3.3 Strength Reduction Factors

The strength reduction factors, $\phi$, are applied on the nominal strength to obtain the design strength provided by a member. The factors for flexure, axial force, shear, and torsion are as follows:

\[
\phi = 0.90 \quad \text{for tension-controlled reinforced concrete section under normal loading,} \quad \text{(AASHTO 5.5.4.2.1)}
\]

\[
\phi = 0.75 \quad \text{for all compression-controlled reinforced concrete sections with spirals or ties used in Seismic Zone 1 or used in any seismic zone for non-seismic load combinations under normal loading,} \quad \text{(AASHTO 5.5.4.2.1)}
\]

\[
\phi = 0.90 \quad \text{for all compression-controlled reinforced concrete sections with spiral or ties used in Seismic Zones 2, 3 or 4 for seismic load combinations (Extreme Events) under normal loading,} \quad \text{(AASHTO 5.5.4.2.1, 5.10.11.3, 5.10.11.4.1b)}
\]

\[
\phi = 0.90 \quad \text{for shear and torsion in normal weight concrete,} \quad \text{(AASHTO 5.5.4.2.1)}
\]

\[
\phi = 0.80 \quad \text{for shear and torsion in light-weight concrete.} \quad \text{(AASHTO 5.5.4.2.1)}
\]

Sections are considered compression controlled when the tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of $\varepsilon_{c,\text{max}}$, which is 0.003. The compression-controlled strain limit is the tensile strain in the reinforcement at balanced strain condition, which is taken as the yield strain of the steel reinforcing, $f_y/E$ (AASHTO C5.5.4.2.1).
Sections are tension-controlled when the tensile strain in the extreme tension steel is equal to or greater than 0.005, just as the concrete in compression reaches its assumed strain limit of 0.03 (AASHTO C.5.5.4.2.1).

Sections with $\varepsilon_t$ between the two limits are considered to be in a transition region between compression-controlled and tension-controlled sections.

When the section is tension controlled, a $\phi$ factor for tension control is used. When the section is compression controlled, a $\phi$ factor for compression control is used. When the section is within the transition region, $\phi$ is linearly interpolated between the two values (AASHTO C5.5.4.2.1).

### 3.4 Column Design/Check

The program can be used to check column capacity or to design columns. If the geometry of the reinforcing bar configuration of each concrete column section has been defined, the program will check the column capacity. Alternatively, the program can calculate the amount of reinforcing required to design the column based on provided reinforcing bar configuration. The reinforcement requirements are calculated or checked at a use-defined number of check/design stations along the column span. The design procedure for the reinforced concrete columns of the structure involves the following steps:

- Generate axial force-biaxial moment interaction surfaces for all of the different concrete section types of the model. A typical biaxial interacting diagram is shown in Figure 3-1. For reinforcement to be designed, the program generates the interaction surfaces for the range of allowable reinforcement ratio $(A_{sn}/A_y)$: 0.135 $f_u/f_y$ to 0.008 for columns in Seismic Zone 1 (AASHTO 5.7.4.2), 0.01 to 0.06 percent for columns in Seismic Zone 2 (AASHTO 5.10.11.3), and 0.01 to 0.04 percent for columns in Seismic Zones 3 and 4 (AASHTO 5.10.11.4.1a).

- Calculate the capacity ratio or the required reinforcing area for the factored axial force and biaxial (or uniaxial) bending moments obtained from each loading combination at each station of the column. The target capacity ratio is taken as the Utilization Factor Limit when calculating the required reinforcing area.

- Design the column shear reinforcement.
The following four sections describe in detail the algorithms associated with this process.

### 3.4.1 Generation of Biaxial Interaction Surfaces

The column capacity interaction volume is numerically described by a series of discrete points that are generated on the three-dimensional interaction failure surface. In addition to axial compression and biaxial bending, the formulation allows for axial tension and biaxial bending considerations. A typical interaction surface is shown in Figure 3-1.

![Figure 3-1 A typical column interaction surface](image)

The coordinates of these points are determined by rotating a plane of linear strain in three dimensions on the section of the column, as shown in Figure 3-2. The linear strain diagram limits the maximum concrete strain, $\varepsilon_c$, at the extremity of the section, to 0.003 (AASHTO 5.7.2.1).
The formulation is based consistently on the general principles of ultimate strength design for Strength and Extreme Event Limit States (AASHTO 5.7.2.1).

The stress in the steel is given by the product of the steel strain and the steel modulus of elasticity, \( \varepsilon E_s \), and is limited to the yield stress of the steel, \( f_y \) (AASHTO 5.7.2.1). The area associated with each reinforcing bar is assumed to be placed at the actual location of the center of the bar, and the algorithm does...
not assume any further simplifications with respect to distributing the area of steel over the cross-section of the column, as shown in Figure 3-2.

The concrete compression stress block is assumed to be rectangular, with a stress value of $0.85f'c$ (AASHTO 5.7.2.2), as shown in Figure 3-3. The interaction algorithm provides correction to account for the concrete area that is displaced by the reinforcement in the compression zone.

![Concrete Section, Strain Diagram, Stress Diagram](image)

Figure 3-3 Idealization of stress and strain distribution in a column section

The depth of the equivalent rectangular block, $a$, is taken as:

$$a = \beta_1 \cdot c$$  \hspace{1cm} (AASHTO 5.7.2.2)

where $c$ is the depth of the stress block in compression strain and,

$$\beta_1 = 0.85 - 0.05 \left( f'c - 4 \right), \quad 0.65 \leq \beta_1 \leq 0.85.$$  \hspace{1cm} (AASHTO 5.7.2.2)

The effect of the strength reduction factor, $\phi$, is included in the generation of the interaction surface. The value of $\phi$ used in the interaction diagram varies from compression controlled $\phi$ to tension controlled $\phi$ based on the maximum tensile strain in the reinforcing at the extreme edge, $\varepsilon_t$ (AASHTO 5.5.4.2.1, C5.5.4.2.1).

Sections are considered compression controlled when the tensile strain in the extreme tension steel is equal to or less than the compression controlled strain.
limit at the time the concrete in compression reaches its assumed strain limit of \( \varepsilon_{c,\text{max}} \), which is 0.003. The compression controlled strain limit is the tensile strain in the reinforcement at balanced strain condition, which is taken as the yield strain of the steel reinforcing, \( f_y / E \) (AASHTO 5.5.4.2.1, C5.5.4.2.1).

Sections are tension controlled when the tensile strain in the extreme tension steel is equal to or greater than 0.005, just as the concrete in compression reaches its assumed strain limit of 0.003 (AASHTO 5.5.4.2.1, C5.5.4.2.1).

Sections with \( \varepsilon_t \) between the two limits are considered to be in a transition region between compression controlled and tension controlled sections.

When the section is tension controlled, a \( \phi \) factor for tension control is used. When the section is compression controlled, a \( \phi \) factor for compression control is used. When the section is within the transition region, \( \phi \) is linearly interpolated between the two values (AASHTO 5.5.4.2.1), as shown in the following:

\[
\phi = \begin{cases} 
\phi_t & \text{if } \varepsilon_t \leq \varepsilon_y \\
\phi_t - (\phi_c - \phi_t) \frac{0.005 - \varepsilon_t}{0.005 - \varepsilon_y} & \text{if } \varepsilon_y < \varepsilon_t \leq 0.005 \\
\phi_c & \text{if } \varepsilon_t \geq 0.005
\end{cases}
\]  

(AASHTO C5.5.4.2.1)

where

\[
\phi_t = \phi \text{ for tension controlled sections, which is } 0.90 
\]  

(AASHTO 5.5.4.2.1)

\[
\phi_c = \phi \text{ for compression controlled sections}
\]  

= 0.75 for all column sections with spirals or ties used in Seismic Zone 1 or used in any seismic zone for non-seismic load combinations  

(AASHTO 5.5.4.2.1)

= 0.90 for all column sections with spirals or ties used in Seismic Zones 2, 3, or 4 for seismic load combinations (Extreme Events)  

(AASHTO 5.10.11.3, 5.10.11.4.1b)

The limit of \( f'_{c,t} \) is taken to be 10 ksi for all seismic regions:
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\[ f'_{c} \leq 10 \text{ ksi} \]  
(AASHTO 5.1, 5.4.2.1)

The limit of \( f'_{c} \) is taken to be 75 ksi for all frames:

\[ f_{c} \leq 75 \text{ ksi} \]  
(AASHTO 5.4.3.1)

The maximum compressive axial load is limited to \( \phi P_{n_{\text{max}}} \), where

\[
\phi P_{n_{\text{max}}} = 0.85 \phi [0.85 f'_{c} (A_{g} - A_{st}) + f_{y} A_{st}], \text{ spiral} \]  
(AASHTO 5.7.4.4)

\[
\phi P_{n_{\text{max}}} = 0.80 \phi [0.85 f'_{c} (A_{g} - A_{st}) + f_{y} A_{st}], \text{ tied} \]  
(AASHTO 5.7.4.4)

In calculating \( \phi P_{n_{\text{max}}} \), the \( \phi \) for the compression-control case is used.

### 3.4.2 Calculate Column Capacity Ratio

The column capacity ratio is calculated for each design load combination at each output station of each column. The following steps are involved in calculating the capacity ratio of a particular column for a particular design load combination at a particular location:

- Determine the factored moments and forces from the analysis cases and the specified load combination factors to give \( P_{u}, M_{u2}, \) and \( M_{u3}. \)
- Determine the moment magnification factors for the column moments.
- Apply the moment magnification factors to the factored moments. Determine whether the point, defined by the resulting axial load and biaxial moment set, lies within the interaction volume.

The factored moments and corresponding magnification factors depend on the identification of the individual column as either “sway” or “non-sway.”

The following three sections describe in detail the algorithms associated with that process.

#### 3.4.2.1 Determine Factored Moments and Forces

The loads for a particular design load combination are obtained by applying the corresponding factors to all of the analysis cases, giving \( P_{u}, M_{u2}, \) and \( M_{u3}. \) The factored moments are further increased, by using “Moment Magnification Factors” to allow for stability effects.
3.4.2.2 Determine Moment Magnification Factors

The moment magnification factors are calculated separately for sway (overall stability effect), $\delta_s$, and for non-sway (individual column stability effect), $\delta_b$. Also, the moment magnification factors in the major and minor directions are, in general, different (AASHTO 4.5.3.2, 5.7.4.3).

The moment obtained from analysis is separated into two components: the sway $M_s$ and the non-sway $M_b$ components. The non-sway components, which are identified by “$b$” subscripts, are primarily caused by gravity load. The sway components are identified by the “$s$” subscript. The sway moments are primarily caused by lateral loads and are related to the cause of sidesway.

For individual columns or column-members, the magnified moments about two axes at any station of a column can be obtained as

$$M_c = \delta_b M_b + \delta_s M_s \quad \text{(AASHTO 4.5.3.2.2)}$$

The factor $\delta_s$ is the moment magnification factor for moments causing side-sway. The program takes this factor to be 1 because the component moments $M_s$ and $M_b$ are assumed to be obtained from a "second order elastic (P-\Delta) analysis" (AASHTO C4.5.3.2.2a). For more information about P-\Delta analysis, refer to Appendix A.

For the P-\Delta analysis, the analysis should correspond to a load combination of $(1.25 \text{ dead load} + 1.75 \text{ live load}) / \phi$, where $\phi$ is the resistance factor for axial compression, which is taken as 0.75 for seismic Zone 1 and as 0.9 for Seismic Zones 2, 3, and 4 (AASHTO 5.5.4.2.1). See also White and Hajjar (1991).

The non-sway moment magnification factor, $\delta_b$, associated with the major or minor direction of the column is given by (AASHTO 4.5.3.2.2b)

$$\delta_{bs} = \frac{C_m}{P_u} \geq 1.0 \quad \text{where} \quad C_m = 0.6 + 0.4 \frac{M_{lb}}{M_{2b}}$$

(AASHTO 4.5.3.2.2b-3, 4.5.3.2.2b-6)
$M_{1b}$ and $M_{2b}$ are the moments at the ends of the column, and $M_{2b}$ is numerically larger than $M_{1b}$. $M_{1b}/M_{2b}$ is positive for single curvature bending and negative for double curvature bending. The preceding expression of $C_m$ is valid if there is no transverse load applied between the supports. If transverse load is present on the span, or the length is overwritten, $C_m = 1$. The user can overwrite $C_m$ on an object-by-object basis.

$$\phi_K = 0.75, \text{ stiffness reduction factor for concrete member} \quad \text{(AASHTO 4.5.3.2.2b)}$$

$$P_e = \frac{\pi^2 EI}{(Kl_u)^2} \quad \text{(AASHTO 4.5.3.2.2b-5)}$$

$K$ is conservatively taken as 1; however, the program allows the user to overwrite this value (AASHTO 4.5.3.2.2b, 4.6.2.5, 5.7.4.3).

$l_u$ is the unsupported length of the column for the direction of bending considered. The two unsupported lengths are $l_{22}$ and $l_{33}$, corresponding to instability in the minor and major directions of the object, respectively, as shown in Figure B-1 in Appendix B. These are the lengths between the support points of the object in the corresponding directions. Refer to Appendix B for more information about how the program automatically determines the unsupported lengths. The program allows users to overwrite the unsupported length ratios, which are the ratios of the unsupported lengths for the major and minor axes bending to the overall member length.

$EI$ is associated with a particular column direction:

$$EI = \frac{0.4E_c I_g}{1 + \beta_d} \quad \text{(AASHTO 5.7.4.3)}$$

$$\beta_d = \frac{\text{maximum factored sustained (dead) moment}}{\text{maximum factored total moment}} \quad \text{(AASHTO 5.7.4.3)}$$

The magnification factor, $\delta_b$, must be a positive number and greater than one. Therefore, $P_u$ must be less than $\phi_k P_e$. If $P_u$ is found to be greater than or equal to $\phi_k P_e$, a failure condition is declared.
The preceding calculations are performed for major and minor directions separately. That means that $\delta_n$, $\delta_b$, $C_m$, $K$, $l_u$, $EI$, and $P_c$ assume different values for major and minor directions of bending.

If the program assumptions are not satisfactory for a particular member, the user can explicitly specify values of $\delta_n$ and $\delta_b$.

### 3.4.2.3 Determine Capacity Ratio

As a measure of the stress condition of the column, a capacity ratio is calculated. The capacity ratio is basically a factor that gives an indication of the stress condition of the column with respect to the capacity of the column.

Before entering the interaction diagram to check the column capacity, the moment magnification factors are applied to the factored loads to obtain $P_u$, $M_{u2}$, and $M_{u3}$. The point $(P_u, M_{u2}, M_{u3})$ is then placed in the interaction space shown as point $L$ in Figure 3-4. If the point lies within the interaction volume, the column capacity is adequate. However, if the point lies outside the interaction volume, the column is overstressed.

This capacity ratio is achieved by plotting the point $L$ and determining the location of point $C$. Point $C$ is defined as the point where the line $OL$ (if extended outwards) will intersect the failure surface. This point is determined by three-dimensional linear interpolation between the points that define the failure surface, as shown in Figure 3-4. The capacity ratio, $CR$, is given by the ratio $OL/OC$.

- If $OL = OC$ (or $CR = 1$), the point lies on the interaction surface and the column is stressed to capacity.
- If $OL < OC$ (or $CR < 1$), the point lies within the interaction volume and the column capacity is adequate.
- If $OL > OC$ (or $CR > 1$), the point lies outside the interaction volume and the column is overstressed.
The maximum of all the values of $CR$ calculated from each design load combination is reported for each check station of the column along with the controlling $P_u$, $M_{u2}$, and $M_{u3}$ set and associated design load combination name.

### 3.4.3 Required Reinforcing Area

If the reinforcing area is not defined, the program computes the reinforcement that will give a column capacity ratio equal to the Utilization Factor Limit, which is set to 0.95 by default.
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3.4.4 Design Column Shear Reinforcement

The shear reinforcement is designed for each design combination in the major and minor directions of the column. The following steps are involved in designing the shear reinforcing for a particular column for a particular design load combination resulting from shear forces in a particular direction:

- Determine the factored forces acting on the section, \( P_u \), \( M_u \), and \( V_u \). Note that \( M_u \) and \( P_u \) are needed for the calculation of \( V_c \).

- Determine the shear force, \( V_c \), which can be resisted by concrete alone.

- Calculate the reinforcement steel required to carry the balance.

For moment resisting frames in seismic Zones 2, 3, and 4, the shear design of the columns also is based on the overstrength moment strengths, in addition to the factored shear forces (AASHTO 3.10.9.4.3). Effects of the axial forces on the column moment capacities are included in the formulation.

For moment resisting frames in seismic Zone 2, the design shear from overstrength moment capacities of the member does not need to be larger than the shear force from a special seismic load combination with earthquake forces doubled (AASHTO 3.10.9.3).

The following three sections describe in detail the algorithms associated with this process.

3.4.4.1 Determine Section Forces

- In the design of the column shear reinforcement of a moment resisting concrete frame in Seismic Zone 1, the forces for a particular design load combination, namely, the column axial force, \( P_u \), the bending moment \( M_u \), and the column shear force, \( V_u \), in a particular direction are obtained by factoring the analysis cases with the corresponding design load combination factors.

- In the shear design of moment resisting frames in Seismic Zones 3 and 4 (seismic design), the shear capacity of the column is checked for capacity shear in addition to the requirement for the moment resisting frames in Zone 1. The capacity shear force in the column, \( V_u \), is determined from consideration of the maximum shear force that can be generated at the column. One capacity shear is calculated for each direction (major and minor).
In calculating the capacity shear of the column, \( V_{c}^{e} \), the overstrength moment resistance at the two ends of the column is calculated for the existing factored axial load (AASHTO 3.10.9.4.3). Clockwise rotation of the joint at one end and the associated counter-clockwise rotation of the other joint produces one shear force. The reverse situation produces another capacity shear force, and both of those situations are checked, with the maximum of those two values taken as the \( V_{c}^{e} \).

For each design load combination, the factored axial load, \( P_u \), is calculated. Then, the positive and negative overstrength moment resistances, \( M_{r}^{+} \) and \( M_{r}^{-} \), of the column in a particular direction under the influence of the axial force \( P_u \) are calculated using the uniaxial interaction diagram in the corresponding direction. Then the capacity shear force is obtained by applying the calculated overstrength moment resistances at the two ends of the column acting in two opposite directions. Therefore, \( V_{c}^{e} \) is the maximum of \( V_{c1}^{e} \) and \( V_{c2}^{e} \),

\[
V_{c}^{e} = \max \left\{ V_{c1}^{e}, V_{c2}^{e} \right\} + V_{D+L}
\]  
(AASHTO 3.10.9.4.3)

where,

\[
V_{c1}^{e} = \frac{M_{r}^{-} + M_{r}^{+}}{L},
\]  
(AASHTO 3.10.9.4.3)

\[
V_{c2}^{e} = \frac{M_{r}^{+} + M_{r}^{-}}{L},
\]  
(AASHTO 3.10.9.4.3)

\( M_{r}^{+}, M_{r}^{-} \) = Positive and negative probable maximum moment capacities \( \left( M_{r}^{+}, M_{r}^{-} \right) \) at end \( I \) of the column obtained by multiplying the nominal resistance by 1.3,

\( M_{r}^{+}, M_{r}^{-} \) = Positive and negative probable maximum moment capacities \( \left( M_{r}^{+}, M_{r}^{-} \right) \) at end \( J \) of the column obtained by multiplying the nominal resistance by 1.3, and

\( L \) = Clear span of the column.

\( V_{D+L} \) is the contribution of shear force from the in-span distribution of gravity loads. For most of the columns, it is zero. See also Table 3-1 for details.
If the column section was identified as a section to be checked, the user-specified reinforcing is used for the interaction curve. If the column section was identified as a section to be designed, the reinforcing area envelope is calculated after completing the flexural \((P-M-M)\) design of the column. This envelope of reinforcing area is used for the interaction curve.

If the column section is a variable (non-prismatic) section, the cross-sections at the two ends are used, along with the user-specified reinforcing or the envelope of reinforcing for check or design sections, as appropriate. If the user overwrites the length factor, the full span length is used. However, if the length factor is not overwritten by the user, the clear span length will be used.

- In the shear design of moment resisting frames in Seismic Zone 2, the shear capacity of the column also is checked for the capacity shear based on the overstrength moment capacities at the ends and the factored gravity loads, in addition to the check required for moment resisting frames in Seismic Zone 1. The design shear force is taken to be the minimum of that based on the overstrength \((\phi = 1.3)\) moment resistance and the modified factored shear force.

\[
V_u = \min \left\{ V^c_e, V_{ef} \right\} \geq V_{u,\text{factored}} \quad \text{(AASHTO 3.10.9.3, 3.10.9.4.3)}
\]

where, \(V^c_e\) is the capacity shear force of the column based on the overstrength moment resistance of the column ends alone. The calculation of \(V^c_e\) is the same as that described for moment resisting frames in Seismic Zones 2 and 3.

\(V_{ef}\) is the shear force in the column obtained from the modified design load combinations. In that case, the factored design forces \((P_u, V_u, M_u)\) are based on the specified design load factors, except that the earthquake load factors are doubled (AASHTO C3.10.9.3). When designing for this modified shear force, the modified \(P_u\) and \(M_u\) are used for calculation concrete shear strength. However, the modified \(P_u\) and \(M_u\) are not used for the P-M-M interaction.

In designing for \(V_u\), the factored \(P_u\) and \(M_u\) are used for calculating concrete shear strength. In no case is the column designed for a shear force less than the original factored shear force.
3.4.4.2 Determine Concrete Shear Capacity

Given the design force set \( M_u, P_u, \) and \( V_u, \) the shear force carried by the concrete, \( V_c, \) is calculated as follows:

- For designing moment resisting concrete frames in any seismic zone, \( V_c, \) is set to

\[
V_c = 0.0316\lambda\beta\sqrt{f'_c b_v d_v},
\]

(AASHTO 5.8.3.3-3)

where,

\( \beta \) is a factor indicating the ability of the diagonal cracked concrete to transmit tension and shear. It is a fraction of stress condition and its approximate values range from 0.5 to 6.0 (AASHTO Table B5.2-1, Table B5.2-2). It is determined in accordance with section 5.8.3.4.2 of the code, which is described in this section.

\( \lambda \) is the strength reduction factor to account for low density concrete (AASHTO 5.8.2.2). For normal density concrete, its value is 1, which is the program default value. For concrete using lower density aggregate, the user can change the value of \( \lambda \) in the material properties. The recommended values for \( \lambda \) are as follows (AASHTO 5.8.2.2):

\[
\lambda = \begin{cases} 
1.00, & \text{for normal density concrete,} \\
0.85, & \text{for semi-low density concrete in which all of the fine aggregate is natural sand,} \\
0.75, & \text{for low-density concrete in which none of the fine aggregate is natural sand.}
\end{cases}
\]

\( b_v \) is the width of the cross-section resisting the shear perpendicular to the shear force direction. For columns with rectangular cross-sections, \( b_v \) is taken as the width of the section perpendicular to the shear direction. For columns with circular cross-sections, \( b_v \) is taken as the diameter of the section (AASHTO 5.8.2.9).

\( d_v \) is the effective shear depth. It is taken as the greater of 0.9\( d \) or 0.72\( h \).

\[
d_v = \max\{0.9d, 0.72h\}
\]

(AASHTO 5.8.2.9)
where $d$ is the distance from the extreme compression fiber to the centroid of tension reinforcement, and $h$ is the overall depth of the cross-section in the direction of shear force.

- For designing moment resisting concrete frames in seismic Zones 2, 3, and 4, $V_c$ is taken as that given earlier (AASHTO 5.8.3.3) if compressive axial force is high $P_u > 0.1f'_cA_g$ (AASHTO 5.10.11.4.1c). $V_c$ is taken as zero if axial force is tensile. $V_c$ is linearly interpolated between zero and that given by AASHTO 5.8.3.3, if the factored axial compressive force, $P_u$, including the earthquake effect is small $\left( P_u \leq 0.1f'_cA_g \right)$ (AASHTO 5.10.11.4.1c). This provision is applied to all locations of the column irrespective of whether it is in the end region or not.

The procedure for determining shear resistance rests on the determination of the $\beta$ factor. For column shear design, the program uses the general method of the code (AASHTO 5.8.3.4.2).

- When the section contains at least the minimum transverse reinforcement, $\beta$ is computed as follows (AASHTO 5.8.3.4.2-1).

\[
\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \quad \text{(AASHTO 5.8.3.4.2-1)}
\]

- When the section contains no transverse reinforcement, $\beta$ is computed as follows (AASHTO 5.8.3.4.2-2).

\[
\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \cdot \frac{51}{(39 + S_{xe})} \quad \text{(AASHTO 5.8.3.4.2-2)}
\]

- The value of $\theta$ in both cases is specified as:

\[
\theta = 29 + 3500\varepsilon_s \quad \text{(AASHTO 5.8.3.4.2-3)}
\]

- In the preceding expression, the equivalent crack spacing parameter, $S_{xe}$, is determined as follows:

\[
S_{xe} = \frac{S_x}{a_g + 0.63} \quad \text{(AASHTO 5.8.3.4.2-5)}
\]

where,
In the preceding expression, the crack spacing parameter, $S_x$, shall be taken as the minimum of $d_v$ and the maximum distance between layers of distributed longitudinal reinforcement. However, $S_x$ is conservatively taken as equal to $d_v$ (AASHTO 5.8.3.4.2).

$$S_x = d_v \quad \text{(AASHTO 5.8.3.4.2)}$$

$a_g =$ maximum aggregate size in inches. Its value is assumed to be 0.75" by default.

- The longitudinal strain, $\varepsilon_s$, at mid-depth of the cross-section is computed from the following equation:

$$\varepsilon_s = \frac{|M_u|/d_v + 0.5N_u + |V_u|}{E_s A_s} \quad \text{(AASHTO 5.8.3.4.2-4)}$$

$\varepsilon_s$ is positive for tensile action.

$N_u$ is positive for tensile action.

In evaluating $\varepsilon_s$, the following conditions apply:

- $V_u$ and $M_u$ are taken as positive quantities and $M_u$ is taken as at least $|V_u|/d_v$. \hfill (AASHTO 5.8.3.4.2-4)

- $A_s$ is taken as the total area of longitudinal reinforcement in the column section. For the column section check option, the program uses the sum of user-defined reinforcement in the section. For the column section design option, the longitudinal reinforcement area is taken as the envelope of reinforcement required for all design load combinations. Actual provided reinforcement might be slightly higher than this quantity. The reinforcement should be developed to achieve full strength (AASHTO 5.8.3.4.2).

- If the value of $\varepsilon_s$ calculated from the preceding equation is negative, it is recalculated as follows:

$$\varepsilon_s = \frac{|M_u|/d_v + 0.5N_u + |V_u|}{2\left(E_s A_s + E_{ct} A_{ct}\right)} \geq -0.0004 \quad \text{(AASHTO 5.8.3.4.2)}$$
For sections closer than $d_v$ from the face of the support, $\varepsilon$ is calculated based on $M_u$, $V_u$, and $N_u$ at a section at a distance $d_v$ from the face of the support (AASHTO 5.8.3.4.2).

- If the axial tension is large enough to crack the flexural compression face of the section, the value of $\varepsilon$ is increased by a factor of 2 (AASHTO 5.8.3.4.2). The program uses a linear elastic stress distribution to check the condition.

- An upper limit on $\varepsilon$ is imposed as follows:

$$\varepsilon_s \leq 0.006$$

(AASHTO 5.8.3.4.2)

The shear strength of the section due to concrete, $V_c$, depends on whether the minimum transverse reinforcement is provided. To check this condition, the program performs the design in two passes. In the first pass, it is assumed that no transverse shear reinforcement is needed. When the program determines that shear reinforcement is needed, the program performs the second pass with the assumption that at least minimum shear reinforcement is provided.

### 3.4.4.3 Determine Required Shear Reinforcement

Given $V_u$ and $V_c$, the required shear reinforcement in the form of stirrups or ties within a spacing, $s$, is given for rectangular and circular columns by the following:

- The shear force is limited to the following upper limit:

$$V_{\text{max}} = 0.25 f'_c b_d$$

(AASHTO 5.8.3.3-2)

- The required shear reinforcement per unit spacing, $A_v/s$, is calculated as follows:

If $V_u \leq \phi(V_c/2)$,

$$\frac{A_v}{s} = 0,$$

(AASHTO 5.8.2.4)

else if $V_u \leq \phi V_{\text{max}},$
\[
\frac{A_c}{s} = \frac{(V_u - \phi V_v)}{\phi f_{ys} d_y \cot \theta},
\]
(AASHTO 5.8.3.3-4)

\[
\frac{A_c}{s} \geq 0.0316 \lambda \sqrt{f'_c b_w} \over f_{ys}
\]
(AASHTO 5.8.2.5-1)

else if \( V_u > \phi V_{\text{max}} \),

a failure condition is declared. (AASHTO 5.8.3.3)

Here \( \theta \) is an angle of inclination of diagonal compressive stresses. It is a function of current stress condition. It is computed following a procedure described above (AASHTO 5.8.3.4.2-3). Here the value of \( \phi \), the strength reduction factor, is 0.90 (AASHTO 5.5.4.2.1).

If \( V_u \) exceeds its maximum permitted value \( \phi V_{\text{max}} \), the concrete section size should be increased (AASHTO 5.8.3.3).

The limit of \( f'_{ys} \) is taken to be 60 ksi for all frames:

\[ f_{ys} \leq 60 \text{ ksi} \]  
(AASHTO 5.8.2.8)

The limit of \( f'_c \) is taken to be 10 ksi for all seismic regions:

\[ f'_c \leq 10 \text{ ksi} \]  
(AASHTO 5.1, 5.4.2.1)

For all columns and at any station, the minimum area of transverse circular hoop reinforcement is imposed as follows:

\[
\frac{A_x}{s} \geq 0.45 \left[ \frac{A_c}{A_c} - 1 \right] \frac{f'_c h_{\text{core}}}{f_{ys}} 4
\]
(AASHTO 5.7.4.6-1)

In potential plastic hinge locations, as described later, of seismic moment resisting frames in Zones 2, 3, and 4, the minimum area of circular hoops and transverse stirrups is imposed as follows:

\[
\frac{A_x}{s} \geq 0.12 \frac{f'_c h_{\text{core}}}{f_{ys}} 4
\]  
(Hoops)  
(AASHTO 5.10.11.4.1d-1)
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\[ \frac{A_s}{s} \geq 0.30 \left[ \frac{A_e}{A_{ec}} - 1 \right] \left( f'c \right)_{\text{core}} \]  
(Stirrups) \hspace{1cm} \text{(AASHTO 5.10.11.4.1d-2)}

\[ \frac{A_s}{s} \geq 0.12 \left( f'c \right)_{\text{core}} \]  
(Stirrups) \hspace{1cm} \text{(AASHTO 5.10.11.4.1d-3)}

For the definition of the potential plastic hinge, it is assumed in the current version of the program that any beam and column segment near the joint is a potential plastic hinge. The length of the plastic hinge, \( L_{\text{hinge}} \), in a column is taken as follows:

\[ L_{\text{hinge}} = \max \left\{ h, b, \frac{l}{6}, 18'' \right\} \]  
(AASHTO 5.10.11.4.1c)

The maximum of all the calculated \( A_s/s \) values, obtained from each design load combination, is reported for the major and minor directions of the column, along with the controlling combination name.

The column shear reinforcement requirements reported by the program are based purely on shear strength consideration. Any minimum stirrup requirements to satisfy spacing considerations or transverse reinforcement volumetric considerations must be investigated independently of the program by the user.

3.5 Beam Design

In the design of concrete beams, the program calculates and reports the required areas of steel for flexure and shear based on the beam moments, shear forces, torsions, design load combination factors, and other criteria described in the text that follows. The reinforcement requirements are calculated at a user-defined number of check/design stations along the beam span.

All beams are designed for major direction flexure, shear and torsion only. Effects resulting from any axial forces and minor direction bending that may exist in the beams must be investigated independently by the user.

The beam design procedure involves the following steps:

- Design flexural reinforcement
- Design shear reinforcement
• Design torsion reinforcement

3.5.1 Design Beam Flexural Reinforcement

The beam top and bottom flexural steel is designed at check/design stations along the beam span. The following steps are involved in designing the flexural reinforcement for the major moment for a particular beam for a particular section:

• Determine the maximum factored moments
• Determine the reinforcing steel

3.5.1.1 Determine Factored Moments

In the design of flexural reinforcement of concrete beams, the factored moments for each design load combination at a particular beam section are obtained by factoring the corresponding moments for different analysis cases with the corresponding design load combination factors.

The beam section is then designed for the factored moments obtained from all of the design load combinations. Positive moments produce bottom steel. In such cases, the beam may be designed as a Rectangular or a T beam. Negative moments produce top steel. In such cases, the beam is always designed as a rectangular section.

3.5.1.2 Determine Required Flexural Reinforcement

In the flexural reinforcement design process, the program calculates both the tension and compression reinforcement. Compression reinforcement is added when the applied design moment exceeds the maximum moment capacity of a singly reinforced section. The user has the option of avoiding the compression reinforcement by increasing the effective depth, the width, or the grade of concrete.

The design procedure is based on the simplified rectangular stress block, as shown in Figure 3-5 (AASHTO 5.7). Furthermore, it is assumed that the net tensile strain of the reinforcing steel shall not be less than 0.005 (tension controlled) (AASHTO 5.5.4.2.1). When the applied moment exceeds the moment capacity at this design condition, the area of compression reinforcement is calculated on
the assumption that the additional moment will be carried by compression and additional tension reinforcement.

In designing the beam flexural reinforcement, the following limits are imposed on the steel tensile strength and the concrete compressive strength:

\[ f_y \leq 75 \text{ ksi} \quad \text{(AASHTO 5.4.3.1)} \]
\[ f'_c \leq 10 \text{ ksi} \quad \text{(AASHTO 5.1, 5.4.2.1)} \]

The design procedure used by the program for both rectangular and flanged sections (T beams) is summarized in the following subsections. It is assumed that the design ultimate axial force is small; hence, all of the beams are designed ignoring axial force.

### 3.5.1.2.1 Design for Rectangular Beam

In designing for a factored negative or positive moment, \( M_o \) (i.e., designing top or bottom steel), the depth of the compression block is given by \( a \) (see Figure 3-5), where,

\[
a = d - \sqrt{d^2 - \frac{2|M_o|}{0.85f'_c\phi b}}, \quad \text{(AASHTO 5.7.3.2.5)}
\]

where, the value \( \phi \) is taken as that for a tension-controlled section, which is 0.90 (AASHTO 5.5.4.2.1) in the preceding and the following equations.
The maximum depth of the compression zone, \( c_{\text{max}} \), is calculated based on the limitation that the tensile steel tension shall not be less than \( \varepsilon_{s,\text{min}} \), which is equal to 0.005 for tension-controlled behavior (AASHTO 5.5.4.2.1, 5.7.2.1):

\[
\varepsilon_{c,\text{max}} = \varepsilon_{c,\text{max}} + \varepsilon_{s,\text{min}}
\]

where, (AASHTO C5.5.4.2.1)

\[
\varepsilon_{c,\text{max}} = 0.003
\]

(AASHTO 5.5.4.2.1, 5.7.2.1)

\[
\varepsilon_{s,\text{min}} = 0.005
\]

(AASHTO 5.5.4.2.1, 5.7.2.1)

The maximum allowable depth of the rectangular compression block, \( a_{\text{max}} \), is given by

\[
a_{\text{max}} = \beta_1 c_{\text{max}}
\]

(AASHTO 5.7.2.2)

where \( \beta_1 \) is calculated as follows:

\[
\beta_1 = 0.85 - 0.05 \left( f_c' - 4 \right), \quad 0.65 \leq \beta_1 \leq 0.85
\]

(AASHTO 5.7.2.2)
If $a \leq a_{\text{max}}$ (AASHTO 5.5.4.2.1, 5.7.2.1), the area of tensile steel reinforcement is then given by

$$A_t = \frac{M_u}{\phi f_y \left( d - \frac{a}{2} \right)}$$

This steel is to be placed at the bottom if $M_u$ is positive, or at the top if $M_u$ is negative.

If $a > a_{\text{max}}$, compression reinforcement is required (AASHTO 5.5.4.2.1, 5.7.2.1) and is calculated as follows:

The compressive force developed in concrete alone is given by

$$C = 0.85 f'_c b a_{\text{max}}, \quad \text{(AASHTO 5.7.2.2)}$$

the moment resisted by concrete compression and tensile steel is

$$M_{uc} = C \left( d - \frac{a_{\text{max}}}{2} \right) \phi.$$ 

Therefore, the moment resisted by compression steel and tensile steel is

$$M_{ac} = M_{\text{u}} - M_{uc}.$$ 

So the required compression steel is given by

$$A'_c = \frac{M_{uc}}{(f'_s - 0.85 f'_c)(d - d')} \phi,$$

where

$$f'_s = E_s \varepsilon_{c,\text{max}} \left[ \frac{c_{\text{max}} - d'}{c_{\text{max}}} \right] \leq f_y. \quad \text{(AASHTO 5.7.2.1, 5.7.3.2.5)}$$

The required tensile steel for balancing the compression in concrete is

$$A_{ts} = \frac{M_{uc}}{f_y \left[ d - \frac{a_{\text{max}}}{2} \right] \phi},$$
the tensile steel for balancing the compression in steel is given by

\[ A_{s2} = \frac{M_{uw}}{f_y (d - d') \phi} \]

Therefore, the total tensile reinforcement is \( A_s = A_{s1} + A_{s2} \), and the total compression reinforcement is \( A_s' \). \( A_s \) is to be placed at the bottom and \( A_s' \) is to be placed at the top if \( M_u \) is positive, and \( A_s' \) is to be placed at the bottom and \( A_s \) is to be placed at the top if \( M_u \) is negative.

3.5.1.2.2 Design for T-Beam

In designing a T-beam, a simplified stress block, as shown in Figure 3-6, is assumed if the flange is under compression, i.e., if the moment is positive. If the moment is negative, the flange comes under tension, and the flange is ignored. In that case, a simplified stress block similar to that shown in Figure 3-5 is assumed in the compression side (AASHTO 5.7.2.2, 5.7.3.2.5).

Flanged Beam Under Negative Moment

In designing for a factored negative moment, \( M_u \) (i.e., designing top steel), the calculation of the steel area is exactly the same as described for a rectangular beam, i.e., no T-beam data is used.

Flanged Beam Under Positive Moment

If \( M_u > 0 \), the depth of the compression block is given by

\[ a = d - \sqrt{d^2 - \frac{2M_u}{0.85 f'c' \phi b_f}} \]  \hspace{1cm} \text{(AASHTO 5.7.2.2, 5.7.3.2.5)}

where, the value of \( \phi \) is taken as that for a tension controlled section, which is 0.90 (AASHTO 5.5.4.2.1) in the preceding and the following equations.

The maximum depth of the compression zone, \( c_{\text{max}} \), is calculated based on the limitation that the tensile steel tension shall not be less than \( \varepsilon_{s,\text{min}} \), which is equal to 0.005 for tension-controlled behavior (AASHTO 5.5.4.2-1, 5.7.2.1):

\[ c_{\text{max}} = \frac{\varepsilon_{c,\text{max}}}{\varepsilon_{c,\text{max}} + \varepsilon_{s,\text{min}}} d \]  \hspace{1cm} \text{(AASHTO C5.5.4.2.1)}

where

\[ \varepsilon_{c,\text{max}} = 0.003 \]  \hspace{1cm} \text{(AASHTO 5.5.4.2.1, 5.7.2.1)}
The maximum allowable depth of the rectangular compression block, $a_{\text{max}}$, is given by

$$a_{\text{max}} = \beta_1 c_{\text{max}}$$  \hfill (AASHTO 5.7.2.2)

where $\beta_1$ is calculated as follows:

$$\beta_1 = 0.85 - 0.05 \left( \frac{f'_c}{4} - 0.65 \right), \quad 0.65 \leq \beta_1 \leq 0.85 \quad \hfill (\text{AASHTO 5.7.2.2})$$

- If $a \leq d_s$, the subsequent calculations for $A_s$ are exactly the same as previously defined for the Rectangular section design. However, in that case, the width of the beam is taken as $b_f$, as shown in Figure 3-6. Compression reinforcement is required if $a > a_{\text{max}}$.

- If $a > d_s$, the calculation for $A_s$ has two parts. The first part is for balancing the compressive force from the flange, $C_f$, and the second part is for balancing the compressive force from the web, $C_w$, as shown in Figure 3-6. $C_f$ is given by

$$C_f = 0.85 f'_c \left( b_f - b_w \right) \cdot \min\left(d_s, a_{\text{max}}\right) \quad \hfill (\text{AASHTO 5.7.2.2, 5.7.3.2.5})$$

Therefore, $A_{s1} = \frac{C_f}{f_y}$ and the portion of $M_u$ that is resisted by the flange is given by

$$M_{uf} = C_f \left( d - \frac{\min(d_s, a_{\text{max}})}{2} \right) \phi$$
Again the value of $\phi$ is 0.9. Therefore, the balance of the moment, $M_u$, to be carried by the web is given by

$$M_{aw} = M_u - M_{wf}$$

The web is a rectangular section of dimensions $b_w$ and $d$, for which the design depth of the compression block is recalculated as

$$a_t = d - \sqrt{d^2 - \frac{2M_{aw}}{0.85f'_c \phi b_w}}$$  \hspace{1cm} (AASHTO 5.7.2.2, 5.7.3.2.5)

- If $a_t \leq a_{max}$ (AASHTO 5.5.4.2.1, 5.7.2.1), the area of tensile steel reinforcement is then given by

$$A_{t2} = \frac{M_{aw}}{\phi f_y (d - a_t/2)}$$, and

$$A_s = A_{s1} + A_{s2}$$

This steel is to be placed at the bottom of the T-beam.

- If $a_t > a_{max}$, compression reinforcement is required (AASHTO 5.5.4.2.1, 5.7.2.1) and is calculated as follows:
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The compression force in the web concrete alone is given by

\[ C = 0.85 f' b_n a_{max} \]  

(Applying AASHTO 5.7.2.2)

Therefore the moment resisted by the concrete web and tensile steel is

\[ M_{uc} = C \left( d - \frac{a_{max}}{2} \right) \phi, \text{ and} \]

The moment resisted by compression steel and tensile steel is

\[ M_{us} = M_{uw} - M_{uc} \]

Therefore, the compression steel is computed as

\[ A'_s = \frac{M_{us}}{(f'_s - 0.85 f'_t) (d - d') \phi}, \text{ where} \]

\[ f'_s = E_s \epsilon_{c, max} \left[ \frac{c_{max} - d'}{c_{max}} \right] \leq f_y \]  

(Applying AASHTO 5.7.2.1, 5.7.3.2.5)

The tensile steel for balancing compression in the web concrete is

\[ A_{s2} = \frac{M_{uc}}{f_y \left[ d - \frac{a_{max}}{2} \right] \phi}, \text{ and} \]

the tensile steel for balancing the compression steel is

\[ A_{s3} = \frac{M_{us}}{f_y (d - d') \phi} \]

The total tensile reinforcement is \( A_s = A_{s1} + A_{s2} + A_{s3} \), and the total compression reinforcement is \( A_s' \). \( A_s \) is to be placed at the bottom and \( A_s' \) is to be placed at the top.

3.5.1.2.3 Minimum and Maximum Tensile Reinforcement

The minimum flexural tensile steel required in a beam section is given by:
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\[ A_{x,\text{min}} = \frac{1.30b_wh}{2(b_w + h)f_y} = \frac{b_w}{12} \]  
(AASHTO 5.10.8)

with a limit of

\[ 0.11 \leq \frac{1.30b_wh}{2(b_w + h)f_y} \leq 0.6 \]  
(AASHTO 5.10.8)

The minimum flexural tensile steel provided to prevent premature flexural failure in a rectangular section is given by:

\[ \phi M_n \geq \min\{1.33M_u, M_{cr}\} \]  
(AASHTO 5.7.3.3.2)

An upper limit of 0.04 times the gross web area on both the tension reinforcement and the compression reinforcement is imposed as follows:

\[ A_v \leq \begin{cases} 
0.04bd & \text{Rectangular Beam} \\
0.04b_wd & \text{T-Beam} 
\end{cases} \]

\[ A'_v \leq \begin{cases} 
0.04bd & \text{Rectangular Beam} \\
0.04b_wd & \text{T-Beam} 
\end{cases} \]

3.5.2 Design Beam Shear Reinforcement

The shear reinforcement is designed for each design load combination at a user-defined number of stations along the beam span. The following steps are involved in designing the shear reinforcement for a particular station because of beam major shear:

- Determine the factored shear force, \( V_u \).
- Determine the shear force, \( V_s \), that can be resisted by the concrete.
- Determine the reinforcement steel required to carry the balance.

For moment resisting frames in Seismic Zones 2, 3, and 4, the shear design of the beams is also overstrength moment capacities of the members in addition to the factored shear forces (AASHTO 3.10.9.4.3). Effects of axial forces on the beam shear design are neglected.
For moment resisting frames in Seismic Zone 2, the design shear from overstrength moment capacities of the member does not need to be larger than the shear force from a special seismic load combination with earthquake forces doubled (AASHTO 3.10.9.3).

The following three sections describe in detail the algorithms associated with this process.

### 3.5.2.1 Determine Shear Force and Moment

- In the design of the beam shear reinforcement of a moment resisting concrete frame in Seismic Zone 1, the shear forces and moments for a particular design load combination at a particular beam section are obtained by factoring the associated shear forces and moments with the corresponding design load combination factors.

- In the shear design of moment resisting concrete frames in Seismic Zones 3 and 4 (i.e., seismic design), the shear capacity of the beam is also checked for the capacity shear resulting from the overstrength moment capacities at the ends along with the factored gravity load. This check is performed in addition to the design check required for moment resisting frames in Zone 1. The capacity shear force, $V_p$, is calculated from the overstrength moment capacities of each end of the beam and the gravity shear forces. The procedure for calculating the design shear force in a beam from the overstrength capacity is the same as that described for a column earlier in this chapter. See Table 3-1 for a summary.

The design shear force is then given by (AASHTO 3.10.9.4.3):

$$V_u = \max\{V_{e1}, V_{e2}\}$$

$$V_{e1} = V_{p1} + V_{D+L}$$

$$V_{e2} = V_{p2} + V_{D+L}$$

where $V_p$ is the capacity shear force obtained by applying the calculated overstrength ultimate moment capacities at the two ends of the beams acting in two opposite directions. Therefore, $V_p$ is the maximum of $V_{p1}$ and $V_{p2}$, where
\[ V_{p1} = \frac{M_I^- + M_J^+}{L} \], and
\[ V_{p2} = \frac{M_J^+ + M_I^-}{L} \], where

\[ M_I^- = \text{Moment capacity at end I, with top steel in tension, obtained by multiplying the nominal resistance by 1.3.} \]

\[ M_J^+ = \text{Moment capacity at end J, with bottom steel in tension, obtained by multiplying the nominal resistance by 1.3.} \]

\[ M_I^+ = \text{Moment capacity at end I, with bottom steel in tension, obtained by multiplying the nominal resistance by 1.3.} \]

\[ M_J^- = \text{Moment capacity at end J, with top steel in tension, obtained by multiplying the nominal resistance by 1.3.} \]

\[ L = \text{Clear span of beam.} \]

The moment strengths are obtained by multiplying the nominal resistance by 1.3 (AASHTO 3.10.9.4). If the reinforcement area has not been overwritten for beams, the value of the reinforcing area envelope is calculated after completing the flexural design of the beam for all the design load combinations. Then this enveloping reinforcing area is used in calculating the moment capacity of the beam. If the reinforcing area has been overwritten for beams, this area is used in calculating the moment capacity of the beam. If the beam section is a variable cross-section, the cross-sections at the two ends are used along with the user-specified reinforcing or the envelope of reinforcing, as appropriate. If the user overwrites the major direction length factor, the full span length is used. However, if the length factor is not overwritten, the clear length will be used. In the latter case, the maximum of the negative and positive moment capacities will be used for both the negative and positive moment capacities in determining the capacity shear.

\[ V_{D+L} \] is the contribution of shear force from the in-span distribution of gravity loads with the assumption that the ends are simply supported.
For moment resisting frames in Seismic Zone 2, the shear capacity of the beam also is checked for the capacity shear based on the overstrength moment capacities at the ends along with the factored gravity loads, in addition to the check required for moment resisting frames in Seismic Zone 1. The design shear force in beams is taken to be the minimum of that based on the overstrength moment capacity ($\phi = 1.3$) and modified factored shear force.

$$V_u = \min\{V_e, V_{ef}\} \geq V_{u,\text{factored}} \quad \text{(AASHTO 3.10.9.3, 3.10.9.4.3)}$$

where, $V_e$ is the capacity shear force in the beam determined from the overstrength moment capacities of the beam (AASHTO 3.10.9.4.3). The calculation of $V_e$ is the same as that described for moment resisting frames.

$V_{ef}$ is the shear force in the beam obtained from the modified design load combinations. In that case, the factored design forces ($P_u, V_u, M_u$) are based on the specified design load factors, except that the earthquake factors are doubled (AASHTO C3.10.9.3). In no case is the beam designed for a shear force less than the original factored shear force.

The computation of the design shear force in a beam of a moment resisting frame in Seismic Zone 2 is the same as described for columns earlier in this chapter. See Table 3-1 for a summary.

### 3.5.2.2 Determine Concrete Shear Capacity

Given the design force set $M_u, P_u$ and $V_u$, the shear force carried by the concrete, $V_c$, is calculated as follows. This procedure is exactly the same as that for columns, except that for beams, $P_u$ is neglected.

- For designing moment resisting concrete frames in any seismic zone, $V_c$, is set to

$$V_c = 0.0316\lambda\beta\sqrt{f'_c b_n d_n} \quad \text{(AASHTO 5.8.3.3-3)}$$

where,

$\beta$ is a factor indicating the ability of the diagonal cracked concrete to transmit tension and shear. It is a faction of stress condition and its approximate values range from 0.5 to 6.0 (AASHTO Table B5.2-1, Table B5.2-
2). It is determined in accordance with section 5.8.3.4.2 of the code, which is described in this section.

\( \lambda \) is the strength reduction factor to account for low density concrete (AASHTO 5.8.2.2). For normal density concrete, its value is 1, which is the program default value. For concrete using lower density aggregate, the user can change the value of \( \lambda \) in the material properties. The recommended values for \( \lambda \) are as follows (AASHTO 5.8.2.2):

\[
\lambda = \begin{cases} 
1.00, & \text{for normal density concrete,} \\
0.85, & \text{for semi-low density concrete in which all of the fine aggregate is natural sand,} \\
0.75, & \text{for low-density concrete in which none of the fine aggregate is natural sand.}
\end{cases}
\]

\( b_w \) is the effective web width. For rectangular beams, it is the width of the beam. For T-beams, it is the width of the web of the beam (AASHTO 5.8.2.9).

\( d_v \) is the effective shear depth. It is taken as the greater of 0.9\( d \) or 0.72\( h \).

\[
d_v = \max\{0.9d, 0.72h\} 
\]

(AASHTO 5.8.2.9)

where \( d \) is the distance from the extreme compression fiber to the centroid of tension reinforcement, and \( h \) is the beam.

The procedure for determining shear resistance rests on the determination of the \( \beta \) factor. For beam shear design, the program uses the general method of the code (AASHTO 5.8.3.4.2).

- When the section contains at least the minimum transverse reinforcement, \( \beta \) is computed as follows (AASHTO 5.8.3.4.2-1).

\[
\beta = \frac{4.8}{(1 + 750e_v)} 
\]

(AASHTO 5.8.3.4.2-1)

- When the section contains no transverse reinforcement, \( \beta \) is computed as follows (AASHTO 5.8.3.4.2-2).
The value of $\theta$ in both cases is specified as:

$$\theta = 29 + 3500\varepsilon_s$$  \hspace{1cm} (AASHTO 5.8.3.4.2-3)

In the preceding expression, the equivalent crack spacing parameter, $S_{xe}$, is determine as follows:

$$S_{xe} = S_v \frac{1.38}{a_g + 0.63}$$ \hspace{1cm} (AASHTO 5.8.3.4.2-5)

where,

$$12.0 \text{ in.} \leq S_{xe} \leq 80.0 \text{ in.}$$ \hspace{1cm} (AASHTO 5.8.3.4.2)

In the preceding expression, the crack spacing parameter, $S_v$, shall be taken as the minimum of $d_v$ and the maximum distance between layers of distributed longitudinal reinforcement. However, $S_v$ is conservatively taken as equal to $d_v$. (AASHTO 5.8.3.4.2).

$$S_v = d_v$$ \hspace{1cm} (AASHTO 5.8.3.4.2)

$a_g =$ maximum aggregate size in inches. Its value is assumed to be 0.75" by default.

The longitudinal strain, $\varepsilon_s$, at mid-depth of the cross-section is computed from the following equation:

$$\varepsilon_s = \frac{\left| M_s \right|/d_v + 0.5N_u + |V_u|}{E_s A_s}$$ \hspace{1cm} (AASHTO 5.8.3.4.2-4)

$\varepsilon_s$ is positive for tensile action.

$N_u$ is positive for tensile action. For beams, $N_u = 0$.

In evaluating $\varepsilon_s$, the following conditions apply:

- $V_u$ and $M_u$ are taken as positive quantities and $M_u$ is taken as at least $|V_u|/d_v$. (AASHTO 5.8.3.4.2-4)
- $A_s$ is taken as the total area of longitudinal reinforcement in the beam section. It is taken as the envelope of reinforcement required for all design load combinations. Actual provided reinforcement might be slightly higher than this quantity. The reinforcement should be developed to achieve full strength (AASHTO 5.8.3.4.2).

- If the value of $\varepsilon_s$ calculated from the preceding equation is negative, it is recalculated as follows:

$$\varepsilon_s = \frac{|M_u|/d_u + 0.5N_u + |V_u|}{2(E_sA_s + E_cA_{ct})} \geq -0.0004$$  (AASHTO 5.8.3.4.2)

- For sections closer than $d_v$ from the face of the support, $\varepsilon_s$ is calculated based on $M_u$, $V_u$, and $N_u$ at a section at a distance $d_v$ from the face of the support (AASHTO 5.8.3.4.2).

- If the axial tension is large enough to crack the flexural compression face of the section, the value of $\varepsilon_x$ is increased by a factor of 2 (AASHTO 5.8.3.4.2). The program uses a linear elastic stress distribution to check the condition.

- An upper limit on $\varepsilon_s$ is imposed as follows:

$$\varepsilon_s \leq 0.006$$  (AASHTO 5.8.3.4.2)

The shear strength of the section due to concrete, $V_c$, depends on whether the minimum transverse reinforcement is provided. To check this condition, the program performs the design in two passes. In the first pass, it is assumed that no transverse shear reinforcement is needed. When the program determines that shear reinforcement is needed, the program performs the second pass with the assumption that at least minimum shear reinforcement is provided.

### 3.5.2.3 Determine Required Shear Reinforcement

Given $V_u$ and $V_v$, the required shear reinforcement in the form of stirrups within a spacing, $s$, is given for rectangular and T-beams by the following:

- The shear force is limited to the following upper limit:

$$V_{\text{max}} = 0.25f'_{c}b_l d_v$$  (AASHTO 5.8.3.3-2)
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The required shear reinforcement per unit spacing, \( A_v/s \), is calculated as follows:

If \( V_u \leq \phi(V_c/2) \),

\[
\frac{A_v}{s} = 0,
\]

(AASHTO 5.8.2.4)

else if \( V_u \leq \phi V_{\text{max}} \),

\[
\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{ys} d_i \cot \theta}.
\]

(AASHTO 5.8.3.3-4)

\[
\frac{A_v}{s} \geq \frac{0.0316 \lambda \sqrt{f'_{cs} b_w}}{f_{ys}}
\]

(AASHTO 5.8.2.5-1)

else if \( V_u > \phi V_{\text{max}} \),

a failure condition is declared. (AASHTO 5.8.3.3)

Here \( \theta \) is an angle of inclination of diagonal compressive stresses. It is a function of current stress condition. It is computed following a procedure described above (AASHTO 5.8.3.4.2-3). Here the value of \( \phi \), the strength reduction factor, is 0.90 (AASHTO 5.5.4.2.1).

If \( V_u \) exceeds its maximum permitted value \( \phi V_{\text{max}} \), the concrete section should be increased in size (AASHTO 5.8.3.3).

The limit of \( f'_{ys} \) is taken to be 60 ksi for all frames:

\[
f'_{ys} \leq 60 \text{ ksi}
\]

(AASHTO 5.8.2.8)

The limit of \( f'_{cs} \) is taken to be 10 ksi for all seismic regions:

\[
f'_{cs} \leq 10 \text{ ksi}
\]

(AASHTO 5.1, 5.4.2.1)

Note that if torsion design is performed and torsion rebar is needed, the equation given in AASHTO 5.8.3.3 does not need to be satisfied independently. See the next section Design of Beam Torsion Reinforcement for details.
The maximum of all the calculated $A_s/s$ values, obtained from each load combination, is reported for the major direction of the beam along with the controlling shear force and associated load combination name.

The beam shear reinforcement requirements reported by the program are based purely on shear strength considerations. Any minimum stirrup requirements to satisfy spacing and volumetric consideration must be investigated independently of the program by the user.

### 3.5.3 Design Beam Torsion Reinforcement

The torsion reinforcement is designed for each design load combination at a user-defined number of stations along the beam span. The following steps are involved in designing the shear reinforcement for a particular station because of beam torsion:

- Determine the factored torsion, $T_u$.
- Determine special section properties.
- Determine critical torsion capacity.
- Determine the reinforcement steel required.

Note that the torsion design can be turned off by choosing not to consider torsion in the Preferences.

#### 3.5.3.1 Determine Factored Torsion

In the design of torsion reinforcement of any beam, the factored torsions for each design load combination at a particular design station are obtained by factoring the corresponding torsion for different analysis cases with the corresponding design load combination factors (AASHTO 3.4.1).

In a statistically indeterminate structure where redistribution of the torsional moment in a member can occur due to redistribution of internal forces upon cracking, the design $T_u$ is permitted to be reduced in accordance with code. However, the program does not try to redistribute the internal forces and to reduce $T_u$. If redistribution is desired, the user should release the torsional DOF in the structural model.
3.5.3.2 Determine Special Section Properties

For torsion design, special section properties such as $A_{cp}$, $A_{oh}$, $A_o$, $p_c$, and $p_h$ are calculated. These properties are described as follows (AASHTO 5.8.2.1).

\[
\begin{align*}
A_{cp} &= \text{Area enclosed by outside perimeter of concrete cross-section} \\
A_{oh} &= \text{Area enclosed by centerline of the outermost closed transverse torsional reinforcement} \\
A_o &= \text{Gross area enclosed by shear flow path} \\
p_c &= \text{Outside perimeter of concrete cross section} \\
p_h &= \text{Perimeter of centerline of outermost closed transverse torsional reinforcement}
\end{align*}
\]

In calculating the section properties involving reinforcement, such as $A_{oh}$, $A_o$, and $p_h$, it is assumed that the distance between the centerline of the outermost closed stirrup and the outermost concrete surface is 1.75 inches. This is equivalent to 1.5 inches clear cover and a #4 stirrup placement. For torsion design of T beam sections, it is assumed that placing torsion reinforcement in the flange area is inefficient. With this assumption, the flange is ignored for torsion reinforcement calculation. However, the flange is considered during $T_{cr}$ calculation. With this assumption, the special properties for a Rectangular beam section are given as follows:

\[
\begin{align*}
A_{cp} &= bh, \\
A_{oh} &= (b-2c)(h-2c), \\
A_o &= 0.85 A_{oh}, \\
p_c &= 2b + 2h, \text{ and} \\
p_h &= 2(b-2c) + 2(h-2c),
\end{align*}
\]

where, the section dimensions $b$, $h$ and $c$ are shown in Figure 3-7. Similarly, the special section properties for a T beam section are given as follows:

\[
\begin{align*}
A_{cp} &= b_w h + (b_f - b_w) d_s, \\
A_{oh} &= (b_w - 2c)(h-2c), \\
A_o &= 0.85 A_{oh}, \\
p_c &= 2b_f + 2h, \text{ and}
\end{align*}
\]
\[ P_h = 2(h - 2c) + 2(b_w - 2c), \]

where the section dimensions \( b_f, b_w, h, d_s \) and \( c \) for a T-beam are shown in Figure 3-7.

**Figure 3-7 Closed stirrup and section dimensions for torsion design**

### 3.5.3.3 Determine Critical Torsion Capacity

The critical torsion limits, \( T_{cr} \), for which the torsion in the section can be ignored, is calculated as follows:

\[
T_{cr} = 0.125\lambda \sqrt{f_c'} \left( \frac{A_{cp}}{p_c} \right) \sqrt{1 + \frac{f_{pc}}{0.125\lambda \sqrt{f_c'}}} \quad \text{(AASHTO Eqn. 5.8.2.1-4)}
\]

where \( A_{cp} \) and \( p_c \) are the area and perimeter of concrete cross-section as described in detail in the previous section, \( f_{pc} \) is the compressive axial stress (compression positive) which is taken as zero, \( \phi \) is the strength reduction factor for torsion, which is equal to 0.9 for normal weight concrete and 0.8 for lightweight concrete (AASHTO 5.5.4.2), and \( f_c' \) is the specified concrete strength.
3.5.3.4 Determine Torsion Reinforcement

If the factored torsion $T_u$ is less than the threshold limit, $0.25 \phi T_{cr}$, torsion can be safely ignored (AASHTO 5.8.2.1). In that case, the program reports that no torsion is required. However, if $T_u$ exceeds the threshold limit, $0.25 \phi T_{cr}$, it is assumed that the torsional resistance is provided by closed stirrups, longitudinal bars, and compression diagonals (AASHTO 5.8.2).

If $T_u > 0.25 \phi T_{cr}$, the required longitudinal rebar area in addition to flexural reinforcement is calculated as follows:

$$A_l = \frac{T_u P_h}{\phi 2 A_p f_p \tan \theta}$$

(AASHTO 5.8.3.6.3-1)

and the required closed stirrup area per unit spacing, $A_r/s$, is calculated as follows:

$$\frac{A_r}{s} = \frac{T_u \tan \theta}{\phi 2 A_p f_p} b_w$$

(AASHTO 5.8.3.6.2-1)

In the preceding expressions, $\theta$ is the angle of crack as per AASHTO section 5.8.3.4, which has been described in detail in the Design Beam Shear Reinforcement section.

An upper limit of the combination of $V_u$ and $T_u$ that can be carried by the section is also checked using the following equation.

$$\sqrt{V_u^2 + \left(0.9 p_h T_u / 2 A_p \right)^2} \leq 0.25 \phi f_p b_w d$$

(AASHTO 5.8.2.1-6)

For rectangular sections, $b_w$ is replaced with $b$. If the combination of $V_u$ and $T_u$ exceeds this limit, a failure message is declared. In that case, the concrete section should be increased in size.

When torsional reinforcement is required ($T_u > 0.25 \phi T_{cr}$), the area of transverse closed stirrups and the area of regular shear stirrups satisfy the following limit.

$$\left(\frac{A_v}{s} + 2 \frac{A_r}{s}\right) \geq 0.0316 \lambda \sqrt{\frac{T_u}{f_{ys}} b_w}$$

(AASHTO 5.8.2.5-1)
If this equation is not satisfied with the originally calculated $A_t/s$ and $A_v/s$, $A_v/s$ is increased to satisfy this condition.

The maximum of all the calculated $A_t$ and $A_v/s$ values obtained from each design load combination is reported along with the controlling combination names.

The beam torsion reinforcement requirements reported by the program are based purely on strength considerations. Any minimum stirrup requirements and longitudinal rebar requirements to satisfy spacing considerations must be investigated independently of the program by the user.
### Table 3-1 Design Criteria

<table>
<thead>
<tr>
<th>Type of Check/Design</th>
<th>Moment Resisting Frames in Zone 1 (Low Seismic)</th>
<th>Moment Resisting Frames in Zones 2, 3 and 4 (High Seismic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi ) in Compression</td>
<td>( \phi = 0.75 )</td>
<td>( \phi = 0.90 )</td>
</tr>
<tr>
<td>Column Check (Interaction)</td>
<td>NLD(^a) Combinations</td>
<td>NLD(^a) Combinations</td>
</tr>
<tr>
<td>Column Design (Interaction)</td>
<td>NLD(^a) Combinations</td>
<td>NLD(^a) Combinations</td>
</tr>
<tr>
<td>( 0.135 ) ( f'<em>{c} ) / ( f</em>{c} ) &lt; ( \rho &lt; 0.08 )</td>
<td>( 0.01 &lt; \rho &lt; 0.06 )</td>
<td></td>
</tr>
<tr>
<td>Column Shears</td>
<td>( v_{c} = 0.0316 \beta \sqrt{f'_{c}} )</td>
<td>( v_{c} = 0 ) for axial tension</td>
</tr>
<tr>
<td></td>
<td>Minimum volumetric shear reinforcement in potential plastic hinge only for Zone 1</td>
<td>Minimum volumetric shear reinforcement in potential plastic hinge for Zones 2, 3 and 4</td>
</tr>
<tr>
<td>Beam Design (Flexure)</td>
<td>NLD(^a) Combinations</td>
<td>NLD(^a) Combinations</td>
</tr>
<tr>
<td>Beam Minimum Flexural Reinforcement</td>
<td>( M_{x} \geq 1.2 M_{cr} )</td>
<td>( M_{x} \geq 1.2 M_{cr} )</td>
</tr>
<tr>
<td></td>
<td>( \frac{4}{3} A_{s, \text{required}} )</td>
<td>( \frac{4}{3} A_{s, \text{required}} )</td>
</tr>
<tr>
<td>Beam Design (Shear)</td>
<td>NLD(^a) Combinations</td>
<td>NLD(^a) Combinations</td>
</tr>
<tr>
<td></td>
<td>Beam capacity shear ( (V_{p}) ) with overstrength factor 1.3 (Zone 3 and 4)</td>
<td>Minimum of beam capacity shear and modified load combo with earthquake force doubled (Zone 2)</td>
</tr>
<tr>
<td></td>
<td>( v_{c} = 0.0316 \beta \sqrt{f'_{c}} )</td>
<td>( v_{c} = 0.0316 \beta \sqrt{f'_{c}} )</td>
</tr>
</tbody>
</table>

\(^a\)NLD = Number of specific loading
APPENDICES
Typically, design codes require that second order P-delta effects be considered when designing concrete frames. They are the global lateral translation of the frame and the local deformation of members within the frame.

Consider the frame object shown in Figure A-1, which is extracted from a story level of a larger structure. The overall global translation of this frame object is indicated by $\Delta$. The local deformation of the member is shown as $\delta$. The total second order P-delta effects on this frame object are those caused by both $\Delta$ and $\delta$.

The program has an option to consider P-delta effects in the analysis. When P-delta effects are considered in the analysis, the program does a good job of capturing the effect due to the $\Delta$ deformation shown in Figure A-1, but it does not typically capture the effect of the $\delta$ deformation (unless, in the model, the frame object is broken into multiple elements over its length).
Consideration of the second order P-delta effects is generally achieved by computing the flexural design capacity using a formula similar to that shown in the following equation.

\[ MCAP = aM_{nt} + bM_{lt} \]

where,

- \( MCAP \) = Flexural design capacity required
- \( M_{nt} \) = Required flexural capacity of the member assuming there is no joint translation of the frame (i.e., associated with the \( \delta \) deformation in Figure A-1)
- \( M_{lt} \) = Required flexural capacity of the member as a result of lateral translation of the frame only (i.e., associated with the \( \Delta \) deformation in Figure A-1)
- \( a \) = Unitless factor multiplying \( M_{nt} \)
- \( b \) = Unitless factor multiplying \( M_{lt} \) (assumed equal to 1 by the program; see the following text)

When the program performs concrete frame design, it assumes that the factor \( b \) is equal to 1 and calculates the factor \( a \). That \( b = 1 \) assumes that P-delta effects have been considered in the analysis, as previously described. Thus, in general, when performing concrete frame design in this program, consider P-delta effects in the analysis before running the program.
Appendix B
Member Unsupported Lengths and
Computation of K-Factors

The column unsupported lengths are required to account for column slenderness effects. The program automatically determines the unsupported length ratios, which are specified as a fraction of the frame object length. Those ratios times the frame object length give the unbraced lengths for the members. Those ratios also can be overwritten by the user on a member-by-member basis, if desired, using the overwrite option.

There are two unsupported lengths to consider. They are $L_{33}$ and $L_{22}$, as shown in Figure B-1. These are the lengths between support points of the member in the corresponding directions. The length $L_{33}$ corresponds to instability about the 3-3 axis (major axis), and $L_{22}$ corresponds to instability about the 2-2 axis (minor axis).
In determining the values for $L_{22}$ and $L_{33}$ of the members, the program recognizes various aspects of the structure that have an effect on those lengths, such as member connectivity, diaphragm constraints and support points. The program automatically locates the member support points and evaluates the corresponding unsupported length.

It is possible for the unsupported length of a frame object to be evaluated by the program as greater than the corresponding member length. For example, assume a column has a beam framing into it in one direction, but not the other, at a floor level. In that case, the column is assumed to be supported in one direction only at that story level, and its unsupported length in the other direction will exceed the story height.

The program does not calculate the $K$ factors. It assumes that P-$\Delta$ analysis would be performed by the user by choosing the appropriate analysis options. In such cases, the $K$ factor is used only to calculate $\delta_0$. The programs assume $K$ factors to be 1 (AASHTO 4.6.2.5, 5.7.4.3). However, the program allows the user to overwrite the $K$ factors.
The concrete frame design preferences are general assignments that are applied to all of the concrete frame members. The design preferences should be reviewed and any changes from the default values made prior to performing a design. The following table lists the design preferences that are specific to using AASHTO-2012; the preferences that are generic to all codes are not included in this table.

<table>
<thead>
<tr>
<th>Item</th>
<th>Possible Values</th>
<th>Default Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time History Design</td>
<td>Envelopes, Step-by-Step</td>
<td>Envelopes</td>
<td>Toggle for design load combinations that include a time history designed for the envelope of the time history, or designed step-by-step for the entire time history. If a single design load combination has more than one time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified here.</td>
</tr>
<tr>
<td>Item</td>
<td>Possible Values</td>
<td>Default Value</td>
<td>Description</td>
</tr>
<tr>
<td>------------------------------</td>
<td>-----------------</td>
<td>---------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Number of Interaction Curves</td>
<td>Multiple of 4 ≥ 4</td>
<td>24</td>
<td>Number of equally spaced interaction curves used to create a full 360 deg interaction surface (this item should be a multiple of four). We recommend 24 for this item.</td>
</tr>
<tr>
<td>Number of Interaction Points</td>
<td>Any odd value ≥ 5</td>
<td>11</td>
<td>Number of points used for defining a single curve in a concrete frame; should be odd.</td>
</tr>
<tr>
<td>Consider Minimum Eccentricity</td>
<td>No, Yes</td>
<td>Yes</td>
<td>Toggle to specify if minimum eccentricity is considered in design.</td>
</tr>
<tr>
<td>Seismic Zone</td>
<td>Zone 0, Zone 1, Zone 2, Zone 3, Zone 4</td>
<td>Zone 4</td>
<td>This item varies with the geographic location of the structure.</td>
</tr>
<tr>
<td>Pattern Live Load Factor</td>
<td>≥ 0</td>
<td>0.75</td>
<td>The pattern load factor is used to compute positive live load moment by multiplying Live load with Pattern Load Factor (PLF) and assuming that beam is simply supported. This option provides a limited pattern loading to frames. Use zero to turn off this option.</td>
</tr>
<tr>
<td>Utilization Factor Limit</td>
<td>&gt; 0</td>
<td>0.95</td>
<td>Stress ratios that are less than or equal to this value are considered acceptable.</td>
</tr>
</tbody>
</table>
Appendix D
Concrete Frame Overwrites

The concrete frame design overwrites are basic assignments that apply only to those elements to which they are assigned. Table D-1 lists concrete frame design overwrites for AASHTO LRFD 2012. Default values are provided for all overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values to ensure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned.

Table D-1 Overwrites for Columns

<table>
<thead>
<tr>
<th>Item</th>
<th>Possible Values</th>
<th>Default Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Design Section</td>
<td>Any defined concrete section</td>
<td>Analysis section</td>
<td>The design section for the selected frame objects. When this overwrite is applied, any previous auto select section assigned to the frame object is removed.</td>
</tr>
<tr>
<td>Live Load Reduction Factor</td>
<td>$\geq 0$</td>
<td>Calculated</td>
<td>The reduced live load factor. A reducible live load is multiplied by this factor to obtain the reduced live load for the frame object. Specifying 0 means the value is program determined.</td>
</tr>
</tbody>
</table>
### Table D-1 Overwrites for Columns

<table>
<thead>
<tr>
<th>Item</th>
<th>Possible Values</th>
<th>Default Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unbraced Length Ratio (Major)</td>
<td>≥ 0 Calculated</td>
<td></td>
<td>Unbraced length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined.</td>
</tr>
<tr>
<td>Unbraced Length Ratio (Minor)</td>
<td>≥ 0 Calculated</td>
<td></td>
<td>Unbraced length factor for buckling about the frame object minor axis. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined. This factor is also used in determining the length for lateral-torsional buckling.</td>
</tr>
<tr>
<td>Effective Length Factor (K Major)</td>
<td>&gt; 0 1.0</td>
<td></td>
<td>Effective length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length. It is used in $\delta_b$ calculation (AASHTO 4.5.3.2.2b). The value is never calculated by the program. Its value either remains default 1 or the user-defined value. Specifying zero means the value is set to its default value.</td>
</tr>
<tr>
<td>Effective Length Factor (K Minor)</td>
<td>&gt; 0 1.0</td>
<td></td>
<td>Effective length factor for buckling about the frame object minor axis. This item is specified as a fraction of the frame object length. It is used in $\delta_b$ calculation (AASHTO 4.5.3.2.2b). The value is never calculated by the program. Its value either remains default 1 or the user-defined value. Specifying zero means the value is set to its default value.</td>
</tr>
</tbody>
</table>
### Table D-1 Overwrites for Columns

<table>
<thead>
<tr>
<th>Item</th>
<th>Possible Values</th>
<th>Default Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment Gradient Coefficient (C_m Major)</td>
<td>&gt; 0</td>
<td>Calculated</td>
<td>Factor relating actual moment diagram to an equivalent uniform moment diagram about the frame object major axis (AASHTO 4.5.3.2.2b).</td>
</tr>
<tr>
<td>Moment Gradient Coefficient (C_m Minor)</td>
<td>&gt; 0</td>
<td>Calculated</td>
<td>Factor relating actual moment diagram to an equivalent uniform moment diagram about the frame object minor axis (AASHTO 4.5.3.2.2b).</td>
</tr>
<tr>
<td>NonSway Moment Factor (D_b major)</td>
<td>&gt; 0</td>
<td>Calculated</td>
<td>The major moment magnifier for braced mode deflection (AASHTO 4.5.3.2.2b). Specifying 0 means the value is program determined.</td>
</tr>
<tr>
<td>NonSway Moment Factor (D_b minor)</td>
<td>&gt; 0</td>
<td>Calculated</td>
<td>The minor moment magnifier for braced mode deflection (AASHTO 4.5.3.2.2b). Specifying 0 means the value is program determined.</td>
</tr>
<tr>
<td>Sway Moment Factor (D_s major)</td>
<td>&gt; 0</td>
<td>1.0</td>
<td>The major moment magnifier for unbraced mode deflection (AASHTO 4.5.3.2.2b). This value is never calculated by the program. Its value remains either the default 1 or the user-defined value. Specifying zero means the value is set to its default value.</td>
</tr>
<tr>
<td>Sway Moment Factor (D_s minor)</td>
<td>&gt; 0</td>
<td>1.0</td>
<td>The minor moment magnifier for unbraced mode deflection (AASHTO 4.5.3.2.2b). This value is never calculated by the program. Its value remains either the default 1 or the user-defined value. Specifying zero means the value is set to its default value.</td>
</tr>
</tbody>
</table>
### Table D-2 Overwrites for Beams

<table>
<thead>
<tr>
<th>Item</th>
<th>Possible Values</th>
<th>Default Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Design Section</td>
<td>Any defined concrete section</td>
<td>Analysis section</td>
<td>The design section for the selected frame objects. When this overwrite is applied, any previous auto select section assigned to the frame object is removed.</td>
</tr>
<tr>
<td>Live Load Reduction Factor</td>
<td>≥ 0</td>
<td>Calculated</td>
<td>The reduced live load factor. A reducible live load is multiplied by this factor to obtain the reduced live load for the frame object. Specifying 0 means the value is program determined.</td>
</tr>
<tr>
<td>Unbraced Length Ratio (Major)</td>
<td>≥ 0</td>
<td>Calculated</td>
<td>Unbraced length factor for buckling about the frame object major axis. This item is specified as a fraction of the frame object length. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined.</td>
</tr>
<tr>
<td>Unbraced Length Ratio (Minor)</td>
<td>≥ 0</td>
<td>Calculated</td>
<td>Unbraced length factor for buckling about the frame object minor axis. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined. This factor is also used in determining the length for lateral-torsional buckling.</td>
</tr>
</tbody>
</table>


