S.E. Exam Review: Concrete Buildings

Richard A. Miller, P.E., FPCI

Richard.Miller@uc.edu

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NCEES

According to the NCEES website, the exam will use ACI-318-14.

This presentation is based on ACI-318-14.

The main change to ACI-318-14 was a reorganization and some cleaning up of the code language.

Actual changes were limited.

This Webinar

- Will cover seismic frames to assist with Day 2 of the exam, which covers lateral load design. Due to time limitations, this is the only lateral system that will be addressed.
- Some of the information can be used for wind load, another lateral load.
- In the design example, it will be necessary to design for gravity loads (Day 1). Look for this sign for provisions applicable to gravity loads. 

- Applicable ACI Sections will be cited.
- Seismic frames are now CH 18 (were CH 21).

Exam Areas Covered

Day	Area	Approx. % on Exam
Friday Morning	Analysis of Structures - Load	10%
	Analysis of Structures - Methods	20%
	Design and Details of Structures – General Considerations	7.5%
	Design and Details of Structures – Concrete	12.5%
Friday Afternoon	Concrete Building Structures (if chosen)	25%
Saturday Morning	Analysis of Structures – Lateral Force Distribution	22.5%
	Design and Detailing of Structures – General Considerations	7.5%
	Design and Detailing of Structures – Structural System Integration	5%
	Design and Detailing of Structures – Concrete	12.5%
Saturday Afternoon	Concrete Building Structures (if chosen)	25%

Seismic Regions

- ACI 318-14 has Seismic Design Categories (SDC) A – F. (Consistent with ASCE-7)
- SDC is determined by the applicable local code (4.4.6.1).
- The relationship between ACI 318-14 and ACI 318-05:
  - Categories A and B were Low Seismic Risk
  - Category C was Moderate Seismic Risk
  - Categories D – F was High Seismic Risk

Table R18.2 ACI 318-14

Component resisting earthquake effect unless noted	Seismic Design Category			
	A (none)	B (18.2.1.3)	C (18.2.1.4)	D-F (18.2.1.5)
Analysis and Design Requirements	None	18.2.2	18.2.2	18.2.2 18.2.4
Materials		None	None	18.2.5 through 18.2.8
Frame Members		18.3	18.4	18.6 through 18.9
Str. Walls and Coupling Beams		None	None	18.10
Precast Str. Walls		None	18.5	18.5 (if permitted); 18.11
Str. Diaphragms and Trusses		None	None	18.12
Foundations		None	None	18.13
Members not proportioned to resist EQ forces		None	None	18.14
Anchors		None	18.2.3	18.2.3

Must also satisfy Ch 1-17, 19-26, except as modified by Ch 18. 14.1.4 on plain concrete walls applies in SDC D-F.

General – Analysis

- 18.2.2 Applies to SDC B – F
- This covers analysis and proportioning of structural members.
  - Requires the interaction of structural and non-structural elements to be considered in analysis.
  - Allows rigid members to be assumed not part of the seismic resisting system as long as their effect on response and failure is considered.
  - Requires members below the base to be designed according to CH 18 if they transmit load to the foundation.

Anchoring to Concrete

- Applies to SDC C-F
- Anchors resisting earthquake forces in SDC C, D, E or F **must** meet 17.2.3 (18.2.3.1)
- ACI 318-14 CH 17 discusses required anchor strength.

Seismic Moment Frames

- Ordinary Moment Frames
  - For lateral loads in areas of low seismic forces (SDC B)
- Intermediate Moment Frame
  - For lateral loads in conjunction with seismic load in SDC C
- Special Moment Frame
  - For lateral loads in conjunction with seismic load in SDC D-F

Moment Frames – Low or No Seismic

- Moment Frames
  - Used in SDC A .
  - Satisfy ACI 318 Chapters 1-17; 19-26.
  - Do not need to satisfy CH 18
- Principals of Ordinary Moment Frame design apply but CH 18 provisions are ignored.

Ordinary Moment Frames

- Ordinary Moment Frames
  - Used in SDC B.
  - Satisfy ACI 318 Chapters 1-17; 19-26.
  - Satisfy 18.2.1.3 – 7; 4.4.64 as applicable
    - General items such as materials, splices, etc.
  - Satisfy 18.3
    - Specific requirements for OMFs

Intermediate Moment Frames

- Intermediate Moment Frames (18.4)
  - Used in SDC C
  - Satisfy ACI 318 Chapters 1-17; 19-26.
  - Satisfy Chapter 18.2.1.3 – 7; 4.4.6.4 (General Requirements)
  - Satisfy Chapter 18.4
    - Special provisions for IMFs

Special Moment Frames

- Special Moment Frames
  - Used in SDC D - F
  - Satisfy ACI 318 Chapters 1-17; 19-26.
  - Satisfy Chapter 18.2.1.3 – 7; 4.4.6.4
  - Satisfy Chapters 18.2.3-8
  - Satisfy Chapter 18.9

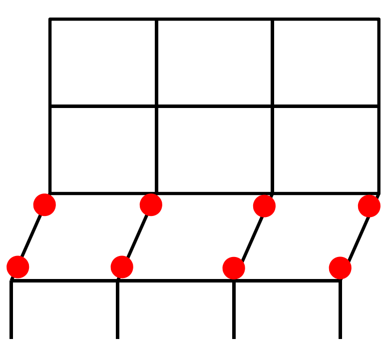
Seismic Frame Philosophy

Seismic failure of frames occurs when plastic hinges form.

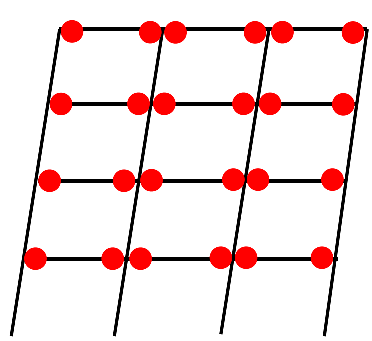
There are two basic ways a building can fail:

The hinges form in the columns (Story Mechanism)

The hinges form in the beams (Sway Mechanism)

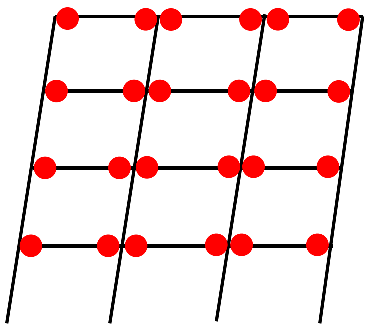


Story Mechanism
Weak Column/Strong Beam
Undesirable!



Sway Mechanism
Strong Column/Weak Beam
Required by Code!

Seismic Frame Philosophy



Sway Mechanism
Strong Column/Weak Beam
Required by Code!

Note: The Code uses both N_u and P_u for axial loads, but in CH 18 it always uses P_u.

We want the beams to fail before the columns.

ACI MAY require:

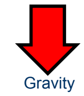
1) If there is a certain level of axial load, P_u, in a beam, it may have to be designed as a beam/column.
2) There may be a requirement for relative strength of columns and beams and joints.
3) There may be requirements for relative strength along the beams.
4) There may be special requirements for shear.
5) There may be a requirement to consider both maximum and minimum material strength.

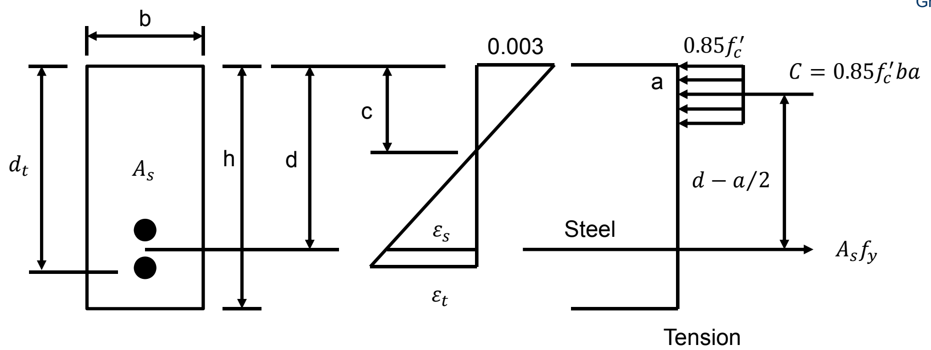
Ordinary Moment Frames

- Used for resisting various lateral loads. May only be used in SDC A or B.
- Note that Chapter 18 does not place restrictions on P_u for beams in an ordinary moment frame.

Reminder

M_n is the nominal moment strength.





$C = T$

$0.85f_c'ba = A_s f_s$ $\qquad$ $M_n = A_s f_y\left(d - \frac{a}{2}\right) = 0.85f_c'ba\left(d - \frac{a}{2}\right)$

$a = \frac{A_s f_s}{0.85f_c'b}$

Note: d is the depth to the centroid of all the tensile steel, d_t is the depth to the tensile steel furthest from the compression face.

Reminder

Recall:

$$\beta_1 c = a$$

Where:

c = depth of neutral axis

a = depth of stress block for finding M_n.

$\beta_1 = 0.85$ if $f_c' \leq 4$ ksi

0.65 if $f_c' \geq 8$ ksi

interpolate in between

Example: For $f_c' = 6$ ksi, $\beta_1 = 0.75$

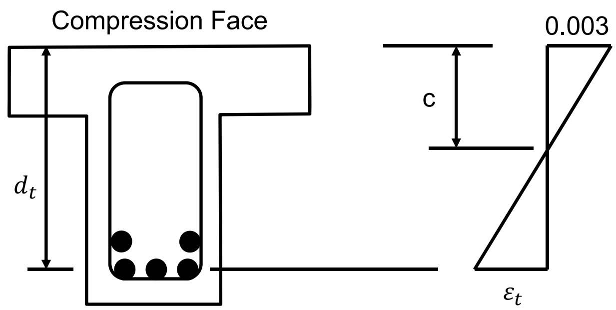
Ordinary Moment Frames

For **non-prestressed** flexural members:

If $P_u \leq 0.1 f_c' A_g$, then:

Extreme steel tensile strain at M_n:

$\varepsilon_t \geq 0.004$ (9.3.3.1)

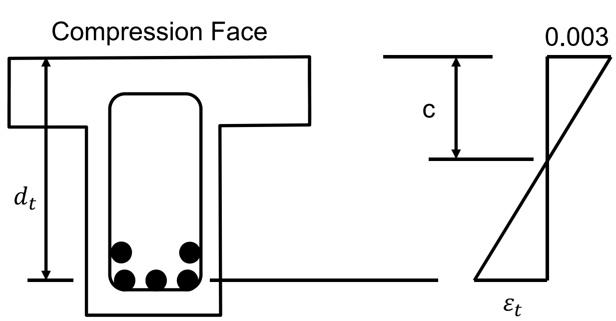


Remember – M_n is defined as the moment capacity when the extreme compressive strain = 0.003. A linear strain diagram is used to find ε_t. The term "c" is the depth to the neutral axis. The width of the compression face is b.

Definition of Section Types (ACI 21.2.2)

Strain in the extreme tensile steel, ε_t	**Type of Section**	**ϕ**
$\varepsilon_t \leq f_y/E_s$	Compression Controlled $c/d_t \geq 3/5$*	0.65 Tied Conf. 0.75 Spiral Conf.
$0.005 > \varepsilon_t > f_y/E_s$	Transition $3/8 < c/d_t < 3/5$*	Interpolated based on ε_t
$\varepsilon_t > 0.005$	Tension Controlled $c/d_t \geq 3/8$	0.9

*For Grade 60 steel and prestressing steel, where f_y/E_s may be taken as 0.002.



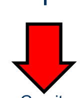
Ordinary Moment Frames

- Non Prestressed Flexural Members (including those not in OMFs)
  - Must meet Section 9.6.1.2 (minimum flexural reinforcement $3\sqrt{f_c'}b_w d/f_y \geq 200 b_w d/f_y$).
  - At each section, $M_u \leq \Phi M_n$ (9.5.1.1)
    - No requirement on relative positive and negative moment strengths at face of joint.
    - No requirement relative moment strength at any section as compared to joint moment strength.

Gravity

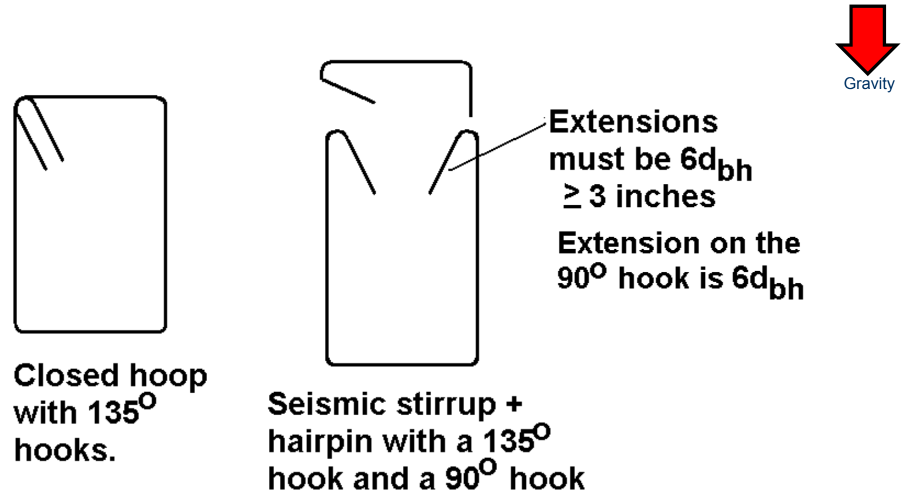
Ordinary Moment Frames

- Non Prestressed Flexural Members (including those not in OMFs)
  - Must meet structural integrity requirements of Section 9.7.7 if not EQ resistant or SDC A; perimeter beams
    - At least 1/6 of the tension reinforcement required for negative moment at the support (2 bars min) shall be made continuous.
    - At least ¼ of the tension reinforcement required for positive moment at midspan (2 bars min) shall be made continuous.
    - Longitudinal structural integrity reinforcement must pass through the region bounded by the longitudinal reinforcement of the column.
    - Longitudinal structural integrity reinforcement must be enclosed by closed stirrups (25.7.1.6) or hoops along the clear span.
    - Longitudinal structural integrity reinforcement must develop f_y at the face of the support.
    - Positive steel is spliced at or near a support, negative steel is spliced at or near midspan. Use Class B, welded or mechanical splices



Gravity

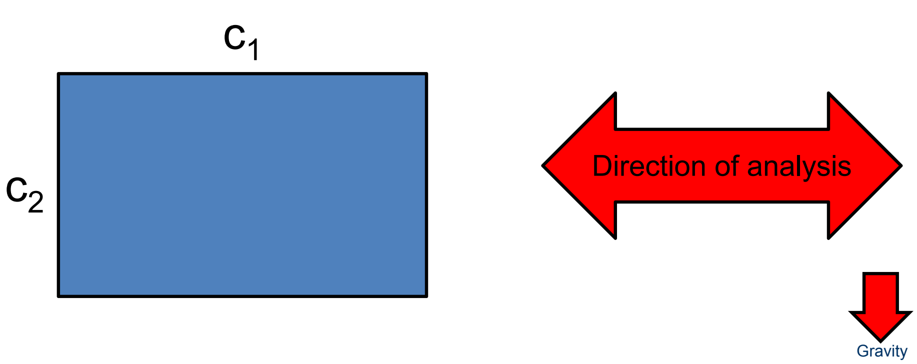
Stirrup Options



Options for beam hoops. For the stirrups on the right, the 90° hook is always on the slab side for a perimeter beam. 25.7.1.6

Definition of c_1 and c_2

Definition: The term c_1 is the dimension of the rectangular column, or equivalent rectangular column, capital or bracket in the direction of analysis. The term c_2 is the column dimension in the perpendicular direction. This definition applies to all columns, even those not in moment frames.



Ordinary Moment Frames

In SDC B – Ch 9.7.7 is modified by Ch 18.3:

18.3.3

Beams must have 2 continuous bars on the top face and on the bottom face.

These bars must develop f_y developed at the face of the support.

Continuous bottom bars must have at least ¼ of the maximum bottom steel area.

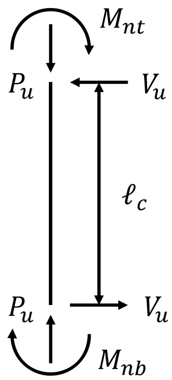
Ordinary Moment Frames

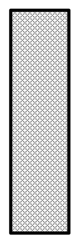
18.3.3: Columns with an unsupported length of < $5c_1$ shall have ϕV_n at least the lesser of:

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of lateral forces considered, resulting in the highest flexural strength.

b) The maximum shear obtained from design load combinations that include E, with $\Omega_0 E$ substituted for E.

Ordinary Moment Frames





$V_u = \frac{M_{nt}+M_{nb}}{\ell_c} + V$

V is factored shear due to all loads but E

Here is the rule for columns with an unsupported length of < $5c_1$.

18.3.3: Use the lesser of applied V_u or the shear caused by

$$U = 1.2D + 1.0L + 0.2S + \Omega_0 E$$

Ω_0 is the overstrength factor in ASCE 7; currently = 3

ℓ_c is the clear height.

Recall, c_1 is column dimension in direction of analysis. To determine M_{nt}, use the interaction diagram value corresponding to P_u. If there are different values of M_{nt}, use the biggest.

Ordinary Moment Frames

- Non Prestressed Flexural Members (including those not in OMFs)
  - No special requirements to encase splices in hoops unless required for bond, development length or splices.
  - No added restrictions on the use of lap splices.
  - For shear and torsion, meet the requirements of the appropriate chapter.
  - **For flexural members subject to stress reversals**, provide closed stirrups around the flexural reinforcement.

Ordinary Moment Frames

- Columns (including those in not in OFMs) (compression members)
  - There are no special rules for columns (except as previously stated for short columns)
  - Columns must meet the requirements of Ch 10.
  - Columns **may** be “slender” and in a non-sway or sway frame. In these cases, the requirements of 6.6.4 must be met.
- Beam Column Joints
  - No special requirements. Meet 15.4.

Intermediate Moment Frames

- Used in SDC C
- Must meet Chapter 18.2.2 for analysis and proportioning.
- Must meet Chapter 18.4

Intermediate Moment Frames

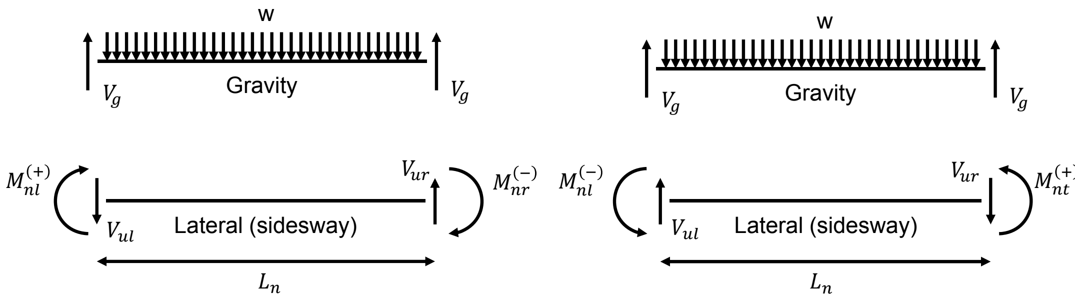
- Beam
  - Is a member primarily subject to flexure and shear; may or may not have axial load and torsion. In a moment resisting frame, they are primarily horizontal (2.3 terminology)
  - 18.4.2.1 Must have at least two top and bottom continuous bars. Continuous bottom bars (positive moment reinforcement) must have at least ¼ the maximum area of bottom bars along the span. Continuous bars must be anchored to develop f_y at the face of the support.

Intermediate Moment Frames

- IMF Flexural Members and Columns
  - 18.4.2.3 – The design shear force shall be the lesser of:
    a) The sum of the shear associated with the development of the nominal moment strength at the end of the member + gravity load.
    b) The maximum shear from any load combination containing E, when E is that prescribed by the governing code for earthquake resistant design, multiplied by the appropriate factor.

This provision is illustrated on the next few slides.

Intermediate Moment Frame Shear Requirements (18.4.2.3)



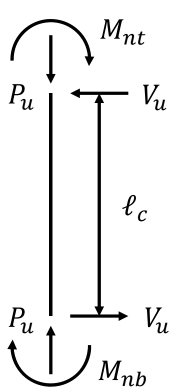
Use: $w = 1.2D + 1.0L + 0.2S$

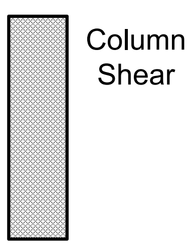
Assume that each joint develops M_n and calculate the corresponding shear. Consider sidesway in **both** directions, using the greater of V_{ul} and V_{ur} at each end.

Use the lesser of V_{ul} and V_{ur} or the shear caused by 2E.

Add the **factored** gravity shear, V_g.

Intermediate Moment Frames





$V_u = \frac{M_{nt}+M_{nb}}{\ell_c} + V$

V is factored shear due to all loads but E

Use the lesser of V_u or the shear caused by

$U = 1.2D + 1.0L + 0.2S + \Omega_0 E$

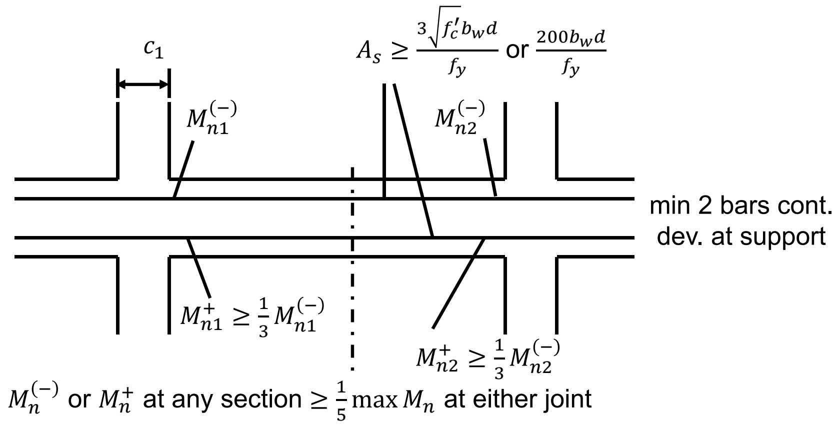
Ω_0 is the overstrength factor in ASCE 7; currently = 3

Here is the same rule for columns. ℓ_c is the clear height.

This is the OMF rule (18.3.3).

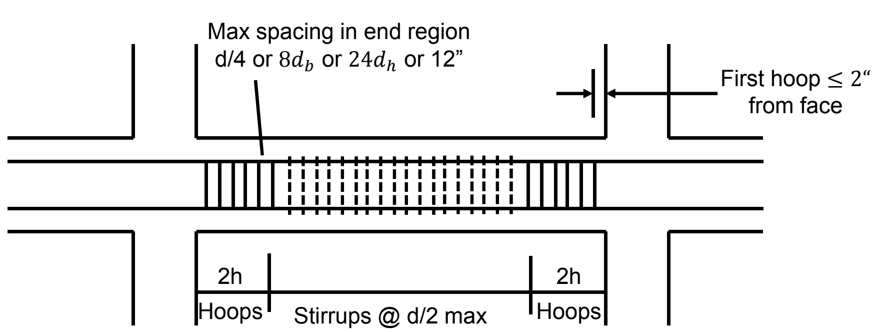
Intermediate Moment Frames

- IMF Flexural Members (18.4.2.2) – **Required Relative Moment Strengths**:
  - At the face of a joint, $M_n^+ \geq M_n^{(-)}/3$
  - At every section, provide M_n^+ and $M_n^{(-)} \geq 1/5$ of the **maximum** M_n (either + or -) at the face of either joint.



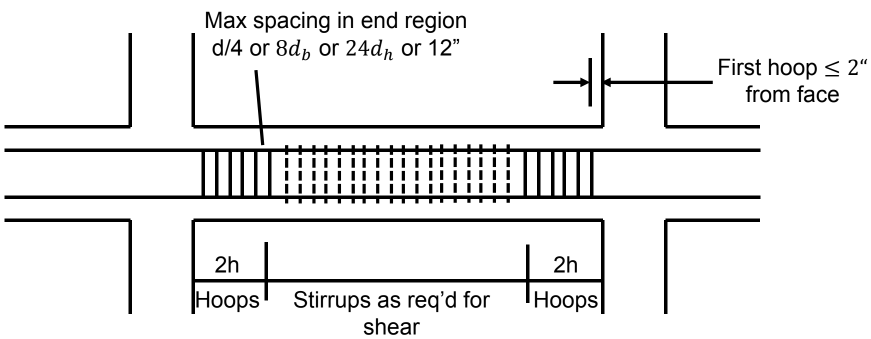
Intermediate Moment Frames

- IMF Flexural Members
  - Transverse Reinforcement (18.4.2.4)
    - Hoops are required over a length equal to 2h from the face of the support toward midspan on both ends.
    - Maximum hoop spacing is d/4; $8d_b$; $24d_h$ or 12 in. The term d_b is longitudinal bar diameter and d_h is the hoop bar diameter. First hoop within 2 inches of the face of support.



Intermediate Moment Frames

- IMF Flexural Members (con't)
  - Transverse Reinforcement (18.4.2.5)
    - Where hoops are not required, provide stirrups as needed for shear throughout the entire member. Maximum spacing as permitted for shear.
    - This provision is not specific about the stirrup type. Probably, the stirrup must be a hoop or a U with seismic (135°) hooks.



Intermediate Moment Frames

- IMF Flexural Members (con't)
  - Transverse Reinforcement (18.4.2.6)
    - If $P > 0.1A_g f_c'$ then provide transverse reinforcement according to 25.7.2.2 and either 25.7.2.3 or 4. (These are the normal provisions for column ties; so basically reinforce as a column).

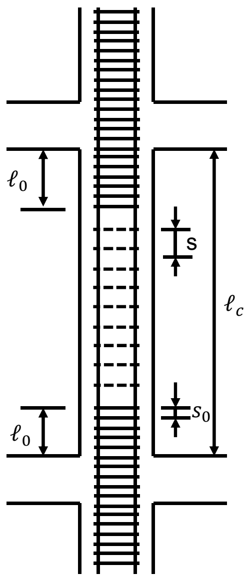
Intermediate Moment Frames

- Columns (18.4.3)
  - A member is considered a column if it is vertical and is used to primarily support axial load; may or may not have shear, flexure and torsion. (2.3 – Terminology).
  - It must meet all requirements for compression members given in Ch 10 (normal rules for columns).
  - Columns must have:
    - Spiral reinforcement meeting 25.7.3
    - **Or** vertical bars and ties meeting 18.4.3.3 through 18.4.3.5, as applicable.

Intermediate Moment Frames

- Columns (18.4.3.3 - 5)
  - Transverse reinforcement (non-spiral)
    - Define ℓ_o as the maximum of h, b, 1/6 clear span of the column or 18 in
    - Within ℓ_o, the maximum spacing of transverse reinforcement, s_o, is the minimum of $8d_b$, $24d_h$, h/2, b/2 or 12 in. The first hoop $\leq s_o$ from the joint face (21.3.5.3). The terms d_b = longitudinal bar diameter and d_h = tie diameter.
    - Outside ℓ_o, the maximum spacing of transverse reinforcement, s, as specified in 10.7.6.5.2 and 25.7.2.3 (normal column rules).

Transverse Reinforcement Requirements for Columns



ℓ_c is clear height of column

ℓ_o max of c_1, c_2, $\ell_c/6$ or 18 in

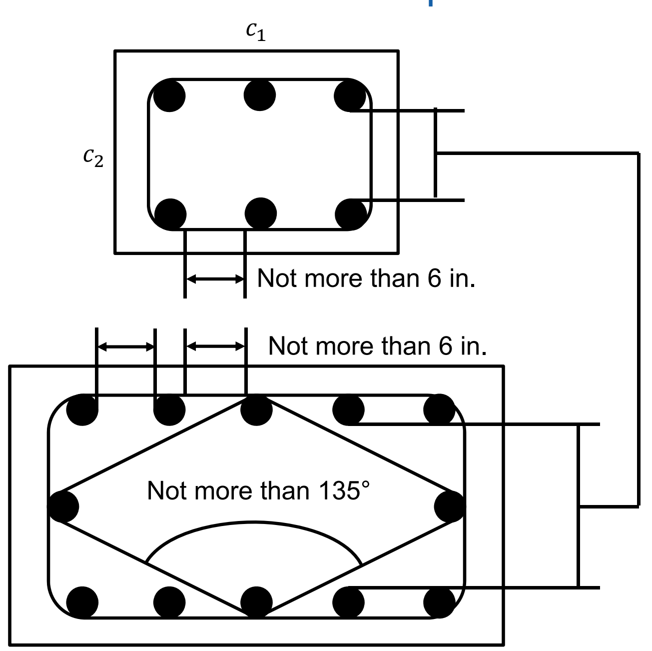
s_0 min of $8d_b$, $24d_{b\ tie}$, $c_1/2$, $c_2/2$ or 12 in.

s = c_1, c_2, $16d_b$, $48d_{b,tie}$ (25.7.2) but s_{max} = d/2 or 24 in (9.7.6.2.2)

First tie within s_0 of face.

Splice at slab/beam (normal splice)

Transverse Reinforcement Requirements for Columns



This applies to all columns – even those not in a lateral load frame.

Bar spacing and lateral support meeting 25.7.2.3

Intermediate Moment Frames

- Columns (con't)
  - Joint transverse reinforcing must meet Ch 15.
  - When loads cause transfer of moment, the resulting shear must be considered.
  - Joint reinforcing must meet equation 15.4.

$$A_{v,min} = 0.75\sqrt{f_c'}\frac{b_w s}{f_{yt}} \geq \frac{50 b_w s}{f_{yt}}$$

Intermediate Moment Frames

- See 18.4.3.6 for columns supporting discontinuous stiff members (like walls).
- See 18.4.5 for two way slabs without beams.

Special Moment Frames

- Excellent reference: **Seismic Design of Reinforced Concrete Special Moment Frames: A Guide for Practicing Engineers (NIST GRC 8-917-1) Moehle, Hooper, Lubke**.
- Can download PDF online.

Special Moment Frames

- Special Moment Frames have material requirements, Table 19.2.1.1*.
- Concrete strength $f_c' \geq 3\ \mathrm{ksi}$.
- Lightweight concrete $f_c' \leq 5\ \mathrm{ksi}$, unless there is experimental evidence that structural members made with that lightweight concrete have toughness and strength equal to structural members made of normal weight concrete of the same strength.

*This is correct – material specification have been moved to Chapter 19!

Special Moment Frames

- Have special reinforcing steel requirements (20.2.2.5)
- Must be ASTM A706 Gr 60 **only**!
- ASTM A615 GR 40 or 60 **may** be used **if**
  - Actual yield strength from mill tests $< f_y + 18\,\mathrm{ksi}$ (retests cannot exceed $f_y + 18\,\mathrm{ksi}$).
- **And** the $f_{u,actual}/f_{y,actual} \geq 1.25$.
- Transverse steel (hoops, ties, stirrups) specified yield strength $(f_{yt}) \leq 100\,\mathrm{ksi}$.

Special Moment Frames

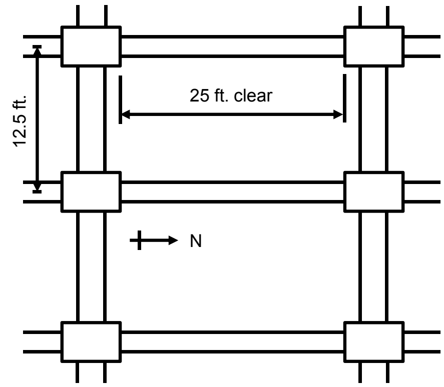
- Mechanical Splices 18.2.7.
  - Type 1
    - must develop $1.25f_y$ (25.5.7)
    - May not be used within a distance 2d from the face of a column or beam or from sections where yielding may occur due to inelastic displacement.
  - Type 2
    - must develop $1.25f_y$ (25.5.7) and must develop the tensile strength of the bar
    - May be used anywhere

Special Moment Frames

- Welded splices (18.2.8)
  - Must develop $1.25f_y$
  - May not be used within a distance 2d from the face of a column or beam in a special moment frame or from sections where yielding may occur due to inelastic displacement.
- Welding of stirrups, ties, inserts, etc. to longitudinal steel is forbidden!

Design of Special Moment Frames Will Be Shown by Example

Design Example



Consider the portion of a building shown. This is a **plan view**.

We want to design a moment frame in the N/S direction.

Given: Concrete: $f_c' = 6{,}000 \text{ psi}$. $\beta_1 = 0.75$

Steel: GR 60 A706

These meet SMF material requirements.

Note, if A615 is used, it must be tested to assure the yield is not too high!

Reminder

Recall:

$$\beta_1 c = a$$

Where:

$c =$ depth of neutral axis

$a =$ depth of stress block for finding M_n.

For $f_c' = 6 \text{ ksi}$, $\beta_1 = 0.75$

Design Example

A note on the calculations:

The calculation in this example were done in a spreadsheet. The results shown are rounded off to the correct number of significant digits.

However, the spreadsheet carries a large number of decimal places through the calculations. Thus, if you try to duplicate the calculations you might find slight differences due to round-off.

Special Moment Frames – Flexural Members

- According to **NIST GRC 8-917-1**: Suggested spans are usually 20-30 feet.
  - Allows support of gravity and lateral loads without overloading adjacent columns or joints.
- 18.6.2.1 – The clear span, ℓ_n, of a flexural member shall not be less than 4 times the effective depth.
- Width b_w at least the lesser of 0.3h and 10 inches.
- Projection of the beam width beyond the width of the supporting column on each side of supporting column on each side shall not exceed the lesser of c_2 and $0.75c_1$.

Special Moment Frames – Flexural Members

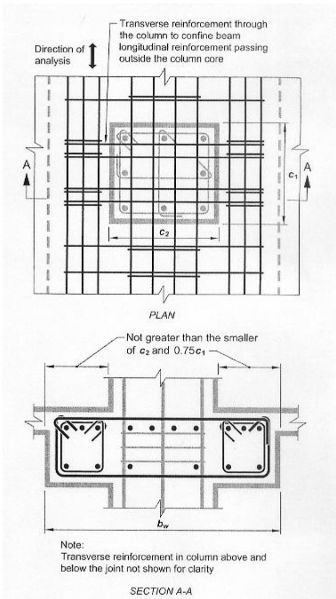


Fig. R21.5.1—Maximum effective width of wide beam and required transverse reinforcement.

18.6.2.1 – The width of the member, b_w shall not be less than the smaller of 10 in or 0.3h, where h is the overall depth of the beam.

The width of the member that projects beyond the edge of the column shall not exceed the lesser of c_2 and $0.75c_1$:

$b_w < c_2 + 2c_2 = 3c_2$

$b_w < c_2 + 1.5c_1$

c_2 is the width of the supporting member and c_1 is the width of the supporting member in the other direction.

R18.6.2 is shown in the figure from ACI 318-14.

Design Example

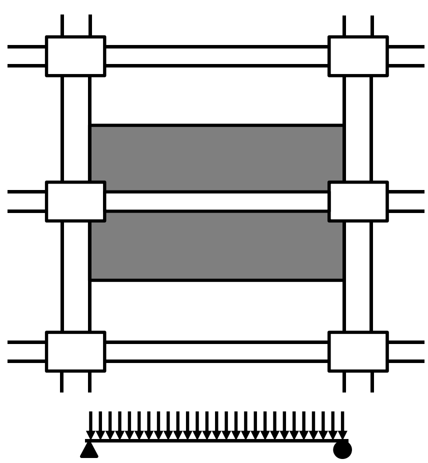
- The following slides show the results of analysis for the beam.
- The analysis was performed according to ASCE-7. The seismic analysis looked at:
  - North/South earthquake with 30% East/West
  - East/West earthquake with 30% North/South
- Because the analysis was done in this way, we can design the N/S frame independently of the E/W.

Beam Loads

- The DL is 1.3 k/ft on the beam.
- The LL is 65 psf which translates to 0.812 k/ft on the beam.
- Assume the slab is a one way slab (because the length is 2 or more times the width).
- The beam is on the upper story a building. Analysis shows some roof live load effect, L_r, but it is orders of magnitude less than other loads, so it was ignored.
- Wind and Earthquake forces were found from ASCE 7. **Note** – Strength Level Forces are used!

Beam Tributary Areas

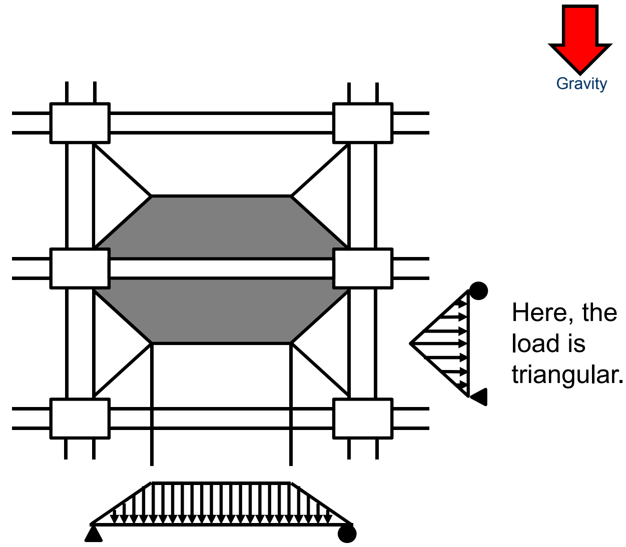
For a one way slab (this example), the tributary areas are shown:



Beam Tributary Areas

Two way slabs:

If this had been a two way slab, the tributary areas would have looked like this.



Shaded area is the tributary area on this beam. Note the trapezoidal shape of the load.

Beam Loads at Supports Unfactored

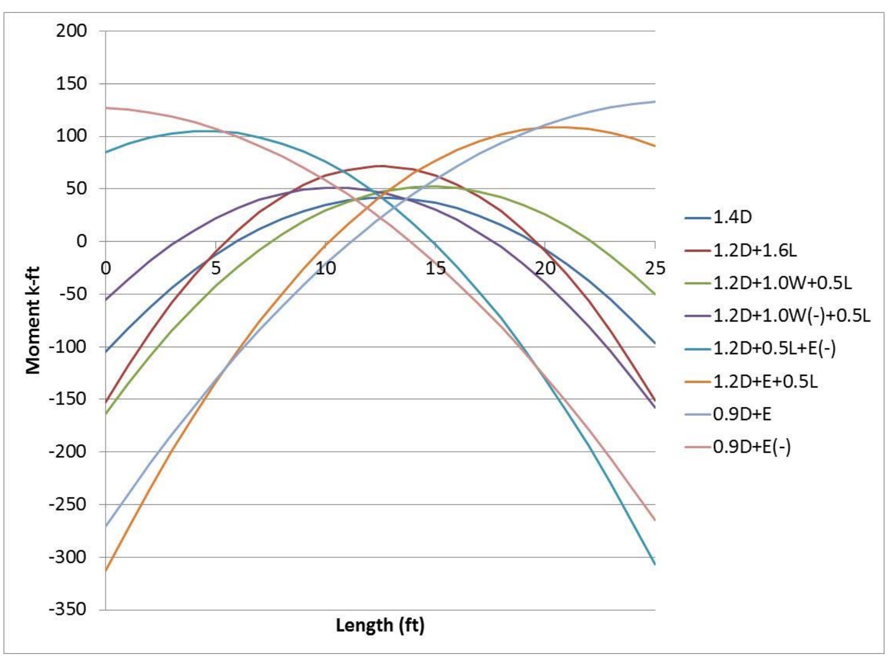
	Negative Moment Left k-ft	Shear Left k	Negative Moment Right k-ft	Shear Right k
D (Dead)	-75	16.5	-68.8	16
L (Live)	-39.1	10	-42.9	10.3
L_r (Roof)	0	0	0	0
W (Wind +)	-53.8	4.3	54.1	4.3
W- (Wind -)	54.1	-4.3	-53.8	-4.3
E (Seismic +)	-203	15.9	194.6	15.9
E- (Seismic -)	194.6	-15.9	-203	-15.9

Must consider +/- Wind and Seismic.

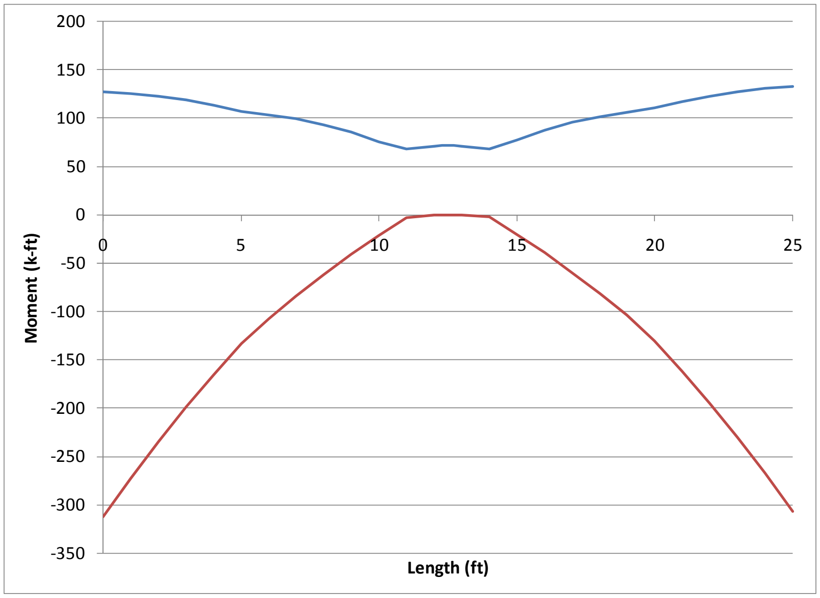
Applicable Load Combinations (5.3)

- $1.4D$
- $1.2D + 1.6L$
- $1.2D + 1.0W + 0.5L$
  - May use 0.5 L because $LL < 100\ \text{psf}$ (5.3.1 note a)
- $1.2D + 1.0E + 0.5L$
- $0.9D + 1.0E$
- Other load combinations do not apply or clearly do not control.
- Must consider +/- for Seismic (E) and Wind
- W and E are Strength Level Loads. If Service Level Loads are used, use 1.6W and/or 1.4E.

Factored Load Moment Diagrams from Analysis

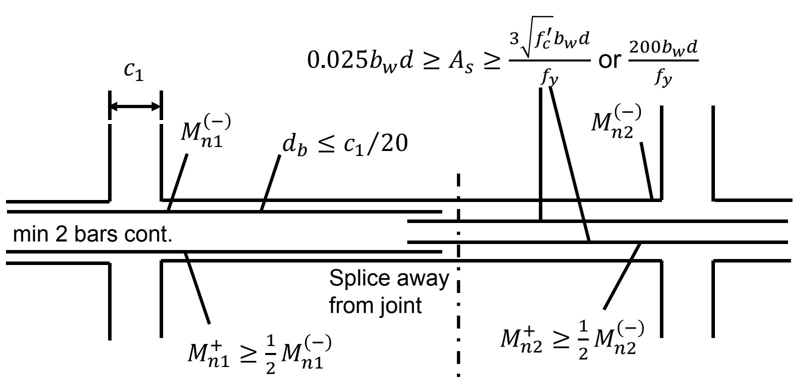


Moment Envelope



$$M_u^{(-)} = 312.4\ \mathrm{k\cdot ft};\ M_u^{+} = 132.8\ \mathrm{k\cdot ft}$$

Longitudinal Steel Requirements



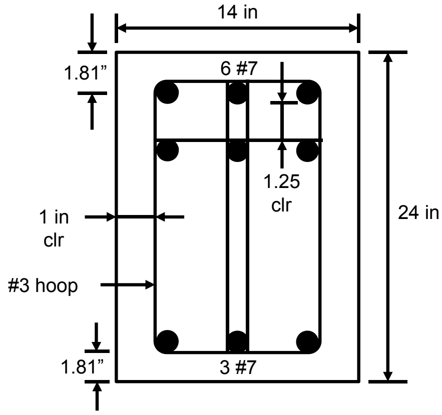
$M_n^{(-)}$ or M_n^{+} at any section $\geq \frac{1}{4} \max M_n$ at either joint

18.6.3 lists minimum steel and relative moment capacity requirements. These requirements are summarized in this figure.

(Adapted from **NIST GRC 8-917-1**)

Proposed Section

Assume the beam will be 14 inches x 24 inches with the reinforcing shown:



Note the use of the #3 double hoop and hairpin. This is to assure confinement.

According to 18.6.2.1:

$b_w > 10 \text{ in}$

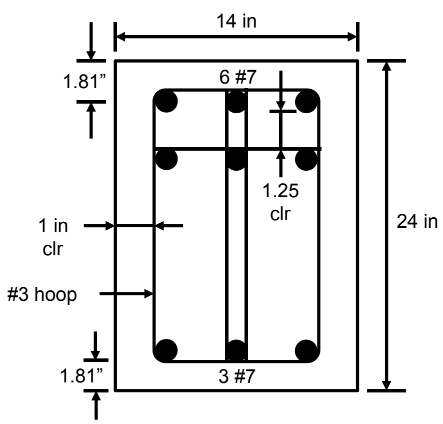
$> 0.3h = 7.2 \text{ in}$

So $b_w = 14 \text{ in}$ is OK!

No overhang – so OK.

18.6.2.1 – The clear span, ℓ_n, of a flexural member shall not be less than 4 times the effective depth. So $d = 21.14$”, so $\ell_{n,min} = 84.6 \text{ in} = 7 \text{ ft} < 25 \text{ ft}$ OK

Proposed Section



18.6.4.7 – Factored axial load, P_u, may not exceed $A_g f_c'/10$ (if it does, additional ties are needed).

$P_u < A_g f_c'/10$ so OK.

The column will be $c_1 = 18$” and $c_2 = 18$”, so the maximum bar diameter for bars through the joint is $c_1/20 = 18/20 = 0.9 \text{ in}$ (18.8.2.3). #7 is largest allowed.

(For LW concrete, the max. bar size = $c_1/26$)

18.6.4.7

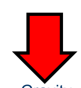
If $P_u > A_g f_c'/10$, provide hoops as required by 18.7.5.2 - 4* over 2h at each support and on 2h at each side of any place likely to yield due to inelastic deformation from lateral loads.

Over the remaining length, hoops satisfying 18.7.5.2* are required with a spacing the lesser of 6 times the smallest flexural bar diameters or 6 in.

If the cover exceeds 4 inches, additional transverse reinforcement must be provided with cover < 4 inches and spacing < 12 inches.

* 18.7.5.2 - 4 details the requirements for columns. This will be shown later.

Definition of Section Types (ACI 21.2.2)



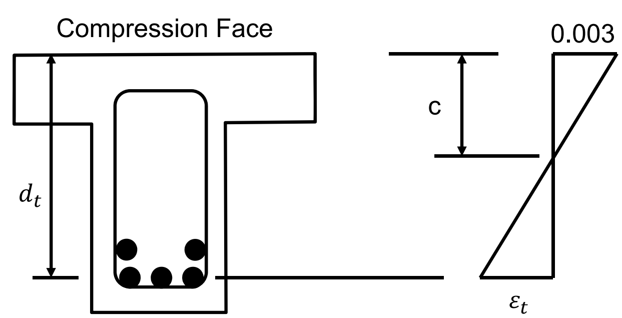
Strain in the extreme tensile steel, ε_t	Type of Section	ϕ
$\varepsilon_t \leq f_y/E_s$	Compression Controlled $c/d_t \geq 3/5$*	0.65 Tied Conf. 0.75 Spiral Conf.
$0.005 > \varepsilon_t > f_y/E_s$	Transition $3/8 < c/d_t < 3/5$*	Interpolated based on ε_t
$\varepsilon_t > 0.005$	Tension Controlled $c/d_t \geq 3/8$	0.9
*For Grade 60 steel and prestressing steel, where f_y/E_s may be taken as 0.002.		

For transition sections, Φ is interpolated based on ε_t.

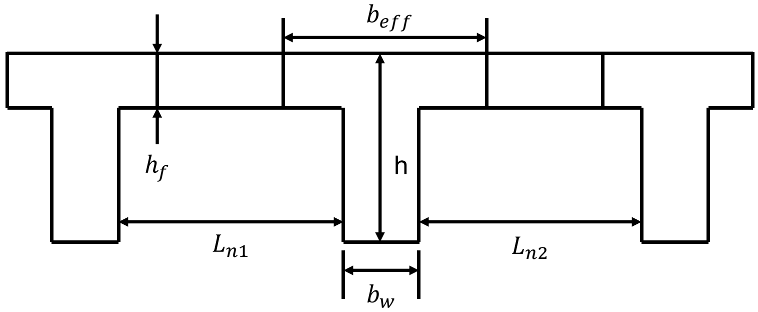
Example: $\varepsilon_t = 0.004$

Section uses ties for transverse reinforcement (Φ = 0.65 at $\varepsilon_t = 0.002$).

Φ = 0.82



T Beam



There is a slab on the beam. ACI is **not** always clear about whether or not to include the slab for negative bending. However for positive and negative bending, the effective flange width is given by (6.3.2.1):

For "T" beams with two overhanging flanges

$$b_{eff} = b_w + \frac{L_{n1}+L_{n2}}{2}$$
$$\boxed{= \frac{Span}{4} + b_w}$$
$$= b_w + 16h_f$$

Note: Change in 2014! This was: $\frac{Span}{4}$

For "T" beams with an overhang on one side only

$$b_{eff} = b_w + \frac{L_{n1}}{2}$$
$$= \frac{Span}{12} + b_w$$
$$= b_w + 6h_f$$

T Beam

The flange width is the smallest of:

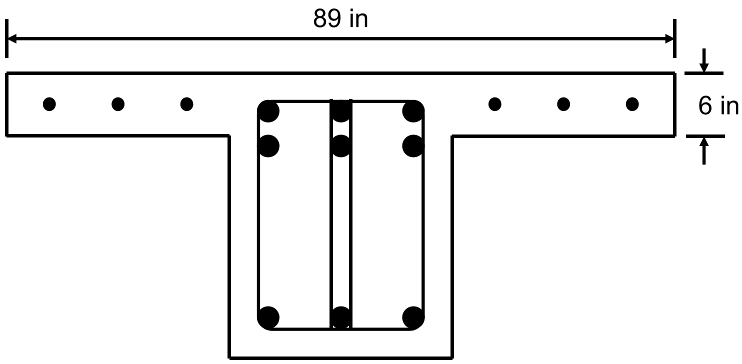
$Span/4 + b_w = 25 \text{ ft}\,(12)/4 + 14 = 89 \text{ in}$ controls

c/c distance between beams $= 12.5 \text{ ft} = 150 \text{ in}$

$b + 16\, h_f = 14 \text{ in} + 16(6 \text{ in}) = 110 \text{ in}$

Assume the slab has #3 @ 12, so 6 bars in flange (picture on next slide).

T Beam



So, find “d” for the negative moment section. Assume the #3 bars are at 3 inches from the top:

$$y_{bs} = \frac{6(0.11\text{ in}^2)(3\text{ in}) + 3(0.6\text{ in}^2)(1.81\text{ in}) + 3(0.6\text{ in}^2)(3.93\text{ in})}{0.66\text{ in}^2 + 3.60\text{ in}^2}$$

$$y_{bs} = 2.89\text{ in}$$

$$d = 24 - 2.89 \approx 21.1\text{ in}$$

Negative Moment Capacity Ignoring Slab

First layer is $1\text{”}\ (\text{clear}) + 0.375\ (hoop) + 0.875\text{”}/2\ (\text{bar}) = 1.81\text{”}$; $d_t = 24 - 1.81 = 22.2\text{ in}$ (distance to extreme tensile steel)

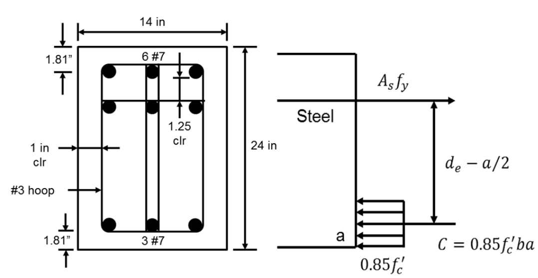
Second Layer is $1.81\text{”} + 1.25\text{”}\ (\text{clear}) + 0.875\text{”}\ (\text{bar}) = 3.93\text{”}$

$$A_s = 6 \times 0.6\text{ in}^2 = 3.60\text{ in}^2$$

$$d = 24 - \frac{3(0.6)1.81 + 3(0.6)3.93}{3.60} = 21.12\text{ in}$$

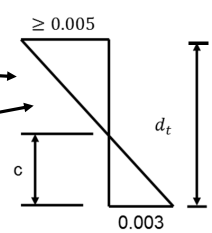
$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{3.60\text{ in}^2 (60\text{ ksi})}{0.85(6\text{ ksi})(14\text{ in})} = 3.02\ in$$

$$c = \frac{a}{\beta_1} = \frac{3.02}{0.75} = 4.03\text{ in}$$



$$\frac{c}{d_t} = \frac{4.03}{22.2} = 0.182 < 0.375$$ Tension Controlled

Or $\varepsilon_s = \left(\frac{d_t}{c} - 1\right) 0.003 = 0.013 > 0.005$



Reminder

Gravity

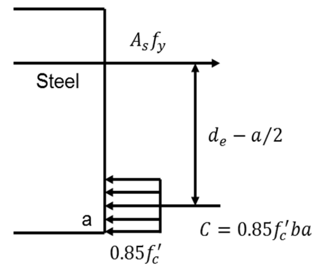
$\varepsilon_t \geq 0.005$ Tension control $\Phi = 0.9$

$\varepsilon_t \geq \varepsilon_y$ Compression control $\Phi = 0.65$ tied

$\Phi = 0.75$ spiral

Transition, interpolate Φ based on ε_t.

$\phi = 0.9$ Tension controlled



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

$$\phi M_n = 0.9(3.60\ \text{in}^2)60\ \text{ksi}\left(21.12 - \frac{3.02}{2}\right) = 3{,}812\ \text{k} - \text{in}$$

$$\phi M_n = 317.7\ \text{k} - \text{ft}$$

$M_u = 312.4\ \text{k} - \text{ft} < \phi M_n$ OK

M_u is from the moment envelope shown previously. The section is adequate for negative moment.

Negative Moment Strength Including Slab Steel

Gravity

$$a = \frac{(0.66 + 3.60)\text{in}^2(60\ \text{ksi})}{0.85(6\ \text{ksi})14\ \text{in}} = 3.58\ \text{in}$$

$$c = \frac{3.58}{0.75} = 4.77\ \text{in};\ \frac{c}{d_t} = \frac{4.77\ \text{in}}{22.2\ \text{in}} = 0.22 < 0.375;\ \phi = 0.9$$

$$\phi M_n = 0.9(4.26\ \text{in}^2)(60\ \text{ksi})\left(21.1\ \text{in} - \frac{3.58\ \text{in}}{2}\right)$$

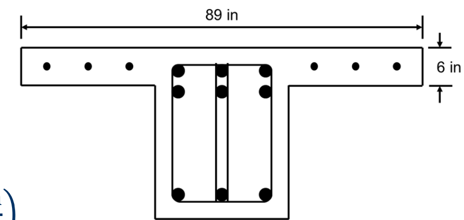
$$\phi M_n = 4{,}935\ \text{k} - \text{in} = 370\ \text{k} - \text{ft} > M_u$$

Positive Moment

The positive moment at the support is 132.8 k-ft.

BUT 16.2.3.2 requires that $\phi M_n^{(+)}$ be at least 0.5 $\phi M_n^{(-)}$.

(Section is tension controlled by inspection; it has the same cross section, but less steel than the tension controlled negative moment section.)



$$a = \frac{3(0.6\ \text{in}^2)60\ \text{ksi}}{0.85(6\ \text{ksi})89\ \text{in}} = 0.24\ \text{in}$$

$$d = 24 - 1.81\ \text{in} = 22.2\ \text{in}$$

$$\varphi M_n = 0.9(1.8\ \text{in}^2)(60\ \text{ksi})\left(22.2\ \text{in} - \frac{0.24\ \text{in}}{2}\right)$$

$$\varphi M_n = 2{,}146\ \text{in} - \text{k} = 179\ \text{k} - \text{ft} > M_u = 132.8\ \text{k} - \text{ft}$$

Slab

If the slab is ignored, $0.5\phi M_n^{(-)} = 159\ \text{k} - \text{ft}$, so $\phi M_n^{(+)}$ is greater.

If slab is counted, $0.5\phi M_n^{(-)} = 185\ \text{k} - \text{ft}$, so $\phi M_n^{(+)}$ is slightly less.

However, the code is not clear about counting the slab steel. The **only** time the code **requires** counting the slab steel is when determining the relative strengths of the beam and column.

NIST GRC 8-917-1 and several text books indicate that both methods – count or ignore the slab – are used. We will ignore the slab unless the code requires us to count it. However, if you choose to include the slab it is not incorrect.

Maximum and Minimum Steel 18.6.3.1

$\rho_{w,max} = 0.025$ 18.6.3.1

$$\rho_w = \frac{A_s}{b_w d}$$

$$\rho_{w,min} = \frac{3\sqrt{f'_c}}{f_y} > \frac{200}{f_y} \quad \text{9.6.2.1}$$

$$\frac{200}{f_y} = \frac{200}{60{,}000} = 0.0033$$

$$\frac{3\sqrt{f'_c}}{f_y} = \frac{3\sqrt{6{,}000}}{60{,}000} = 0.004$$

Where b_w is the **web** width and d is the effective depth from compression face to centroid of all tensile steel.

Gravity

Provisions in the box apply to gravity loads

$$0.025 > \rho_{w,negative} = \frac{3.60\ in^2}{14\ in(21.12\ in)} = 0.012 > 0.004$$

$$0.025 > \rho_{w,positive} = \frac{1.80\ in^2}{14\ in(22.2\ in)} = 0.006 > 0.004$$

In addition, 18.6.3.1 requires **at least** 2 bars be continuous top and bottom!

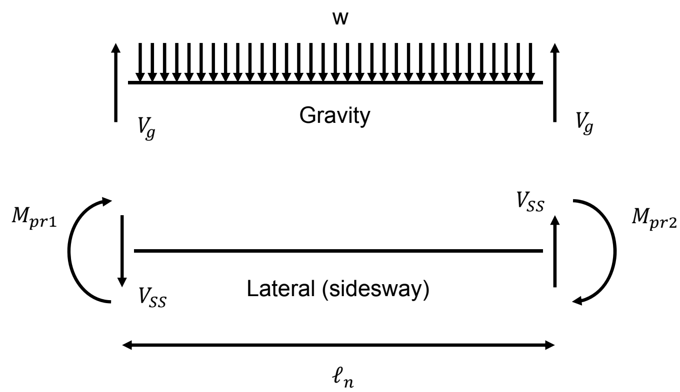
Beam

The beam meets all requirements. We only designed steel at the supports. We do need to check:

1) Adequate along the entire beam? Yes. The end moments are maximum for the entire beam **in this example**.

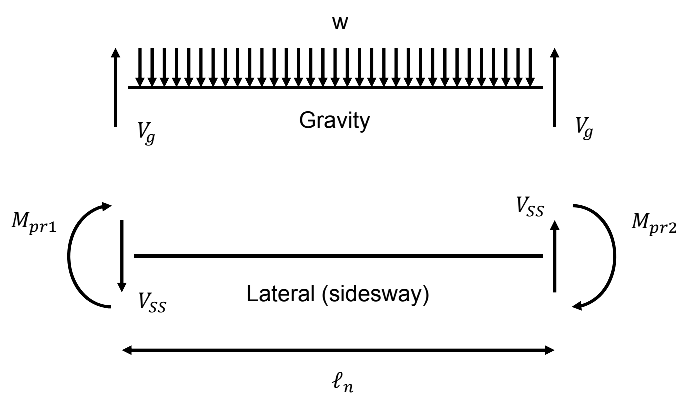
2) Is the strength at **any** section $\geq 0.25 M_{n,max}$ at support? **Yes**. We do not change the steel so this is met.

Special Seismic Frame Philosophy



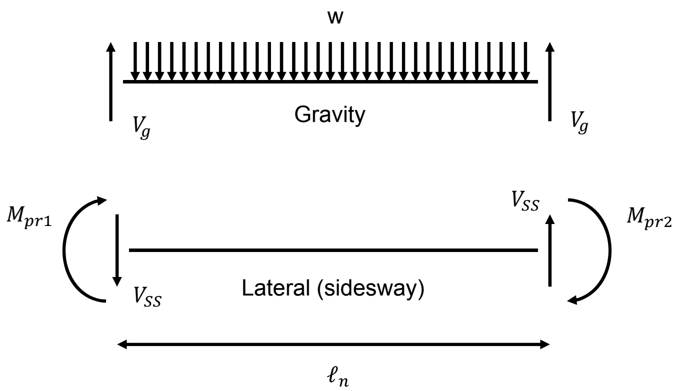
As with IMF, shear in the beam comes from two places: the gravity loads and the lateral load moment (sidesway).

Special Seismic Frame Philosophy



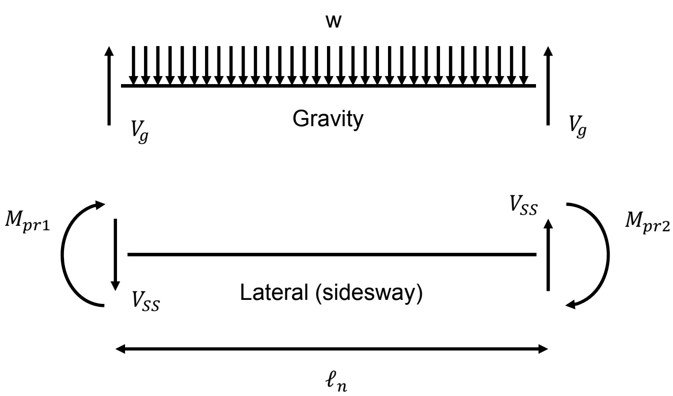
However, there is a difference in how the shear is found. For IMF we used the nominal moment capacity, M_n, which assumes that f_y is exactly as specified (e.g. exactly 60 ksi). But specified yield is a **minimum**. Real steel is stronger!

Special Seismic <u>Frame</u> Philosophy



To prevent a shear failure before yielding the flexural steel, the code requires that the **minimum** design shear forces due to lateral loads in SMFs be based on the "probable" flexural strength of the joints. (18.6.5.1)

Special Seismic <u>Frame</u> Philosophy

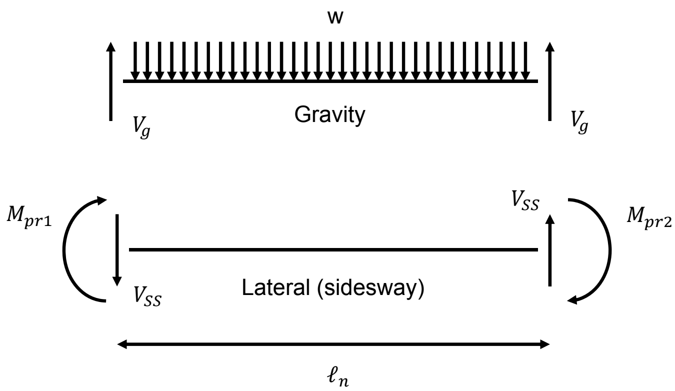


This is why ACI limits the yield strength of steel in seismic frames. A706 specifies a limited f_y. If A615 is used, the test yield strength is limited to $1.25f_y$.

$M_{pr} = M_n$ with $f_s = 1.25f_y$ $(M_{pr} \neq 1.25M_n)$

That is: the probable moment is calculated using 125% of the specified yield strength! Because "a" changes, it's not 1.25 M_n. Also, $\Phi = 1.0$.

Special Seismic Frame Philosophy

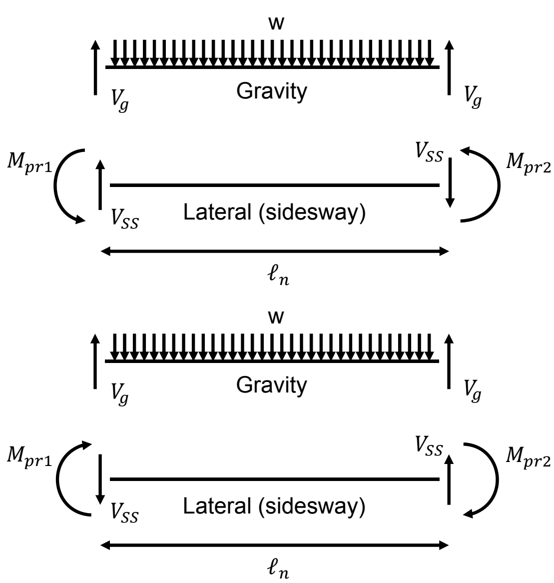


$$V_e = V_g + V_{ss} = \frac{w_u \ell_n}{2} + \frac{M_{pr1}+M_{pr2}}{\ell_n}$$

ℓ_n is the clear span (face/face support) and w_u is the **factored** gravity loads.

$w = 1.2D + 1.0L + 0.2S$ (note, this load combination in 5.3 includes E, but that is calculated from M_{pr})

Note!

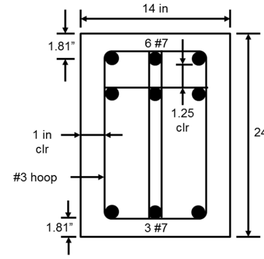


You **must** consider sidesway in both directions and design for the highest shear!

Remember, the longitudinal reinforcement at the left end could be different from the right end!

Negative Probable Moment

The probable moment is found by assuming the steel strength is $1.25 f_y$ and taking $\phi = 1$. Since this a probable moment and phi is assigned 1, we do not need to check tension control.



$$A_s = 3.60 \text{ in}^2$$

$$d = 21.12 \text{ in}$$

$$a = \frac{A_s(1.25 f_y)}{0.85 f'_c b} = \frac{3.60 \text{ in}^2 (1.25)(60 \text{ ksi})}{0.85(6 \text{ ksi})(14 \text{ in})} = 3.78 \text{ in}$$

$$M_{pr} = 3.60 \text{ in}^2 (1.25)(60 \text{ ksi}) \left(21.12 \text{ in} - \frac{3.78 \text{ in}}{2}\right) = 5{,}192 \text{ k} - \text{in}$$

$$M_{pr} = 432.6\ k - ft$$

Note: You can't just multiply M_n by 1.25 because "a" changes, so the moment arm changes.

Negative Probable Moment

Remember, at the joints, the slab steel goes into tension when there is negative moment. The ACI code does **not** say how to handle this steel in seismic applications!

In this case, we will **not** consider the slab steel in finding the negative M_{pr} for beam shear. This is consistent with several design examples in text books. This is not a universally applied concept.

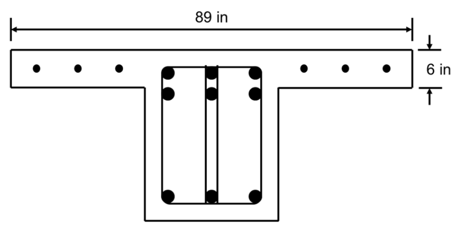
There are places where we will be **required** to consider the slab steel.

Positive Probable Moment

$$a = \frac{3(0.6\ \text{in}^2)(1.25)60\ \text{ksi}}{0.85(6\ \text{ksi})89\ \text{in}} = 0.30\ \text{in}$$

$$\varphi M_n = (1.8\ \text{in}^2)(1.25)(60\ \text{ksi})\left(22.2\ \text{in} - \frac{0.30\ \text{in}}{2}\right)$$

$$\varphi M_n = 2{,}976\ \text{in} - \text{k} = 248\ \text{k} - \text{ft}$$



We do need to consider the slab width for positive bending.

Beam Shear

The controlling load case is:

$1.2D + 0.5L + 1.0\ E$

We can use 0.5L because 5.3.3 allows it if this is not an area of public assembly, a garage or $L > 100\ \text{psf}$.

For E, we calculate the shear from the probable moments.

Beam Shear

The controlling load case is:

Uniform Gravity Load

$$1.2D + 0.5L = 1.2(1.3) + 0.5(0.812) = 1.97\ \mathrm{k/ft}$$

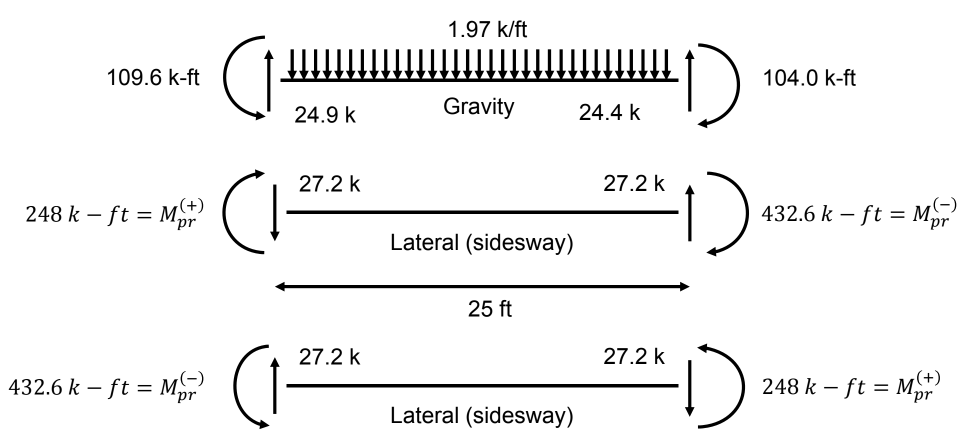
Left

$$1.2M_D + 0.5M_L = 1.2(75) + 0.5(39.1) = 109.6\ \mathrm{k-ft}$$

Right

$$1.2M_D + 0.5M_L = 1.2(68.8) + 0.5(42.9) = 104\ \mathrm{k-ft}$$

Beam Shear



$$V_e = (432.6\ \mathrm{k-ft} + 248\ \mathrm{k-ft})/25\ \mathrm{ft} = 27.2\ \mathrm{k}$$

Beam Shear – Basics

Gravity

$$\phi\, V_u \leq \phi(V_c + V_s)$$
$$V_c = 2\sqrt{f'_c}\, b_w d$$

If minimum shear steel not provided:

$$\sqrt{f'_c} \leq 100\ psi$$
$$V_s = \frac{A_v f_y d}{s}$$
$$V_{s,required} \leq 8\sqrt{f'_c}\, b_w d$$

V_u = Factored Shear
V_c = Concrete Shear Strength
V_s = Stirrup Strength
b_w = Web Width
d = Effective Depth
A_v = Area of Stirrup Crossing Shear Plane
f_y = Stirrup Yield Strength
s = Stirrup Spacing

If not true, resize the section! 22.5.1.2.

For gravity loads, V_u can be taken at "d" from the face of the support if the support reaction, in the direction of the applied shear, induces compression in the end regions; loads are applied at or near the top of the member; there is no point load between the critical section and the support. 9.4.3.2 (beams) 7.4.3.2 (slabs)

Beam Shear

$$V_u = 24.9\text{ k} + 27.2\text{ k} = 52.1\text{ k}$$

$$V_e = 27.2 > \frac{1}{2} V_u = 26.1\text{ k}$$

$$\phi\, V_u \leq \phi(V_c + V_s)$$

$$V_{s,req} = \frac{V_u - V_c}{\phi} = \frac{52.1\text{ k} - 0}{0.75} = 69.5\text{ k}$$

18.6.5.1 this check is at the **face** of the support (not d from face).

If the shear due to earthquake is greater than ½ the total shear and $P_u < A_g f'_c/20$, we must assume the concrete contribution = 0 (18.6.5.2).

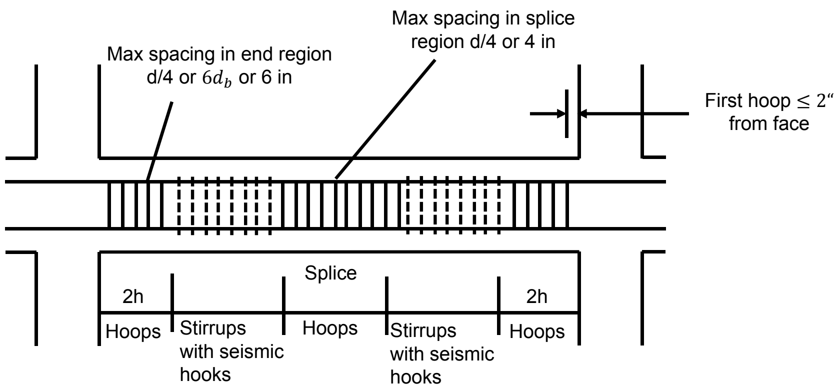
Check:

$$8\sqrt{6{,}000\text{ psi}}(14\text{ in})(21.12\text{ in}) = 183{,}000\text{ lbs}$$

$183\text{ k} > 69.5\text{ k}$ OK

If not true, resize the section! 22.5.1.2

Beam Shear



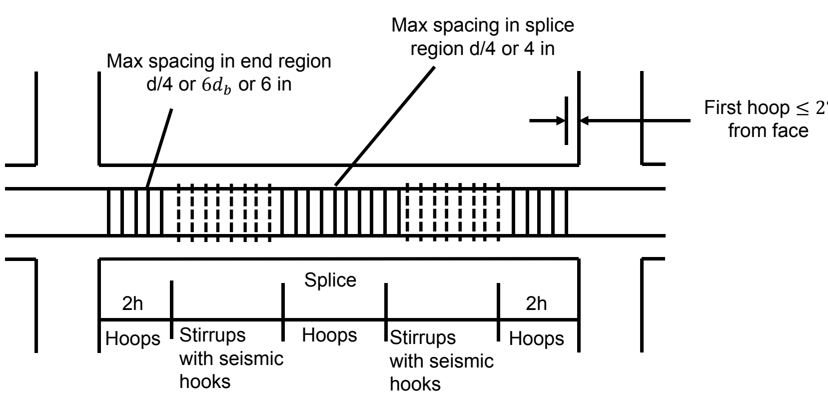
Note: ACI 318 says hoops over a distance of twice the member depth. Some references use 2d, some 2h!

Adapted from **NIST GRC 8-917-1.** Updated to ACI 318-14.

Requirements for transverse reinforcement in beams.

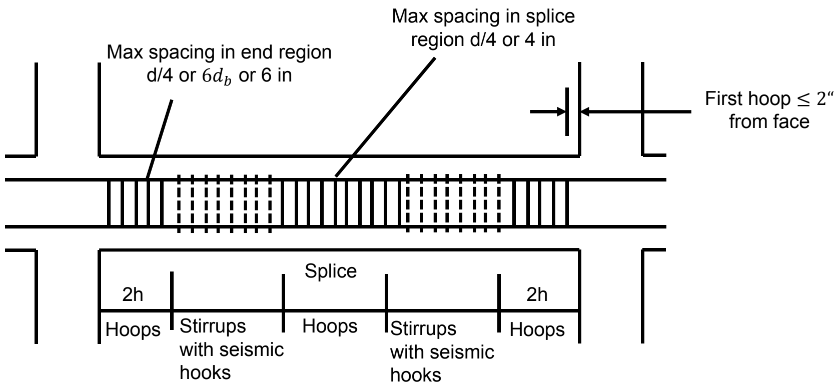
Stirrups have seismic hooks and follow regular spacing rules. Hoops confine end and splice regions.

Special Moment Frames – Flexural Members



18.6.4 Spacing requirements for distances within 2h of the support **or** over a distance 2h on either side of a place where flexural yielding is expected to occur due to lateral displacements beyond the elastic range: d/4; $6d_b$ or 6 in. For d_b use the smallest flexural bar, but exclude skin reinforcing (used for crack control).

Special Moment Frames – Flexural Members



18.6.3.3 Spacing requirements for splices: d/4; 4 in. Lap splices cannot be used within joints, within 2h of the support or over a distance 2h on either side of a place where flexural yielding is expected to occur due to lateral displacements beyond the elastic range.

Beam Shear 21.5.3

Maximum spacing of hoops within 2h of the face.

$$s_{max} = d/4 = 21.12/4 = 5.3 \text{ in}$$

$$= 6d_b = 8(7/8) = 5.3 \text{ in}$$

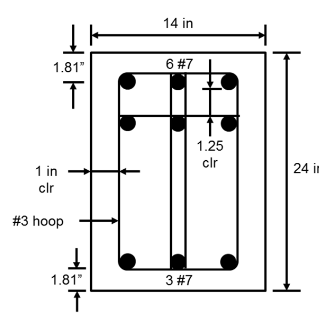
$$= 6 \text{ in}$$

#3 hoop has 4 bars across the shear plane,

so $A_v = 4(0.11\ in^2) = 0.44 in^2$.

Try #3 hoops at 5 inches c/c (Eq'n 22.5.10.5.3):

$$V_s = \frac{A_v f_{yt} d}{s} = \frac{0.44 \text{ in}^2 (60 \text{ ksi}) 21.1 \text{ in}}{5 \text{ in}} = 112 \text{ k} > 69.5 \text{ k}$$



Beam Shear

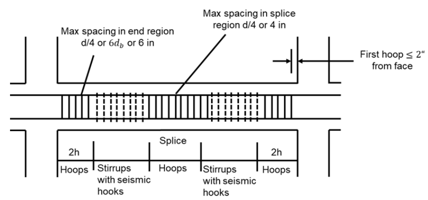
Gravity

$A_{v,min}$ here also applies to gravity loads

Minimum shear steel (9.6.3.3):

$$A_{v,min} = 0.75\sqrt{f_c'}\frac{b_w s}{f_{yh}} = 0.75\sqrt{6{,}000}\frac{(14\text{ in})(5\text{ in})}{60{,}000\text{ psi}} = 0.067\text{ in}^2 \text{ Controls}$$

$$A_{v,min} = \frac{50 b_w s}{f_{yh}} = \frac{50(14\text{ in})(5\text{ in})}{60{,}000\text{ psi}} = 0.058\text{ in}^2$$



$$A_v = 0.44\text{ in}^2 > 0.067\text{ in}^2 \text{ OK}$$

So #3 at 5 in over the first 2h = 48 in.

Start first hoop s/2, from face. Use 2” from face (18.6.4.4) and 11 hoops. This takes the stirrups 52” from the face on each side (11 hoops create 10 spaces at 5 in each).

Beam Shear

Gravity

At 52 inches from the face ($w = 1.97\frac{\text{k}}{\text{ft}} = change\ in\ V$):

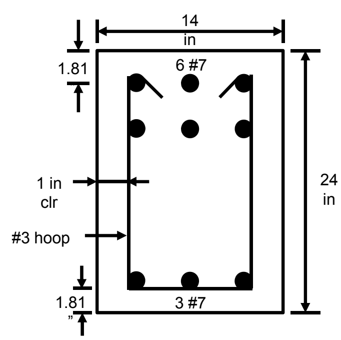
$$V_u = 52.1\text{ k} - \frac{1.97\text{ k}}{\text{ft}}\frac{52\text{ in}}{12} = 43.6\text{ k}$$

$$V_c = 2\sqrt{f'_c}b_w d = \frac{2\sqrt{6{,}000\text{ psi}}(14\text{ in})21.1\text{ in}}{1{,}000} = 45.8\text{ k}$$

$$V_{s,req} = \frac{43.6\text{ k} - (0.75)45.8\text{ k}}{0.75} = 12.3\text{ k} < 4\sqrt{f'_c}b_w d$$

$$s_{max} = \frac{d}{2} \approx 10\text{ in} < 24\text{ in } 18.6.4.6$$

$$V_s = \frac{0.22\text{ in}^2(60\text{ ksi})21.1\text{ in}}{10\text{ in}} = 27.9\text{ k} > 12.3\text{ k}$$

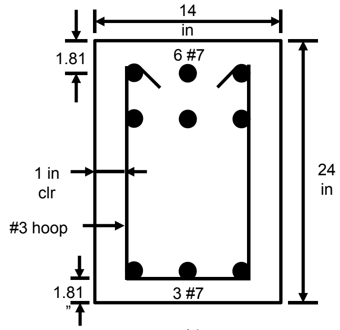


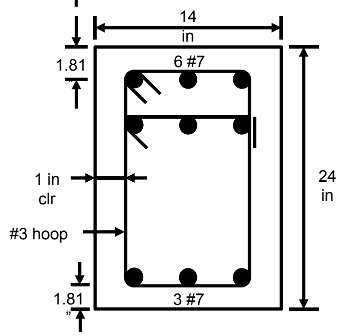
Outside of 2h from the face, we can use V_c (18.6.5.2).

Use #3 seismic stirrups with 10 inch spacing.

If V_s required is between $4\sqrt{f'_c}b_w d$ and $8\sqrt{f'_c}b_w d$; s_{max} is d/4<12 in.

Beam Shear





Outside of 2h* = 48 inches, use of stirrups with seismic hooks is permitted.

Using the closed hoops and a hairpin, as shown, supports the primary reinforcement on both tension and compression faces and is a better detail.

18.6.4.2 requires primary flexural reinforcing bars closest to the tension and compression faces to be supported according to 25.7.2.3 and 4 where hoops are required (within the 2h length at both ends). Spacing of transversely supported bar < 14 in.

Skin reinforcement does not need lateral support.

* Some design examples use 2d.

Beam Splice

Beam splices are either Class A or B (25.5.2.1).

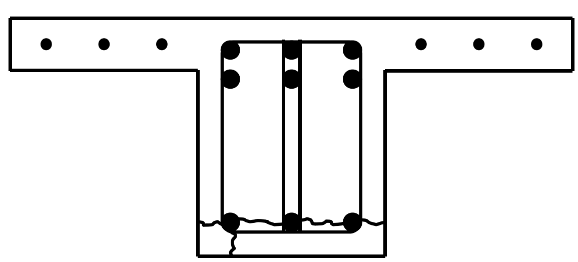
Class A – Splice length = development length ℓ_d

Class B – Splice length = $1.3\ell_d$

To find ℓ_d 25.4.2.3:

$$K_{tr} = \frac{40A_{tr}}{sn}$$

$$\ell_d = \frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\Psi_t\Psi_e\Psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}d_b$$

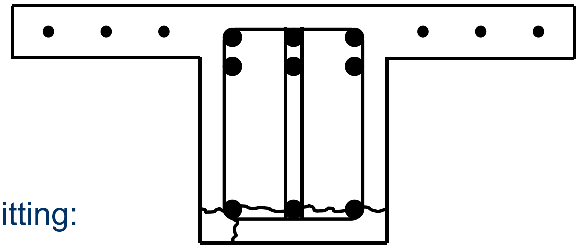


This is also how to find development length for gravity loads.

Beam Splice

$K_{tr} = \frac{40A_{tr}}{sn}$

$\ell_d = \frac{3}{40}\frac{f_y}{\lambda\sqrt{f'c}}\frac{\Psi_t\Psi_e\Psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}d_b$



To find K_{tr} consider a plane of splitting:

A_{sh} = area of stirrup crossing the plane of splitting

F_{sh} = stirrup yield

s = stirrup spacing

n = # of bars being developed

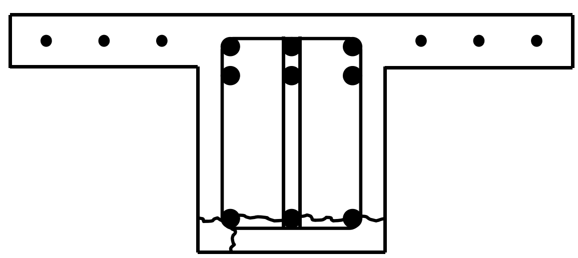
Check K_{tr} for each plane of splitting.

Beam Splice

$K_{tr} = \frac{40A_{tr}}{sn}$

$\ell_d = \frac{3}{40}\frac{f_y}{\lambda\sqrt{f'c}}\frac{\Psi_t\Psi_e\Psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}d_b$

$\frac{c_b+K_{tr}}{d_b} \leq 2.5$



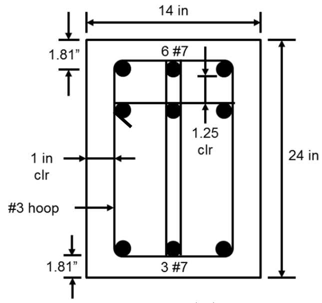
The term c_b is the smaller of the distance from the center of a bar to the nearest surface or ½ the c/c distance between bars.

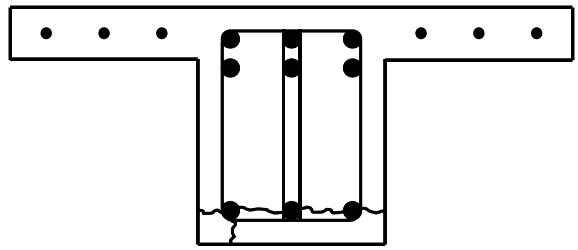
Ψ_e is the epoxy bar factor, Ψ_t is the top bar factor and Ψ_s is the size factor. λ is the lightweight factor. (25.4.2.4)

The term f_y is the longitudinal bar yield and f_c' is the concrete strength.

Finally, d_b is the bar diameter.

Beam Splice





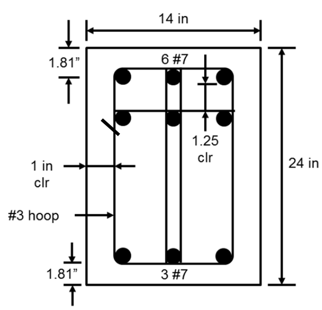
$$c_b = 1 + 0.375 + \frac{1}{2}\left(\frac{7}{8}\right) = 1.81 \text{ in} \leftarrow \text{controls}$$

$$c_b = \frac{1}{2}\frac{\left[14 - 2(1 + 0.375) - 2\left(\frac{1}{2}\right)\left(\frac{7}{8}\right)\right]}{2 \text{ spaces}} = 2.59 \text{ in}$$

For the horizontal splitting plane, n = 3, $A_{sh} = 4(0.11 \text{ in}^2) = 0.44 \text{ in}^2$.

For the vertical splitting plane, n = 1, A_{sh} = 0.11in^2 ; $f_{yh} = 60$ ksi; s = 4 inches for both planes.

Beam Splice



The only Ψ for the top #7 bars is the top bar factor = 1.3. All other Ψ values are 1 as is λ.

For the bottom #7, take out the top bar factor.

$$\ell_d = \frac{35}{1.3} \approx 27 \text{ in}$$

The splice must be Class B (25.5.2.1) as $A_{s,prov} < 2A_{s,req}$; cannot be within 2h of the face (18.6.3.3) and must have hoops as shown over splice at $s = d/4 < 4$ in.

s = 4 inches controls.

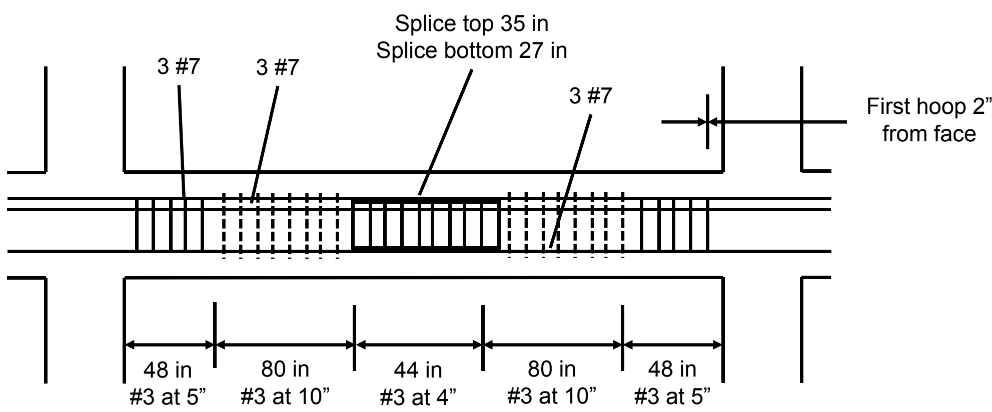
$$K_{tr} = \frac{40(0.44 \text{ in}^2)}{4 \text{ in}(3 \text{ bars})} = 1.47 or\ K_{tr} = \frac{40(0.11 in^2)}{4in(1bar)} = 1.1$$

$$\frac{c_b + K_{tr}}{d_b} = \frac{1.81 + 1.1}{0.875} = 3.32 > 2.5$$

$$\ell_d = \frac{3}{40}\frac{60{,}000(1.3)}{\sqrt{6{,}000}(2.5)}(0.875) = 26.4 \text{ in}$$

$1.3\ell_d = 1.3(26.4) \approx 35$ in Class B splice

Final Beam Design



Column

The columns are 10 feet long (clear span).

From analysis, the column loads are as shown. The moments have been magnified for slenderness effects (Ch 6 covers slenderness).

	P kips	M k-ft
D	88	8.1
L	36	-10.1
L_r	13	-1.5
W	3.6	103
E	50	340

Column

The applicable load combinations are the same as for the beam. The controlling combinations are:

$P = 1.2D + 0.5L + E = 174\ \mathrm{k}$

$P = 0.9D - E = 29.2\ \mathrm{k}$

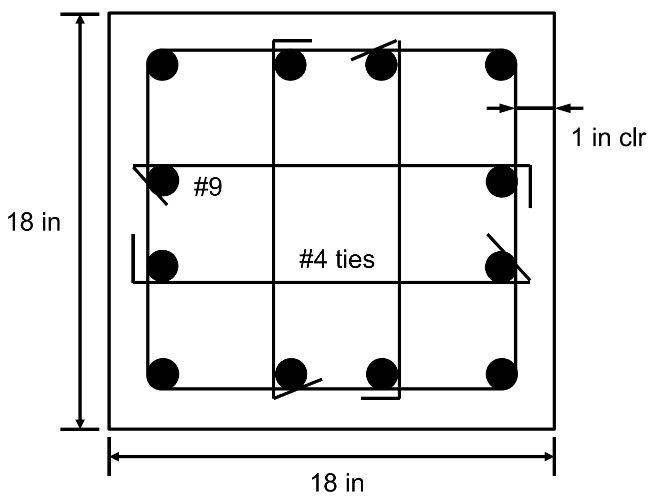
$M = 1.2D + 0.5L + E = 345\ \mathrm{k-ft}$

$M = 0.9D - E = 333\ \mathrm{k-ft}$

Note: Simple consideration of maximum and minimum P with associated M or maximum and minimum M with associated P may be inadequate. All combinations of P and M must be checked. In this problem, the controlling cases were clear from inspection.

Gravity

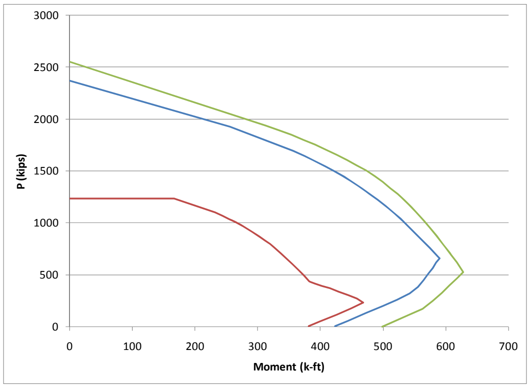
Column



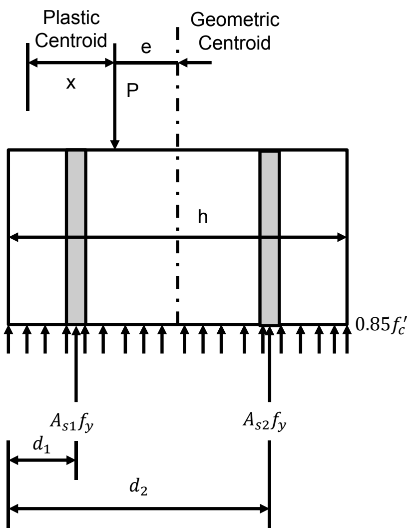
Use an 18 x 18 column with 12 #9 bars. The ties are #4 bars. Check 18.7.4.1: $\rho_g = 12(1)/18^2 = 0.037$; this is between 0.01 and 0.06 as required. (For gravity the limits are 0.01 and 0.08).

Column

Now we have to construct 3 separate interactions diagrams. First, we must construct the nominal interaction diagram with $f_y = 60\ \mathrm{ksi}$. Then, we reduce that by phi to get the design interaction diagram. Finally, we recalculate the nominal diagram with $1.25 f_y = 75\ \mathrm{ksi}$ to get the probable interaction diagram.



Column



First, find the plastic centroid. This is where the load "P" must be placed such that there is no moment.

The entire cross section is stressed to $0.85f_c'$ and the steel is assumed to have a stress of f_y. Find P (next slide). Next, sum the moments. For a rectangular column:

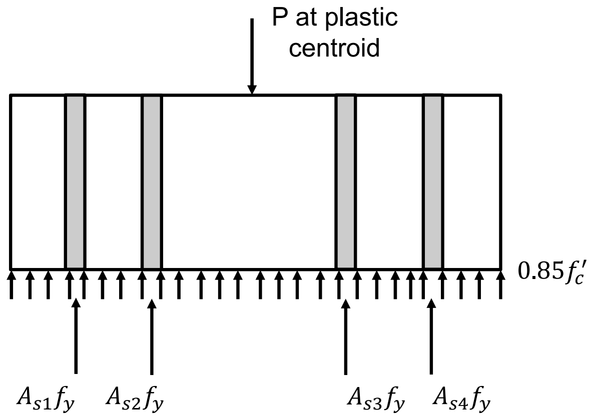
$$Px = 0.85 f_c' (A_c)\left(\frac{h}{2}\right) + \sum A_s f_y d$$

If the column is symmetrical, the plastic centroid is at the geometric centroid.

Column

Now, find P_n. Assume the entire cross section is stressed to $0.85f_c'$ and the steel is assumed to have a stress of f_y.

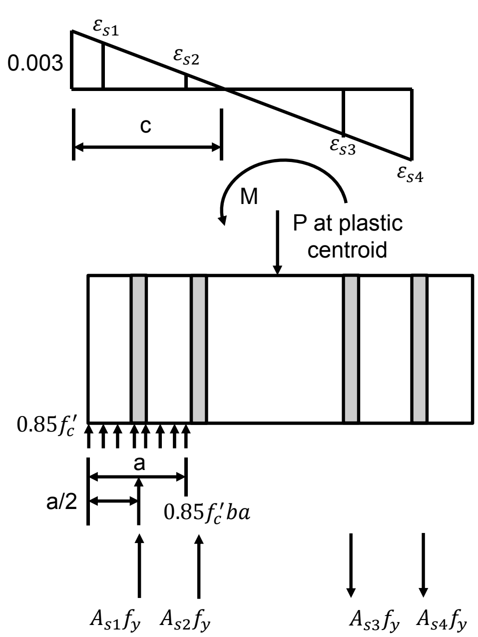
Gravity



$$P = 0.85(6\ \text{ksi})(324\ \text{in}^2 - 12\ \text{in}^2) + 12\ \text{in}^2(60\ \text{ksi}) = 2{,}311\ \text{k}$$

Column

Gravity



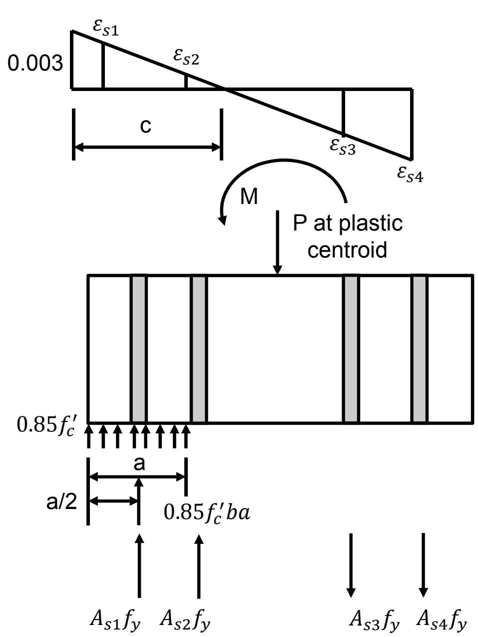
For the rest of the diagram,

1) Set the extreme compressive strain = 0.003
2) Assume a value of "c".
3) $a = \beta_1 c \leq h$
4) Using similar triangles, find the strain in each layer of steel.
5) Steel stress, $f_s = E_s\varepsilon_s \leq f_y$
6) Sum the forces to get P
7) Sum the moments about the plastic centroid to get M.
8) Repeat

Column

c (in)	$\varepsilon_{s4} = \varepsilon_t$	P kips	M k-ft	Comment
18	-0.0003	1,690	357	c = h
16	0	1,495	423	Zero tension in extreme tensile steel
12	+0.001	1,040	528	Tens Steel ≈ 30ksi
9.4	+0.0021	660	590	Balanced ($\varepsilon_t = \varepsilon_y$)
6	+0.005	261	522	Tens Control ($\varepsilon_t = 0.005$)
4.2	+0.008	0	423	Pure Bending (P = 0)

Column



Sample Calculation

Use c = 10

$a = \beta_1 c = 0.75(10) = 7.5 \text{ in} < h$

$F_c = 0.85(6 \text{ ksi})(7.5 \text{ in})(18 \text{ in}) = 689 \text{ k}$

$\varepsilon_{s4} = 0.003\left(\frac{16}{10} - 1\right) = 0.0018$

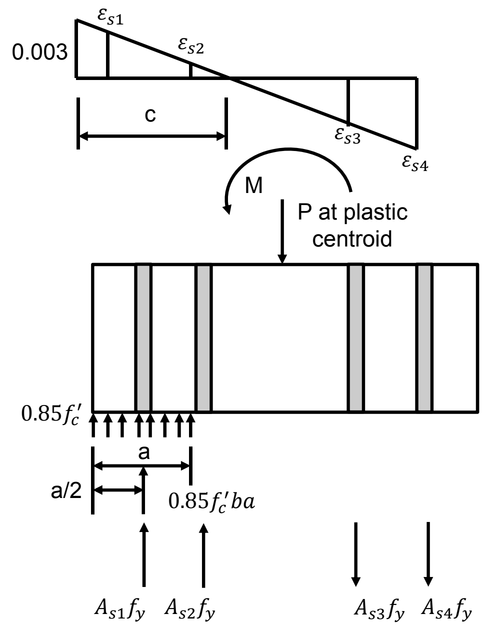
$f_{s4} = 29{,}000 \text{ ksi}(0.0018) = 52.2 \text{ ksi}$

$\varepsilon_{s3} = 0.003\left(\frac{11.3}{10} - 1\right) = 0.00039$

$f_{s3} = 29{,}000 \text{ ksi}(0.00039) = 11.3 \text{ ksi}$

Column

Gravity



Sample Calculation

Use c = 10

$$\varepsilon_{s2} = 0.003\left(\frac{6.7}{10} - 1\right) = -0.001$$

$$f_{s2} = 29{,}000\ \text{ksi}(-0.001) = -29\ \text{ksi}$$

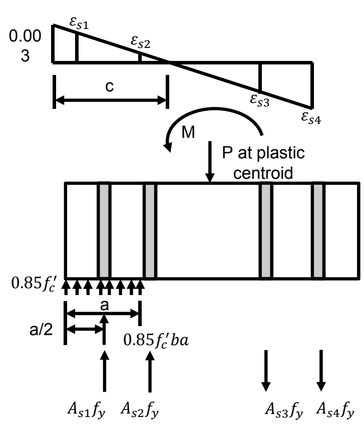
$$\varepsilon_{s1} = 0.003\left(\frac{2.0}{10} - 1\right) = 0.0024$$

$$\varepsilon_{s1} > \varepsilon_y = 0.0021$$

$$f_{s1} = -60\ \text{ksi}$$

Column

Gravity



Sample Calculation

Use c = 10 (Moments are summed about the plastic centroid or you must include P in the calculation.)

(Note: technically, you should subtract out the steel from the concrete area, but it only makes about 2% difference!)

$$P = 689\ \text{k} + 4(60) + 2(29) - 2(11.3) - 4(52.2) = 756\ \text{k}$$

$$M = -689\left(\frac{7.5}{2} - 9\right) - 4(60)(2.0 - 9) - 2(29)(6.7 - 9)$$

$$+2(11.3)(11.3 - 9) + 4(52.2)(16 - 9)$$

$$M = 6{,}944\ \text{k} - \text{in} = 578\ \text{k} - \text{ft}$$

Column

Now reduce by the phi factor:

$P_{max} = 0.8\Phi P_n = 0.52 P_n$ for tied columns.

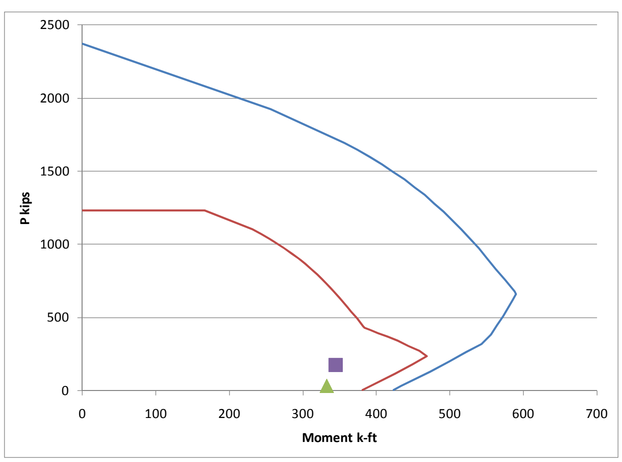
$P_{max} = 0.52(2{,}311\ \mathrm{k}) = 1{,}295\ \mathrm{k}$

For values of extreme tensile steel $< \varepsilon_y\ \phi = 0.65$

For values of extreme tensile steel $> 0.005\ \phi = 0.9$

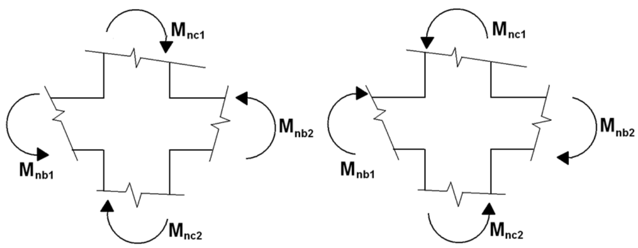
Interpolate in between

Column



Here are the nominal interaction diagrams. The points represent maximum and minimum P with associated moments; 174 k, 345 k-ft and 30 k, 333 k-ft. Both are within the reduced curve, so the column is adequate.

Special Seismic Frame Philosophy

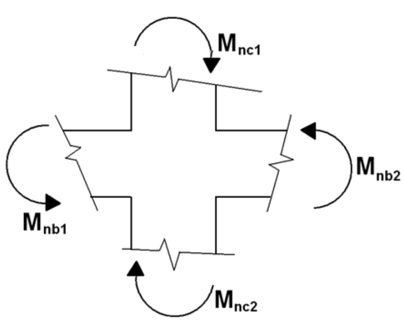


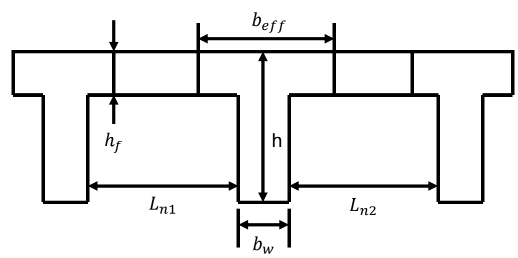
We want the hinges to form by yielding the **beams**, so ACI 18.7.3 **requires**:

$$\boxed{\sum M_{nc} \geq 1.2 \sum M_{nb}}$$

Note that these are the **nominal** moment strengths of the beams and columns. Note that M_{nb} must consider "T" beam flanges (18.7.3.2).

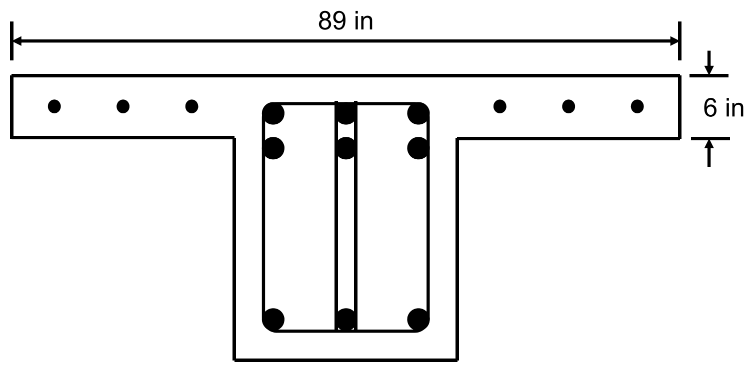
Special Moment Frames Columns 18.7.3.2





M_{nb} **must** include any slab reinforcing in the effective flange width! Effective flange width in tension is defined as the same a flange width in compression (6.3.2.1).

Special Moment Frames Columns 18.7.3.2



ϕM_{nb} for this section, both positive and negative, was found previously.

Special Moment Frames Columns 18.7.3.2

So the nominal strength of the beams is:

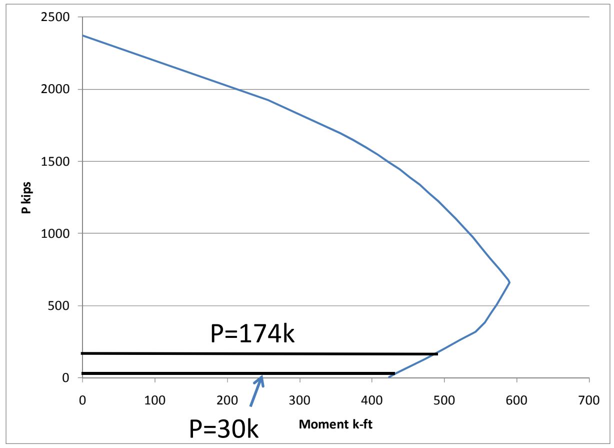
$$\phi M_{nb}^{(+)} = 179\ k - ft$$

$$\phi M_{nb}^{(-)} = 370\ k - ft$$

$$\phi M_{nb}^{(+)} + \phi M_{nb}^{(-)} = (179\ k - ft + 370\ k - ft)/0.9 = 610\ k - ft$$

The ϕ must be divided out since the equation requires M_{nb} not ϕM_{nb}.

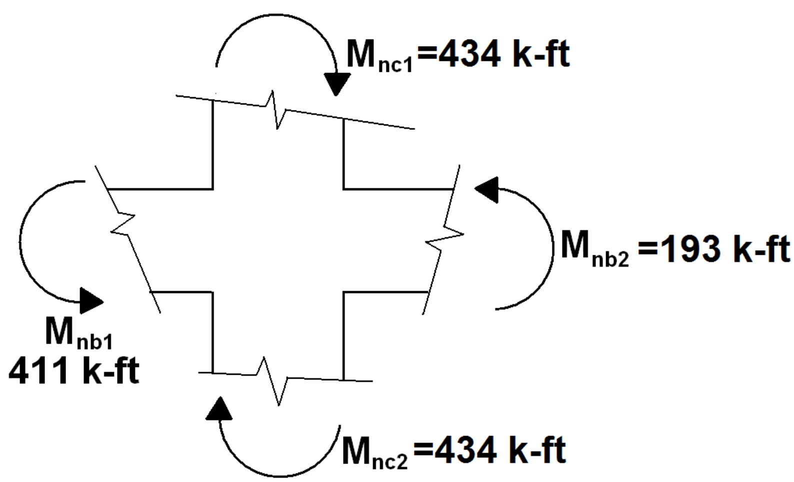
Special Moment Frames Columns 18.7.3.2



Find M_n for the column. That depends on P_u. Using the **nominal** interaction diagram, find M for the maximum P (174 k) and the minimum P (30 k).

Use the smallest moment (434 k ft @ 30 k).

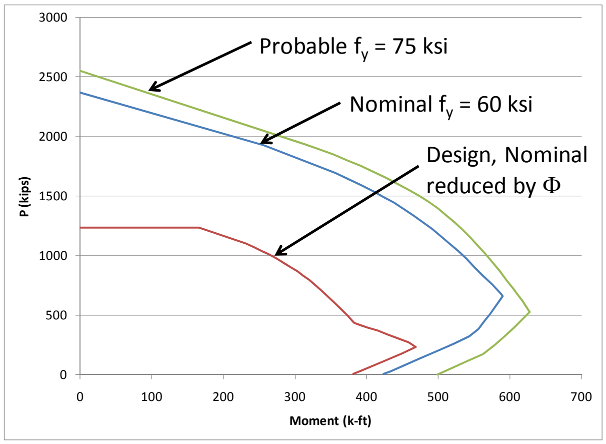
Special Moment Frames Columns 18.7.3.2



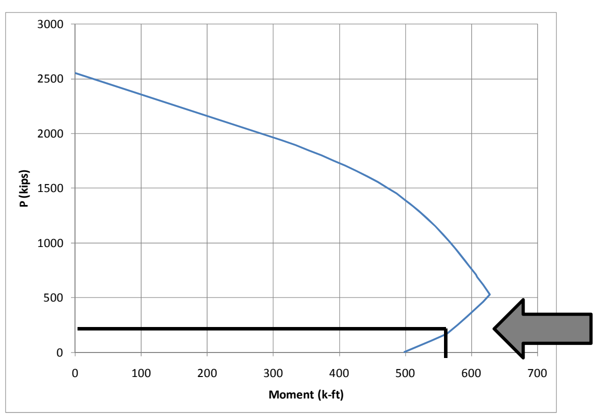
$$1.2 \sum M_{nb} = 732\ \mathrm{k-ft} < \sum M_{nc} = 868\ \mathrm{k-ft}\ \text{OK}$$

Special Moment Frames Columns 18.7.3.2

Now for shear: We must construct the **probable** interaction diagram for the column. We assume $f_y = 1.25(60) = 75 \text{ ksi}$. Note that we cannot just multiply the nominal by 1.25; the steel strength increases, not the concrete.



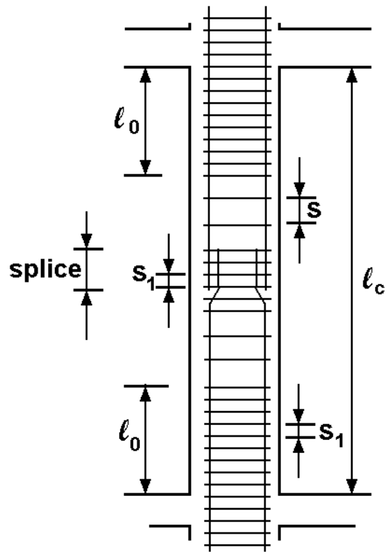
Special Moment Frames Columns 18.7.3.2



The probable interaction diagram is constructed using the same method as the nominal, but uses $1.25 f_y$.

The maximum possible probable moment is 627 k-ft, but 18.7.6.1.1 says "over the range of P_u". The largest moment is for $P_u = 174\ k$, M_{pr} is approximately 565 k-ft.

Special Moment Frames Columns



Column transverse steel requirements.

18.7.5.1 First hoop $s_1/2$ from face of joint.

ℓ_o is the greatest of: 18 in; c_1 or $\ell_c/6$

s_1 is the least of $6d_b$, $c_1/4$, $c_2/$ or

$$4" \leq s_o = 4 + \left(\frac{14 - h_x}{3}\right) \leq 6“$$

The term h_x is the maximum c/c spacing of the longitudinal bars laterally supported by hoop corners or cross ties around column perimeter.

$s \leq$ lesser of $6d_b$ or 6 inches

Or as required to enclose a mechanical or welded splice or as req'd for shear strength.

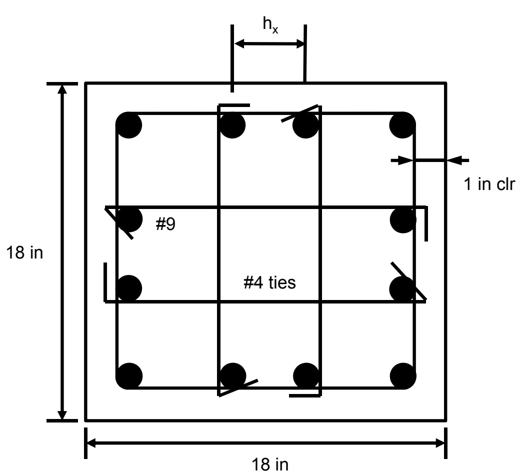
Column Transverse Reinforcement 18.7.5.2

- Must be single or overlapping spiral, circular hoops or rectilinear ties or cross ties.
- Bends of rectilinear hoops must engage peripheral longitudinal bars.
- Cross ties of the same or smaller bar are permitted subject to limitations of 25.7.2.2. Cross tie ends must alternate end for end along the longitudinal reinforcement and around the perimeter of the column.

Column Transverse Reinforcement 18.7.5.2

- Rectilinear hoops or cross ties must provide lateral support to longitudinal bars according to 25.7.2.2 and 3.
- The term $h_x \leq 14$ inches.
- If $P_u > 0.3f_c'A_g$ or $f_c' > 10{,}000\ \mathrm{psi}$, in columns with rectilinear hoops, every longitudinal bar must have lateral support and $h_x \leq 8\ \mathrm{in}$. P_u is largest value of P_u for any factored load combination containing E.

Column Cross Ties



Term h_x: Column width, minus cover and ties, minus one d_b divided by spaces.

The term h_x is the c/c distance between longitudinally supported bars.

$$h_x = \frac{18 - 2(2) - 2(0.5) - 1.125}{3\ spaces}$$

$$= 4.6\ in$$

This column has every bar supported so it meets 18.7.5.2 regardless of the value of P_u.

From the earthquake load combination $P_u = 174\ \mathrm{kips}$.

$0.3(6\ \mathrm{ksi})(18\ \mathrm{in})(18\ \mathrm{in}) = 583\ \mathrm{k} > 174\ \mathrm{k}$

And $f_c' = 6{,}000\ \mathrm{psi} < 10{,}000\ \mathrm{psi}$.

Column Cross Ties

This column has every bar supported so it meets 18.7.5.2 regardless of the value of P_u:

$$s_0 = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 4.6}{3}\right) = 7.1 > 6$$

Maximum spacing:

$s_0 = 6 \text{ in}$

$or\ c_1/4$ or $c_2/4 = 18/4 = 4.5 \text{ in}$ Controls for s_o

$or\ 6d_b = 6(1.125) = 6.75$

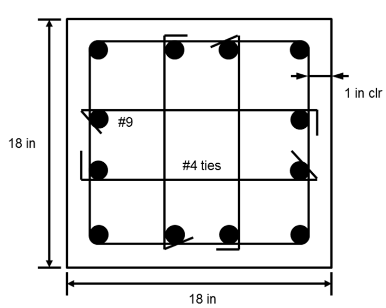
$\ell_o = \text{clear span}/6 = 120 \text{ in}/6 = 20 \text{ in}$

or $c_1 = 18$ in or 18 in min

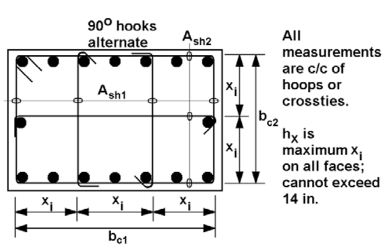
20 in controls (use max).

Round down s_o to 4 inch spacing over at least $\ell_o = 20$ in and at splice.

Column Cross Ties



Definitions:



Minimum tie area (18.7.5.4) for rectilinear ties. A_{ch} is the core area out-to-out of ties.

$A_{ch} = (18 - 2)(18 - 2) = 256 \text{ in}^2$

$A_g = 18^2 = 324 \text{ in}^2$

$b_{c1} = b_{c2} = 18 - 2 - 0.5 = 15.5 \text{ in}$

$$A_{shmin} = 0.3 s b_c \frac{f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}} - 1\right)$$

$$A_{shmin} = 0.3(4)(15.5)\frac{6}{60}\left(\frac{324}{256} - 1\right) = 0.50 \text{ in}^2$$

$$A_{shmin} = 0.09 s b_c \frac{f_c'}{f_{yt}} = 0.09(4)(15.5)\frac{6}{60} = 0.56 \text{ in}^2$$

$A_{sh,provided} = 0.80 \text{ in}^2$ OK

Column Cross Ties

If $P_u \geq 0.3A_g f_c'$ or $f_c' \geq 10{,}000$ psi, a third equation must be checked:

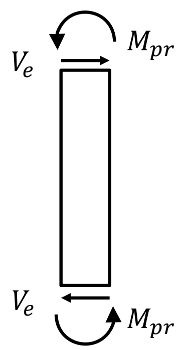
$$A_{sh} \geq 0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}} s b_c$$

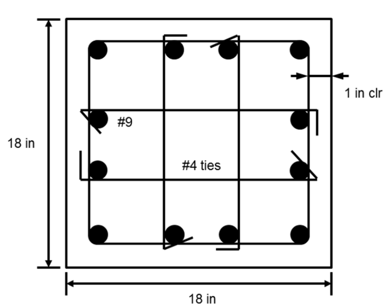
$$k_f = \frac{f_c'}{25000} + 0.6 \leq 1.0$$

$$k_n = \frac{n_i}{n_i - 2}$$

The term n_i is the number of longitudinal bars around the perimeter with rectilinear hoops that are laterally supported by corners or seismic hoops. This is new in ACI 318-14.

Column





18.7.6.2.1 requires V_c taken = 0 within ℓ_o when $V_e > 0.5V_u$ within ℓ_o <u>and</u> $P_u < A_g f_c'/20$.

$$V_e = \frac{2(565\ \text{k–ft})}{10\ \text{ft}} = 113\ \text{k}$$

$$P_{min} = 30\ \text{k} < 0.05A_g f_c' = 97\ \text{k}$$

M_{pr} max. corresponding to range of P_u previously found as 565 k-ft.

$$V_c = 0$$

Stirrups = #4 at 4

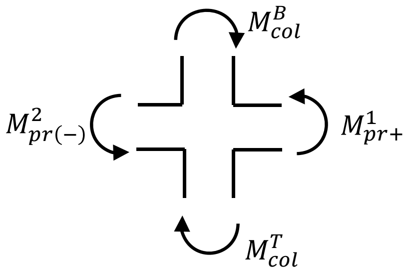
$$\phi V_s = (0.75)\frac{0.8\ \text{in}^2(60\ \text{ksi})(15.9\ \text{in})}{4\ \text{in}} = 143\ \text{k}$$

$$\phi(V_c + V_s) = 143\ \text{k} > V_e = 113\ \text{k}$$

V_e must be > factored $V_{analysis}$

V_e constant, so #4 at 4 throughout column.

Column



$$M_{top} = \frac{\left(\frac{EI}{\ell_c}\right)_{top}}{\left(\frac{EI}{\ell_c}\right)_{top} + \left(\frac{EI}{\ell_c}\right)_{bot}} \left(M^1_{pr+} + M^2_{pr(-)}\right)$$

$$M_{bot} = \frac{\left(\frac{EI}{\ell_c}\right)_{bot}}{\left(\frac{EI}{\ell_c}\right)_{top} + \left(\frac{EI}{\ell_c}\right)_{bot}} \left(M^1_{pr+} + M^2_{pr(-)}\right)$$

There is an alternative way to find V_e:

18.7.6.1.1 says the shear at the end of the column does not need to exceed that calculated from resistance the beams can provide.

Recall that the sum of the beam probable moments was

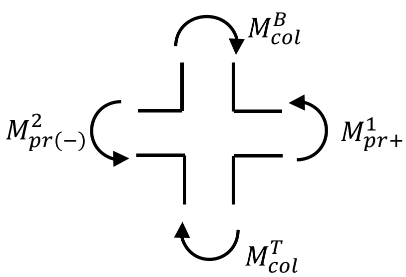
$199 + 411 = 610\ \mathrm{k-ft}$

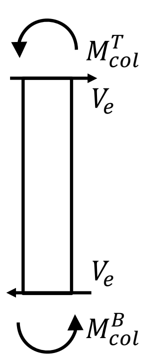
These moments can be distributed to the columns above and below the beam according to the relative stiffness of the columns, as shown in the formula to the left.

If the columns above and below the beam have the same stiffness:

$M^T_{col} = M^B_{col} = 610/2 = 305\ \mathrm{k-ft}$

Column





Next, you would have to repeat this calculation at the joint above (for the top column) and below (for the bottom column).

For the sake of illustration, assume the moment at the bottom of the lower column is 330 k-ft.

So, $V_e = (305 + 330)/10 = 63.5\ \mathrm{k}$

The stirrup calculation is done the same as before, but the smaller V allows a larger stirrup spacing outside of ℓ_o.

Column Splice

Column Splice Length

18.7.4.3 requires:

Mechanical splices conform to 18.2.7 and welded splices conform to 18.2.8.

Lap splices are permitted only within the center half of the column, must be tension splices and must be enclosed within transverse reinforcement according to 18.7.5.2 and 3 (see previous calculations for ℓ_o ties).

Column Splice

s = 4 inches controls.

$$K_{tr} = \frac{40(0.8\text{ in}^2)}{4\text{ in}(4\text{ bars})} = 2$$

$$c_b = 2.1\ inches$$

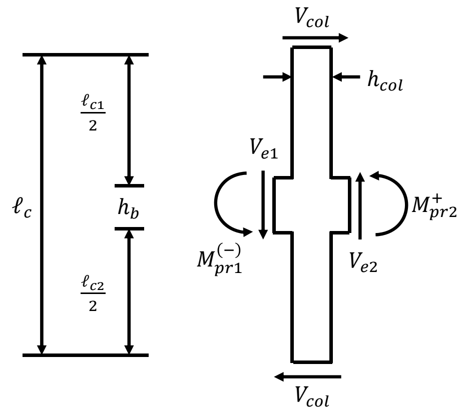
All values of Ψ and λ = 1.

$$\frac{c_b + K_{tr}}{d_b} = \frac{2.10+2}{1.125} = 3.6 > 2.5$$

$$\ell_d = \frac{3}{40}\frac{60000}{\sqrt{6000}(2.5)}(1.125) = 26.0\text{ in}$$

$1.3\ell_d = 1.3(26.0) \approx 34\text{ in}$ Class B splice

Special Moment Frames Joints



To determine joint shear, we use the probable moments including the slab. The beam shears are from beam design. The sum of the probable beam moments is 681 k-ft and the beam shear was 27.2 k.

$$V_{col} = \left[\left(M_{pr}^{(-)} + M_{pr}^{(+)}\right) + (V_{e1} + V_{e2})\frac{c_1}{2}\right]\frac{1}{\ell_c}$$

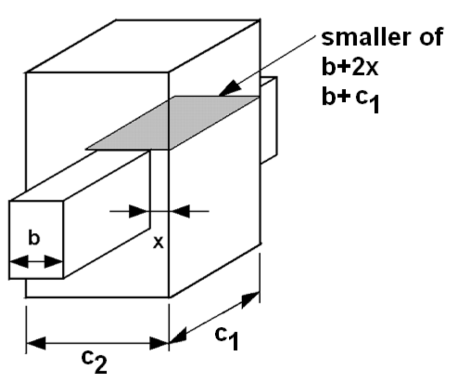
$$\ell_c = 2\frac{10}{2} + 2 = 12 \text{ ft}$$

$$V_{col} = \left[681 \text{ k} - \text{ft} + 2(27.2 \text{ k})\frac{1.5 \text{ ft}}{2}\right]\frac{1}{12 \text{ ft}} = 60 \text{ k}$$

Special Moment Frames Joints

18.8.4.3 defines the shear area of the joint, A_j.

If the beam width is not smaller that the column: $A_j = c_1 c_2$

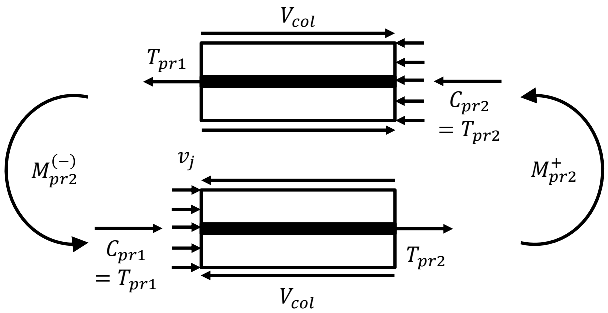


If the beam is smaller than the column, the depth of the joint is still c_1. The width is the smaller of

$$\boldsymbol{c_1 + b_{beam} \leq b_{beam} + 2x \leq c_2}$$

where x is the smallest distance from the face of the beam to the face of the column.

Special Moment Frames Joints



$$V_j = T_{pr2} + C_{pr1} - V_{col}$$

$$V_j = C_{pr2} + T_{pr1} - V_{col}$$

$$T_{pr1} = 3.6\ \text{in}^2(75\ \text{ksi}) = 270\ \text{k}$$

$$T_{pr2} = 1.8\ \text{in}^2(75\ \text{ksi}) = 135\ \text{k}$$

$$V_j = 270 + 135 - 60 = 345\ \text{k}$$

18.8.4.3: The joint is $c_1 = 18$ in long. The width is:

Smallest of $b + c_1 = 14 + 18 = 32\ in$ (b is beam width)
$b + 2x = 14 + 2 + 2 = 18\ in \leq c_2$

(x is the shortest distance from the edge of the beam to the column face.)

ACI 318 does not require considering the slab steel, but some designers do.

Special Moment Frames Joints

18.8.4.1:

If the joint is confined on all four faces:

$$V_n = 20\sqrt{f_c'}A_j$$

If the joint is confined on 3 faces or 2 opposite faces:

$$V_n = 15\sqrt{f_c'}A_j$$

All other cases:

$$V_n = 12\sqrt{f_c'}A_j$$

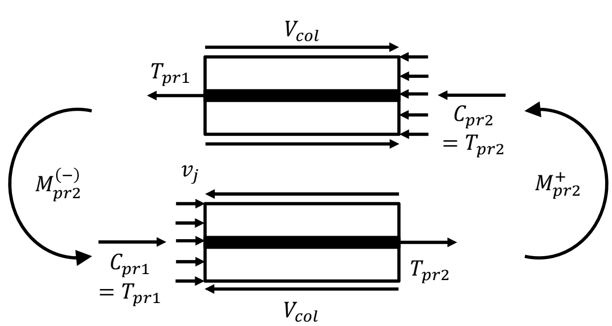
For lightweight concrete, multiply the limits above by 0.75.

Special Moment Frames Joints

18.8.4.2:

- Members that frame into a joint provide confinement **if** the framing member covers at least ¾ of the face of the joint.
- Extensions of beams at least overall beam depth h beyond the joint face may be considered as confining members if the extension meets:
  - $b_w > 0.3\text{ h} > 10\text{ in}$ (18.6.2.1b)
  - Have minimum flexural reinforcing (18.6.3.1)
  - Required hoops (18.6.4.2, 3 and 4)

Special Moment Frames Joints



Joint is confined by beams on all 4 faces.

$$V_j = 345\text{ k}$$

$$A_j = 18\text{ in}(18\text{ in}) = 324\text{ in}^2$$

$$V_n = 20\sqrt{6{,}000\text{ psi}}(324\text{ in}^2)$$

$$= 502{,}000\text{ pounds}$$

$$\phi V_n = 0.75(502\text{ k}) = 377\text{ k} > 345\text{ k}$$

Special Moment Frames Joints

18.8.3.1:

- Transverse Reinforcement
  - Provide transverse hoop reinforcement, the same as in the ℓ_o region (18.7.5.2, 3, 4 and 7) in the joints unless the joint is confined by structural members (see next slide).

Special Moment Frames Joints

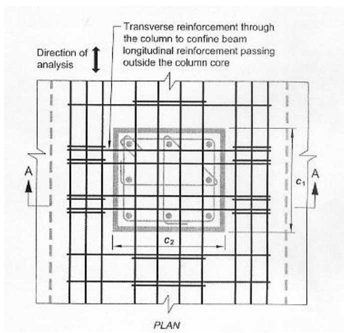
18.8.3.2:

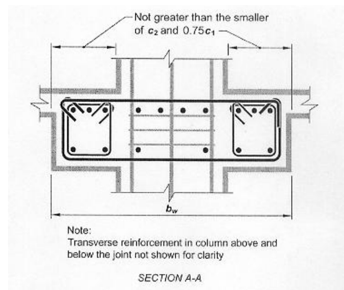
- Transverse Reinforcement
  - If the joint is confined by structural members on all 4 sides and the member width is at least ¾ of the column width, the amount of hoop reinforcing may be reduced by ½ and the spacing may be increased to 6 inches. This may be done within h of the shallowest beam.

Special Moment Frames Joints

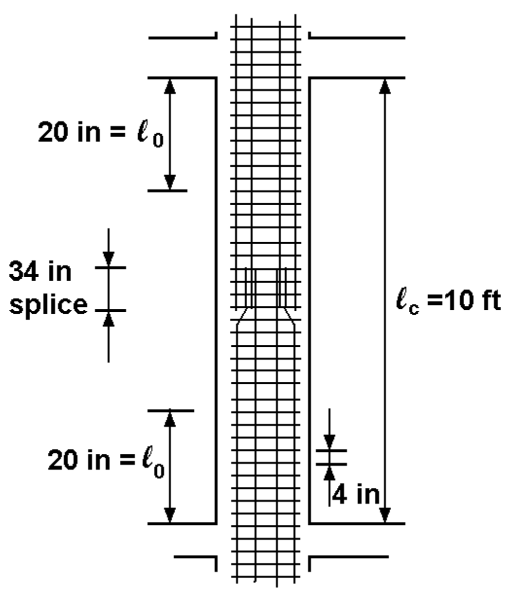
18.8.3.3:

- Transverse Reinforcement
  - Longitudinal beam reinforcement outside the column core shall be confined by transverse steel passing through the column that satisfies spacing requirements of 18.6.4.4, and requirements of 21.6.4.2 and 3.



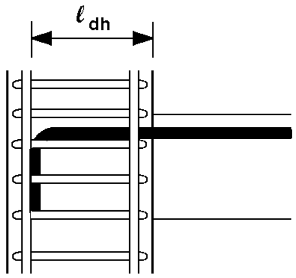


Final Column Design



Development Length Within a Joint

18.8.5.1 For #3 - #11 bar, the development length for a standard, 90° hook in tension shall not be less than the largest of:



$$\ell_{dh} = \begin{cases} 6\ in \\ 8d_b \\ \dfrac{f_y d_b}{65\sqrt{f_c'}} \end{cases}$$

Hook must be confined within the confined column core! It is good detailing to make the hook go all the way through the column.

Development Length Within a Joint

18.8.5.1 For #3 - #11 bar, the development length for a standard, 90° hook in tension **in lightweight concrete** shall not be less than the largest of:

$$\ell_{dh} = \begin{cases} 7.5\ in \\ 10d_b \\ \dfrac{f_y d_b}{65\lambda\sqrt{f_c'}} \end{cases}$$

Take $\lambda = 0.75$ for lightweight concrete.

Development Length Within a Joint

- 18.8.5.3: The development length, ℓ_d, for straight bar, #3 - #11:
  - If the depth of the concrete poured in a single lift below the bar is 12 inches or less, $2.5\ell_{dh}$ from the previous two slides.
  - If the depth of the concrete poured in a single lift below the bar is greater than 12 inches, $3.25\ell_{dh}$ from the previous two slides.

Development Length Within a Joint

- 18.8.5.4: If a straight bar is terminated at a joint
  - It shall pass through the confined core of the column or boundary element
  - Any part of ℓ_d outside of the confined core shall be increased by 1.6.
- Epoxy coated bars or hooks shall be multiplied by the epoxy bar factors in 25.4.2.4 (straight bar) or 25.4.3.2 (hooks).

Development Length Within a Joint

- Epoxy bar factor 25.4.2.4 (straight bar)
  - 1.2
  - 1.5 if the cover is less than $3d_b$ or the clear spacing is less than $6d_b$
- Epoxy bar factor 25.4.3.2 (hooks)
  - 1.2

Development Length Within a Joint

$$\ell_{dh} = \frac{60{,}000(0.875)}{65\sqrt{6{,}000}} = 10.5 \text{ in} > 8d_b = 7 > 6 \text{ in}$$

$$\ell_d = 2.5(10.5) = 26 \text{ in}$$

$$\ell_{d,topbar} = 3.25(10.5) = 33.9 \text{ in}$$

So at the end of the building, the hooks must be embedded 10.5 inches into the columns.

The bars going through the interior columns must have the development lengths shown; 26 inches bottom bars, 34 inches top bar.

Questions

Thank you!

Richard Miller
University of Cincinnati
Richard.Miller@uc.edu