Walking Columns –
A Proposed Design Methodology

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PHILADELPHIA | NEW YORK

DVASE | Delaware Valley Association of Structural Engineers
Eastern Chapter of the Structural Engineers Association of Pennsylvania

1

Seminar objectives

Review

- Walking / sloping column design and details
- Node analysis, design and detailing
- Tension and compression strut design
- Diaphragm design and load path issues
- Diaphragm load transfer to shear walls
- Things to watch out for

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2

Design methodology

There is no ACI 318 design methodology for designing walking columns. Therefore,

- Procedures discussed in this webinar are conservative.
- Conservative approaches will not adversely affect overall cost.
- Multiple (redundant) approaches are used for some checks.
- Conservative approximations are used to facilitate (speed up) design.

Caution: Use this design procedure at your own risk. It has not been peer reviewed and is not recognized by ACI. The procedure is based on structural engineering fundamentals and continues to be a “work in progress”. Your walking columns may have unique conditions that are not considered in this presentation.

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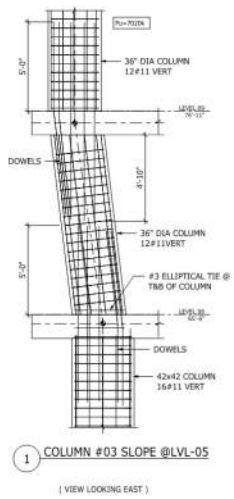
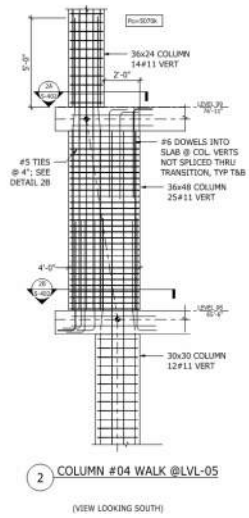
3

What is a walking column?



Sloping column



Walking column

(For this presentation, the term “walking column” usually applies to both walking and sloping columns.)

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Sloping columns

Seminole Hard Rock
Hotel & Casino,
Hollywood, FL

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Walking columns

Vancouver House, Vancouver, British Columbia, Canada

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When to walk? When to slope?

Walking columns are usually simpler, less expensive, and easier to build than sloping columns.

- Walking columns have easier formwork
- Walking columns have simpler column rebar details
- Sloping columns are usually dictated by the architect (when the columns are architecturally exposed).

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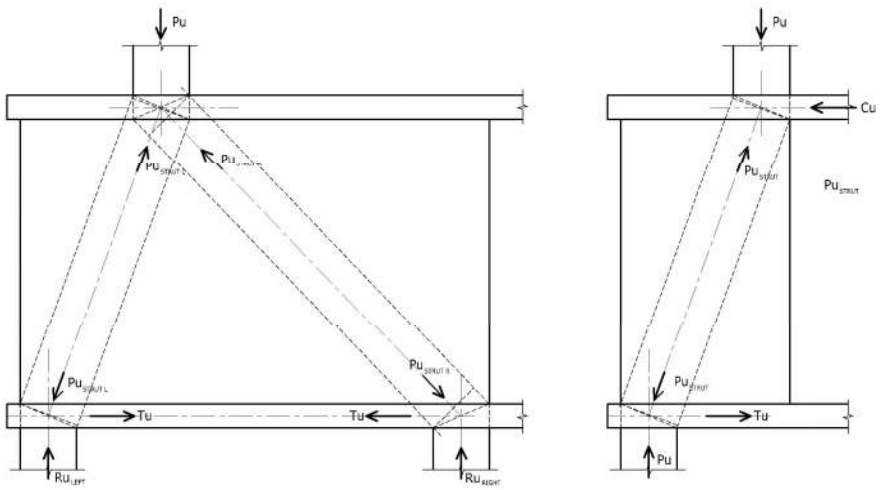
8

Walking column design is similar to deep beam strut and tie design



Deep beam

Walking column

Things to watch out for

- Walking columns can split or crack if not properly reinforced.
- Walking columns can impose big horizontal forces in floor diaphragms and shear walls.
- Reinforcing details at the nodes are critical.
- Diaphragm strut design is critical (especially the tension struts).
- Legitimate and continuous load paths in floor diaphragms are critical. (Complications at steps and openings in floor slabs.)
- Software may not properly consider, analyze, or design,
  - The walking columns
  - Horizontal forces, moments, and shears in floor diaphragms
  - Additional forces in shear walls induced by the walking columns
  - Load path issues in columns, slab struts, diaphragms and shear walls
  - Rebar details at nodes, struts, floor diaphragms and shear walls
  - Tension and compression struts in floor diaphragms
  - Anomalies in diaphragms resulting from slab openings, slab steps, etc.
- **<u>There is little or no redundancy</u> in diaphragm tension/compression strut load paths.**

Rule-of-thumb: Be conservative!

Rules-of-thumb for walking columns

- Provide adequate column overlap (plan view) above and below slab. (Avoid excessive overhang between upper and lower columns.)
- Walk columns over two or more floors to ensure adequate vertical shear strength in column and to minimize horizontal strut forces.
- Rule-of-thumb for maximum walk/slope: 1 horizontal : 6 vertical
- Lightly loaded columns can walk more than heavily loaded columns.
- The larger the walk, the larger the diaphragm strut forces.
- Strive to avoid strut forces > 600k (although strut forces as high as 1,100k have been encountered)
- Large strut forces require a continuous each way top and bottom steel in slab diaphragm for diaphragm shear and for strut compression and tension.
- Large strut forces may require thicker floor slabs.
- Careful attention required where tension struts occur perpendicular to slab edges.
- Walking columns over several floors will reduce strut forces.
- Watch out for steps and large openings in floor diaphragms resisting walking column strut forces. (Steps and large openings will disrupt diaphragm strut load paths.)
- Do not rely on typical details for walking columns.

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11

Do not rely on typical details for design of walking columns.

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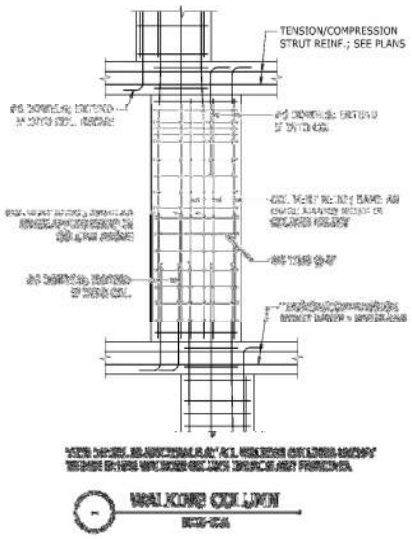
12

Do not rely on typical details



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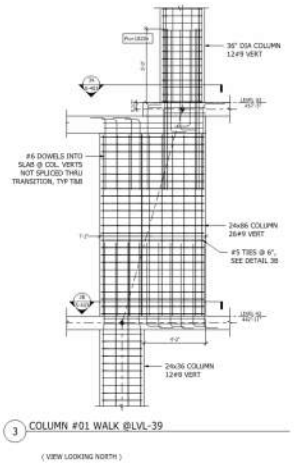
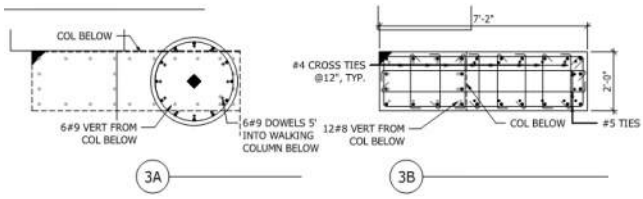
Some examples of walking columns





How do you put this (and the other examples that follow) in a typical detail?

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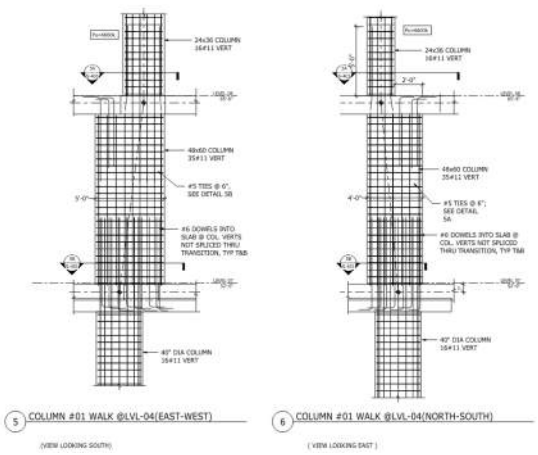
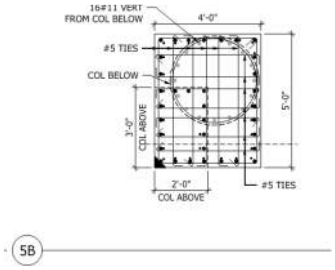
14

Walking column





Drawing column elevations helps designers spot problems and unique conditions. Vertical reinforcing configurations are often unique.

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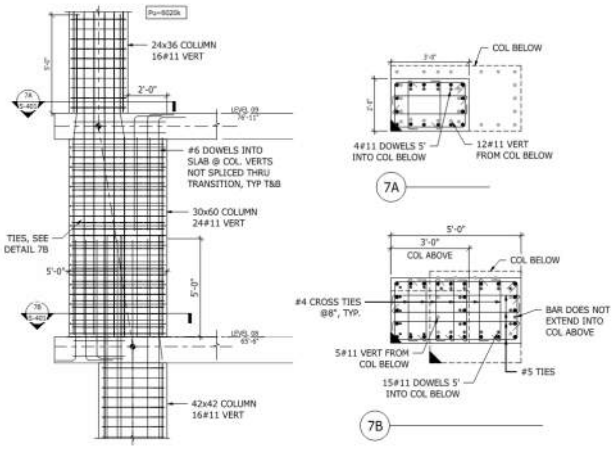
Walking column



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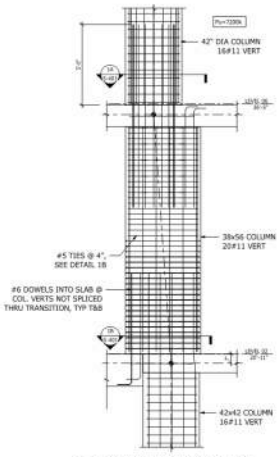
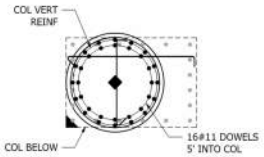
16

Walking column

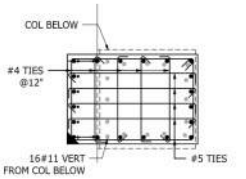


1 COLUMN #03 WALK @LVL-03

(VIEW LOOKING SOUTH)



1A



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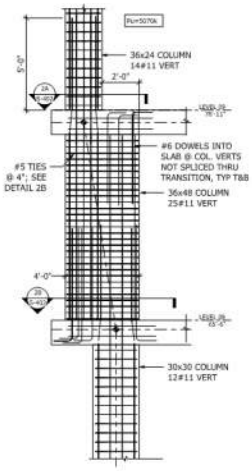
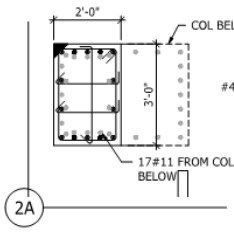
17

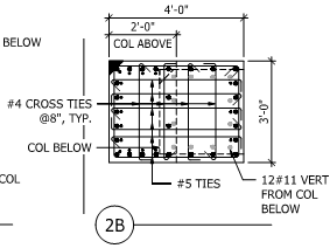
Walking column



2 COLUMN #04 WALK @LVL-05



2A



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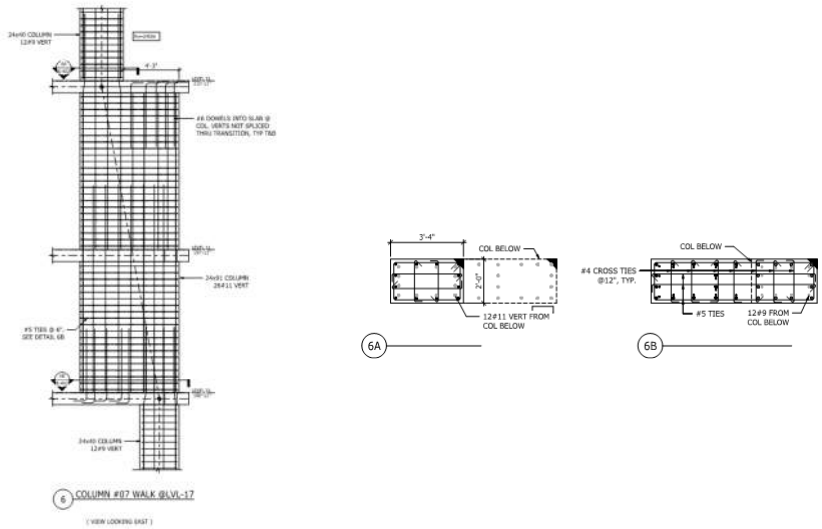
Walking column



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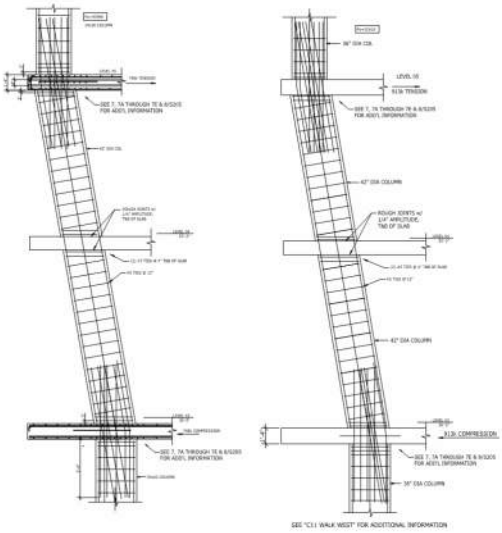
19

Sloping column



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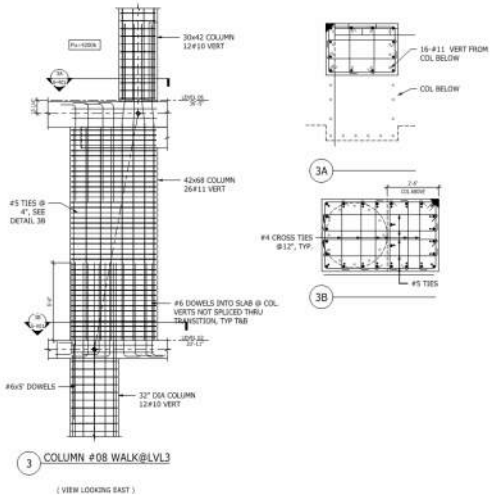
20

Walking column



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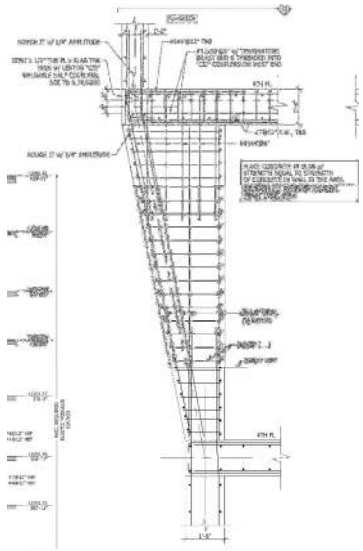
21

Walking wall



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Design Procedure

(Note: Most ACI 318 references in this presentation are to ACI 318-08. Procedure is the same for ACI 318-11, 14, and 19.)

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Design checklist

1. Draw column elevations
2. Calculate load in column, and in horizontal struts at top and bottom of walking column.
3. Check bearing stresses on slab at top and bottom of walking column
4. Check strut strength
5. Check node strength
6. Determine two internal column sizes
7. Calculate strengths of both internal columns (consider slenderness)
8. Determine max permitted ØVn = Ø10 x (fc')**$^{1/2}$ (Make column wider if required)
9. Calculate ØVc = Ø2 x (fc')**$^{1/2}$
10. Calculate required minimum shear deep beam reinforcement (horizontal and vertical)
11. Calculate provided shear strength of shear reinforcing
12. Verify that ØVc + ØVs ≥ Vu

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24

Design and detailing tips / rules-of-thumb

1. Space vertical bars in columns no farther apart than 10” (to prevent cracks).
2. Strive to align vertical bars in walking column with bars in columns above and below.
3. Space column ties at 6” o.c.
4. Use multiple legs of ties to increase shear strength
5. Limit size of ties to #5
6. Favor walking columns over sloping columns
7. Specify rough joints at slab column interface, but don’t account for the rough joint in the calculations.
8. Draw nodes to scale to verify that tension tie bars will be fully developed at edge columns
9. Punching shear strength of concrete (ØVc) will be zero where tension ties connect to walking columns and where tension forces enter the shear walls. Provide studrails in these zones.
10. Extend tension struts deep into the floor diaphragm and shear walls, keeping in mind that these are “tension ties”. Don’t rely on minimum class “B” tension lap splices - not permitted!
11. Weld tension ties to embedded plates where tension ties occur perpendicular to slab edges where tension forces are large and where distance to develop bars is small.
12. Provide transverse bars in slab to spread the tension strut load horizontally where large tension strut forces occur perpendicular to slab edges.

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25

Phi factors

- Column axial load design: Ø = 0.65
- Column bearing on slab: Ø = 0.65
- Tension ties/struts in slab: Ø = 0.75
- Compression struts in slab: Ø = 0.75
- Nodal zones: Ø = 0.75
- Shear calculations: Ø = 0.75
- Diaphragm flexural reinforcing steel calculations: Ø = 0.90

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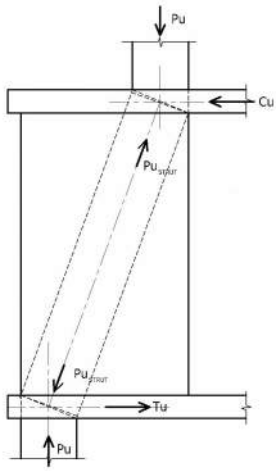
26

Draw column elevations for all walking columns



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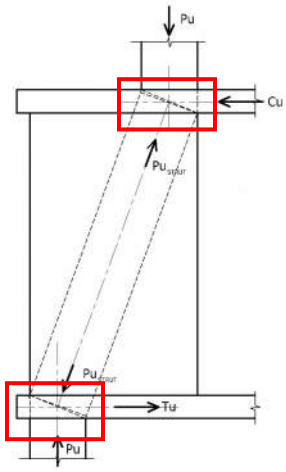
27

Check slab bearing strength at each end of walking column

Only the overlapping column areas can be used for calculating bearing strength.



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Check slab bearing strength at each end of walking column

10.14.1 — Design bearing strength of concrete shall not exceed $\phi(0.85f_c' A_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but by not more than 2.

15.3—Transfer of column axial force through the floor system

15.3.1 If f_c' of a column is greater than 1.4 times that of the floor system, transmission of axial force through the floor system shall be in accordance with (a), (b), or (c):

(a) Concrete of compressive strength specified for the column shall be placed in the floor at the column location. Column concrete shall extend outward at least 2 ft into the floor slab from face of column for the full depth of the slab and be integrated with floor concrete.

(b) Design strength of a column through a floor system shall be calculated using the lower value of concrete strength with vertical dowels and spirals as required to achieve adequate strength.

(c) For beam-column and slab-column joints that are restrained in accordance with 15.2.4 or 15.2.5, respectively, it shall be permitted to calculate the design strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength, where the value of column concrete strength shall not exceed 2.5 times the floor concrete strength.

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Check slab bearing strength at each end of walking column

Sample calculation

Slab: 6ksi
Column: 10ksi
80ksi reinf. steel

24x24 w/ 10 #11 V

UPPER COLUMN

$\phi P_N = .65(.8)(.85(10ksi)(24^2-15.6) + 80ksi(15.6))$ (4763, 1248)

$= 3126k > 3000k$ OK

UPPER COLUMN BEARING ON SLAB

$\phi P_N = .65[(.85)(6ksi)(24^2-15.6) + 80(15.6)]$ (2858, 1248)

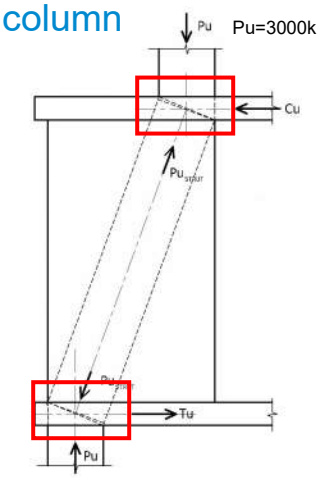
$= 2669k < 3000$ NO GOOD

EFFECTIVE COLUMN STRENGTH @ TOP OF WALK

$f'_{c\,eff} = .75(10) + .35(6) = 9.6$ ksi ← ACI 318-08 10.12.3

$\phi P_N = .65(.8)[(.85(9.6)(576-15.6) + 80(15.6)]$ (4525, 1248)

$= 3002k > 3000k$ OK



Can only use this equation where slab occurs on all sides around column and is adequately confined. (In this example there is adequate confinement on top, but not bottom.)

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30

Calculate vertical shear in column

Compute upper limit on maximum permitted vertical shear in column

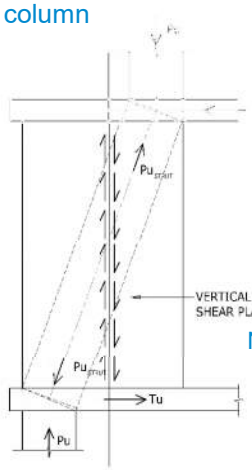
11.7.2 — Deep beams shall be designed using either nonlinear analysis as permitted in 10.7.1, or Appendix A.

11.7.3 — V_n for deep beams shall not exceed $10\sqrt{f'_c}\, b_w d$.

11.7.4 — The area of shear reinforcement perpendicular to the flexural tension reinforcement, A_v, shall not be less than $0.0025 b_w s$, and s shall not exceed the smaller of $d/5$ and 12 in.

11.7.5 — The area of shear reinforcement parallel to the flexural tension reinforcement, A_{vh}, shall not be less than $0.0015 b_w s_2$, and s_2 shall not exceed the smaller of $d/5$ and 12 in.

11.7.6 — It shall be permitted to provide reinforcement satisfying A.3.3 instead of the minimum horizontal and vertical reinforcement specified in 11.7.4 and 11.7.5.



No overlap

Make column thicker or walk over two floors if required.

31

31

Calculate upper limit on vertical shear strength of column

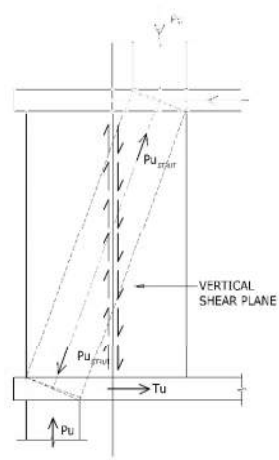
CHECK UPPER LIMIT ON SHEAR STRENGTH

$\phi 10\sqrt{f'_c}\, b_w d = .75(10)\sqrt{10000}\,(24")(120")$

$= 2160^K < 3000^K$ NO GOOD

REQUIRED WIDTH OF WALKING COL

$w_{REQD} = 24"\left(\frac{3000}{2160}\right) = 34"$



Make column thicker or walk over two floors if required.

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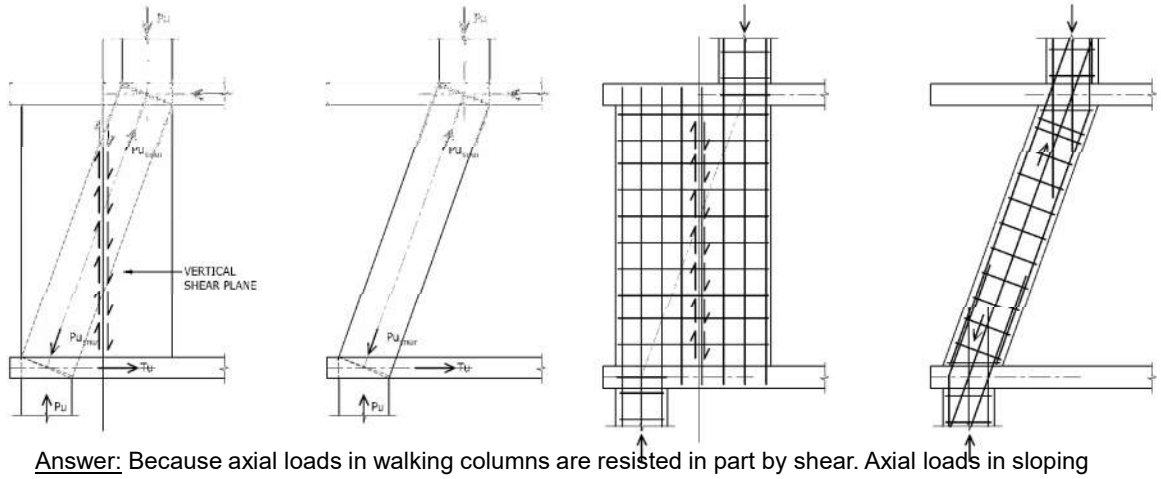
32

Question: Why check shear on walking columns, but not on sloping columns?



Answer: Because axial loads in walking columns are resisted in part by shear. Axial loads in sloping columns are detailed with compression steel aligned with the sloping column and aligned with the compression force vector. (Note: ACI 318 allows sloping compression steel to be used in walking columns, but such steel adds unnecessary complexity.)

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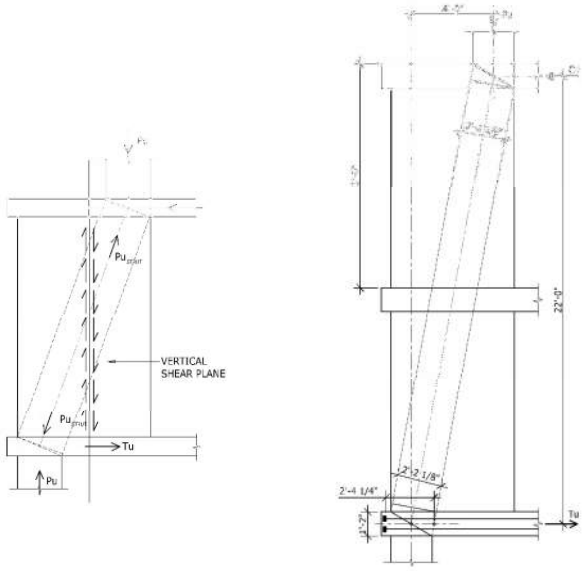
Walk column over two (or more) floors if required to increase maximum permitted shear strength (and reduce diaphragm strut forces).



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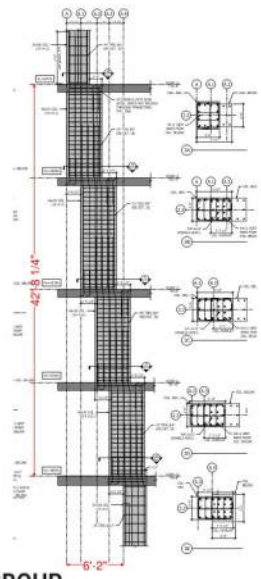
34

Multi-story column walk



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Calculate required minimum deep beam vertical and horizontal reinforcing in column for calculating strut strength

ACI 318-08:

11.7.2 — Deep beams shall be designed using either nonlinear analysis as permitted in 10.7.1, or Appendix A.

11.7.3 — V_n for deep beams shall not exceed $10\sqrt{f'_c}\, b_w d$.

Vertical bars →

11.7.4 — The area of shear reinforcement perpendicular to the flexural tension reinforcement, A_v, shall not be less than $0.0025 b_w s$, and s shall not exceed the smaller of $d/5$ and 12 in.

Horizontal bars →

11.7.5 — The area of shear reinforcement parallel to the flexural tension reinforcement, A_{vh}, shall not be less than $0.0015 b_w s_2$, and s_2 shall not exceed the smaller of $d/5$ and 12 in.

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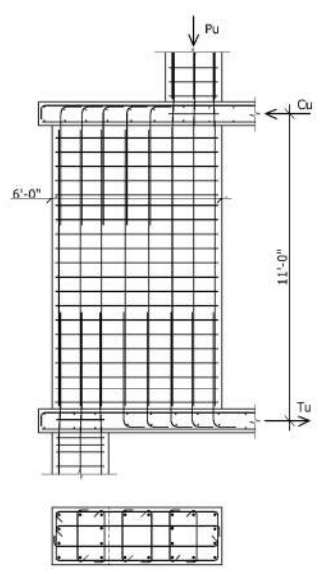

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Compute minimum deep beam shear reinforcing for which Appendix “A” strut strength equation may be used

Example: 34” wide walking column

Column reinf: #11@10” VEF & 4#5 ties @ 6”

STRUT + TIE REQ'D SHEAR REINF

VERT REINF

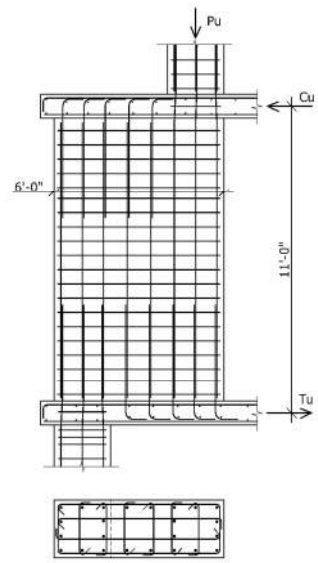
SPACING BETWEEN ALL VERT BARS

$A_v = .0025\,(34)(10") = .85\ in^2/10" < 2\#11\ @10"$ OK

HOR REINF

TIE SPACING

$A_v = .0015\,(34)(6") = .31\ in^2/\cancel{10"}\ 6" < 4\#5\ @6"$ OK



If we have the minimum horizontal and vertical reinforcing, we can develop the Appendix “A” strut strength. (Next slide)

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37

Compute column strut force and strut strength per ACI 318-08, Section A.3

Strut forces can be developed if the walking column has horizontal and vertical shear reinforcing per ACI 318 – 08 Sections 11.7.4 and 11.7.5 (but we still compute shear strength).

A.3 — Strength of struts

A.3.1 — The nominal compressive strength of a strut without longitudinal reinforcement, F_{ns}, shall be taken as the smaller value of

$$F_{ns} = f_{ce}A_{cs} \qquad \text{(A-2)}$$

at the two ends of the strut, where A_{cs} is the cross-sectional area at one end of the strut, and f_{ce} is the smaller of (a) and (b):

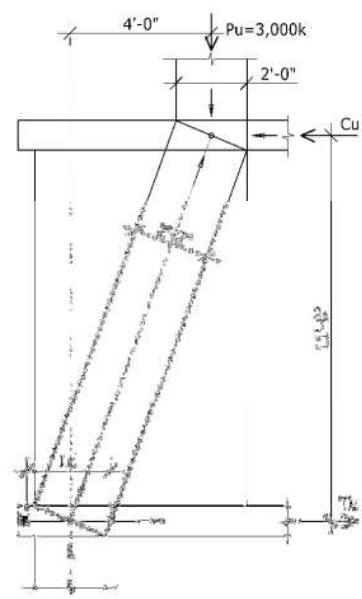
(a) the effective compressive strength of the concrete in the strut given in A.3.2;

(b) the effective compressive strength of the concrete in the nodal zone given in A.5.2.

A.3.2 — The effective compressive strength of the concrete, f_{ce}, in a strut shall be taken as

$$f_{ce} = 0.85\beta_s f'_c \qquad \text{(A-3)}$$

A.3.2.1 — For a strut of uniform cross-sectional area over its length ..... $\beta_s = 1.0$



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Compute node strength (in slab) per ACI 318-08, Section A.5. Node strength usually governs over strut strength due to lower strength concrete in slab.

A.5 — Strength of nodal zones

A.5.1 — The nominal compression strength of a nodal zone, F_{nn}, shall be

$$F_{nn} = f_{ce}A_{nz} \qquad \text{(A-7)}$$

where f_{ce} is the effective compressive strength of the concrete in the nodal zone as given in A.5.2, and A_{nz} is the smaller of (a) and (b):

(a) The area of the face of the nodal zone on which F_u acts, taken perpendicular to the line of action of F_u;

(b) The area of a section through the nodal zone, taken perpendicular to the line of action of the resultant force on the section.

A.5.2 — Unless confining reinforcement is provided within the nodal zone and its effect is supported by tests and analysis, the calculated effective compressive stress, f_{ce}, on a face of a nodal zone due to the strut-and-tie forces shall not exceed the value given by

$$f_{ce} = 0.85\beta_n f'_c \qquad \text{(A-8)}$$

where the value of β_n is given in A.5.2.1 through A.5.2.3.

A.5.2.1 — In nodal zones bounded by struts or bearing areas, or both..... $\beta_n = 1.0$;

A.5.2.2 — In nodal zones anchoring one tie..... $\beta_n = 0.80$;

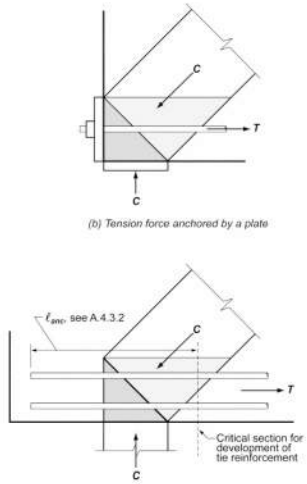
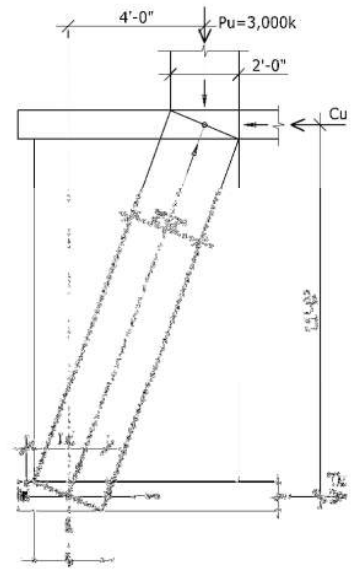


Fig. RA.1.4—Hydrostatic nodes.



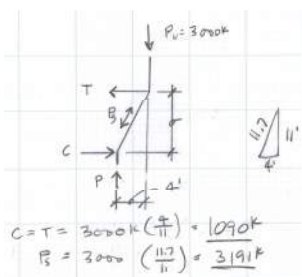
39

39

Compute strut force in walking column, and compute column strut & node strength per ACI 318-08, Section A.3

10 ksi columns
8 ksi slabs

Strut force



Strut strength

STRUT STRENGTH (ACI 318-08, A.3)

$P_u = 3191^k$

$\phi P_N = \phi\ .85\ \beta_s f'_c A_{cs} \times \left(1 - \left(\frac{k\ell}{32h}\right)^2\right)$ .99

$= .75(.85)(1)(10ksi)(26 \times 34) \times \left(1 - \left(\frac{132}{32(34)}\right)^2\right)$

$= 5636^k > 3191^k$ OK

Node strength

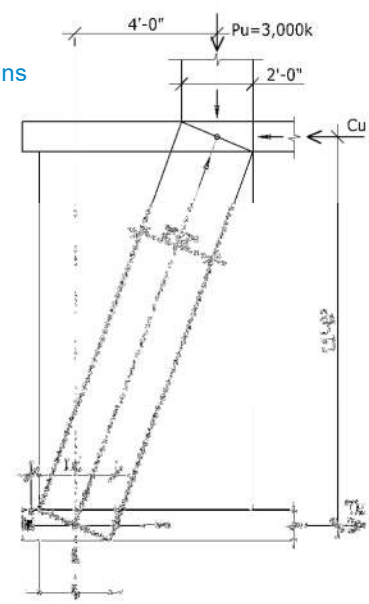
NODE STRENGTH (ACI 318-08, A.5)

$P_u = 3191^k$

$\phi P_N = \phi\ .85\ \beta_n f'_c A_{NZ}$

$= .75(.85)(.8)(8ksi)(26 \times 34)$

$= 3607^k > 3191^k$ OK



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Redunant shear strength calculation

Compute shear strength of walking column per per ACI 318-95, Section 11.8.

Additional info / suggestions:

- For walking columns, the column ties provide more shear strength (per square inch of steel) than the vertical bars.
- Use #4 ties (min.), #5 ties (max.).
- Space ties at 6”.
- Use additional legs of ties as required.
- Provide plan views off all walking columns showing all reinforcing steel.
- Limit vertical bar spacing in columns to 10”.

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41

41

ACI 318-95

11.8.6 — Unless a more detailed calculation is made in accordance with 11.8.7,

$$V_c = 2\sqrt{f_c'}\,b_w d \qquad (11\text{-}28)$$

11.8.7 — Shear strength V_c shall be permitted to be computed by

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(1.9\sqrt{f_c'} + 2500\rho_w\frac{V_u d}{M_u}\right)b_w d \qquad (11\text{-}29)$$

except that the term

$$\left(3.5 - 2.5\frac{M_u}{V_u d}\right)$$

shall not exceed 2.5, and V_c shall not be taken greater than $6\sqrt{f_c'}\,b_w d$. M_u is factored moment occurring simultaneously with V_u at the critical section defined in 11.8.5.

11.8.8 — Where factored shear force V_u exceeds shear strength ϕV_c, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

R11.8.7 — As the span-depth ratio of a member without web reinforcement decreases, its shear strength increases above the shear causing diagonal tension cracking. In Eq. (11-29) it is assumed that diagonal cracking occurs at the same shear strength as for ordinary beams, but that the shear strength carried by the concrete will be greater than the shear strength causing diagonal cracking.

Designers should note that shear in excess of the shear causing diagonal cracking may result in unsightly cracking unless shear reinforcement is provided.

R11.8.8 — The inclination of diagonal cracking may be greater than 45 deg, therefore, both horizontal and vertical shear reinforcement is required in deep flexural members.[11.36] The relative amounts of horizontal and vertical

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42

Design deep beam shear reinforcing per ACI 318-95, Section 11.8

For deep beams,

$$V_s = \left[\frac{A_v}{s}\left(\frac{1+\frac{\ell_n}{d}}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{11-\frac{\ell_n}{d}}{12}\right)\right] f_y d \qquad (11\text{-}30)$$

where A_v is area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s, and A_{vh} is area of shear reinforcement parallel to flexural reinforcement within a distance s_2.

11.8.9 — Area of shear reinforcement A_v shall not be less than $0.0015 b_w s$, and s shall not exceed $d/5$, nor 18 in.

11.8.10 — The area of horizontal shear reinforcement A_{vh} shall not be less than $0.0025 b_w s_2$ and s_2 shall not exceed $d/3$ nor 18 in.

11.8.11 — Shear reinforcement required at the critical section defined in 11.8.5 shall be used throughout the span.

When $A_v/s = A_{vh}/s_2$ then equation 11-30 becomes,

11.4.7.2 — Where shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_v f_{yt} d}{s} \qquad (11\text{-}15)$$

where A_v is the area of shear reinforcement within spacing s.

43

43

Calculate deep beam shear reinforcing per ACI 318-95, Section 11.8

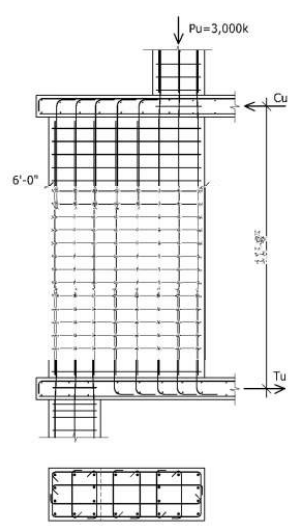
$$V_s = \left[\frac{A_v}{s}\left(\frac{1+\frac{\ell_n}{d}}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{11-\frac{\ell_n}{d}}{12}\right)\right] f_y d \qquad (11\text{-}30)$$

where A_v is area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s, and A_{vh} is area of shear reinforcement parallel to flexural reinforcement within a distance s_2.

11.8.9 — Area of shear reinforcement A_v shall not be less than $0.0015 b_w s$, and s shall not exceed $d/5$, nor 18 in.

11.8.10 — The area of horizontal shear reinforcement A_{vh} shall not be less than $0.0025 b_w s_2$ and s_2 shall not exceed $d/3$ nor 18 in.

11.8.11 — Shear reinforcement required at the critical section defined in 11.8.5 shall be used throughout the span.



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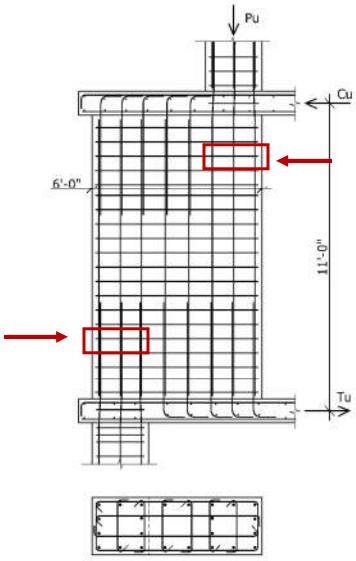
Tip

- Strive to continue vertical bars in columns above and below walking column into the walking column.
- Make vertical bars in walking column same size as vertical bars in columns above and below.



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Calculate deep beam shear reinforcing per ACI 318-95, Section 11.8

WALKING COLUMN

$P_u = 3000^k$

SHEAR STRENGTH

$\phi V_c = .75(2)\sqrt{10000}(34)(120) = 612^k$

SHEAR REINF

HOR $4\#5 @ 6" = 4(.31)(12/6) = 2.48 \text{ in}^2/\text{ft}$

VERT $2\#11 @ 16" = 2(1.56)(12/16) = 3.74 \text{ in}^2/\text{ft}$

$\phi V_s = .75(2.48 \text{ in}^2 \times 60 \text{ ksi})(10'/1') = 1116^k$

$\phi V_N = 612 + 1116^k = 1728^k < 3000^k$

NO GOOD

WALK OVER 2 FLOORS

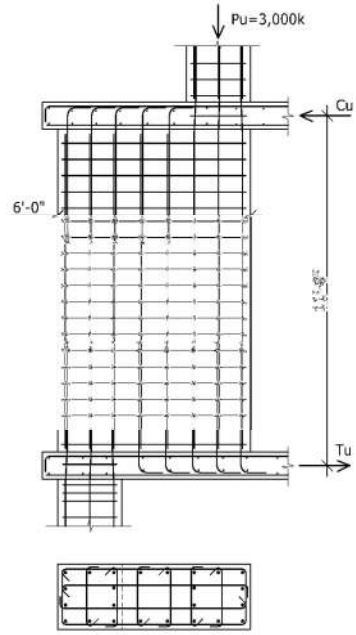
Walk column over two floors (or make column wider)

$\phi V_c = .75(2)\sqrt{10000}(34)(240") = 1224^k$

SHEAR REINF

$\phi V_s = .75(2.48 \text{ in}^2/\text{ft})(60 \text{ ksi})(20'/1') = 2232^k$

$\phi V_N = 3456^k > 3000^k$ OK



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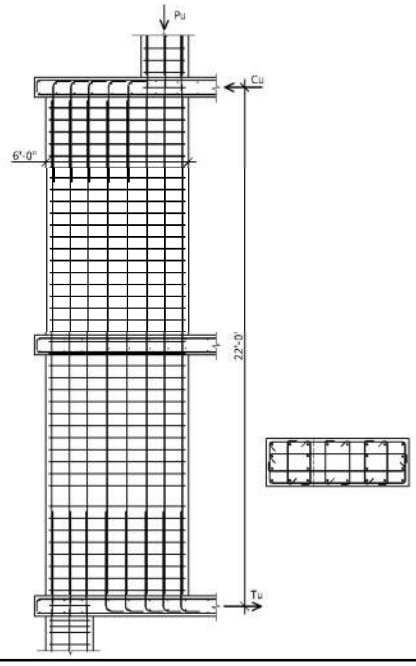
Walk over two floors to increase shear strength



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Design "internal columns"

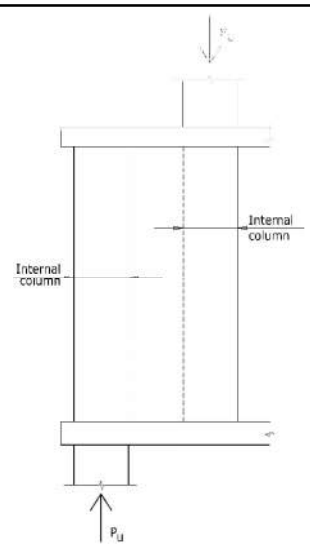
10.3.6 — Design axial strength ϕP_n of compression members shall not be taken greater than $\phi P_{n,max}$, computed by Eq. (10-1) or (10-2).

10.3.6.1 — For nonprestressed members with spiral reinforcement conforming to 7.10.4 or composite members conforming to 10.13:

$$\phi P_{n,max} = 0.85\phi[0.85f_c'(A_g - A_{st}) + f_y A_{st}] \quad (10\text{-}1)$$

10.3.6.2 — For nonprestressed members with tie reinforcement conforming to 7.10.5:

$$\phi P_{n,max} = 0.80\phi[0.85f_c'(A_g - A_{st}) + f_y A_{st}] \quad (10\text{-}2)$$



- Design each internal column to support full column load.
- Design each internal column to support full column load. (Use appropriate strength equations where required to account for slenderness. Equations 10-1 and 10-2 not always appropriate.)
- Strive to align vertical bars in internal columns with vertical bars in columns above and below.

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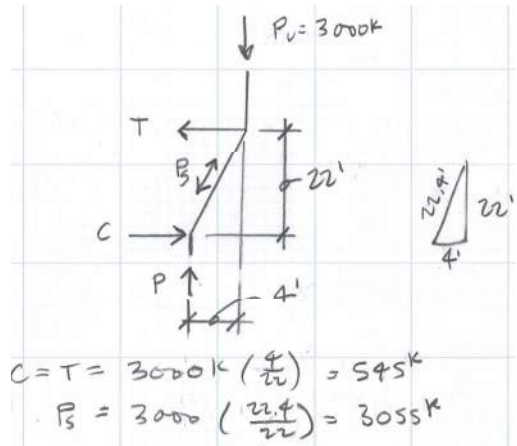
Calculate compression and tension strut forces in slab diaphragm

- Where slab occurs on both sides of column, horizontal force might be resisted by a combination of a tension strut and compression strut.
- Load will follow the stiffest load path (often the shortest distance to the shear walls receiving the diaphragm force).
- Use engineering judgement to determine load path.
- Show strut forces on plan.
- Identify compression and tension struts on plan.



Design slab / column interface for horizontal shear transfer

SHEAR FRICTION SHEAR STRENGTH AT SLAB/COLUMN INTERFACE

$V_u = 545^k$

UPPER LIMIT ON SHEAR FRICTION (SECT 11.6.5)

$\phi V_{n_{MAX}} = .75(.2) f'_c A_c = .75(.2)(6ksi)(34 \times 72) = 2203^k$

OR

$\phi V_{n_{MAX}} = 0.75(.8ksi)(A_c) = .75(.8ksi)(34 \times 72) = 1469^k$ GOVERNS

FOR SMOOTH JOINT

$A_{vf_{REQD}} = \frac{545^k}{.6(60ksi)} = 15.1 \text{ in}^2$

$A_{s\,COL} = 10\#11 = 15.6 \text{ in}^2 > 15.1 \text{ in}^2$ OK

FOR ROUGH JOINT

$A_{vf_{REQD}} = \frac{545^k}{1.0(60)} = 9.08 \text{ in}^2$

SHEAR STRENGTH PROVIDED BY COLUMN COMPRESSION

$\phi V_{n_{COMPRESSION}} = 0.6(3000^k) = 1800^k > 545^k$

(IN THEORY NO SHEAR FRICTION STEEL IS REQUIRED BUT DON'T RELY ON THE COMPRESSION LOAD TO RESIST THE SHEAR FORCES.)

Show rough joint, but design for smooth joint, and ignore beneficial effect of column compression load. (Shear friction reinforcing can be “used twice”, because for column vertical loads it is in compression & for shear friction it is in tension.)

Tension tie design

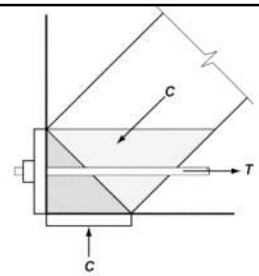
A.4 — Strength of ties

A.4.1 — The nominal strength of a tie, F_{nt}, shall be taken as

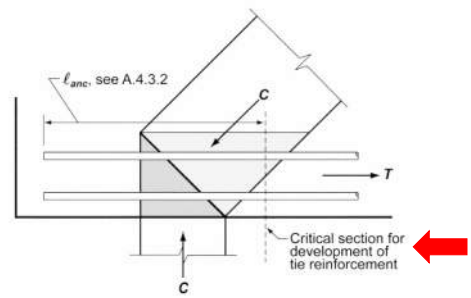
$$F_{nt} = A_{ts}f_y + A_{tp}(f_{se} + \Delta f_p) \qquad \text{(A-6)}$$

where $(f_{se} + \Delta f_p)$ shall not exceed f_{py}, and A_{tp} is zero for nonprestressed members.

9.3.2.6 — Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such models.....0.75



(b) Tension force anchored by a plate



(c) Tension force anchored by bond

Fig. RA.1.4—Hydrostatic nodes.

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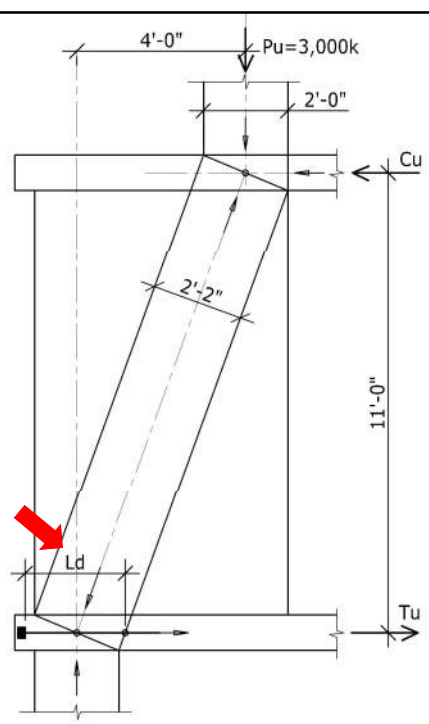

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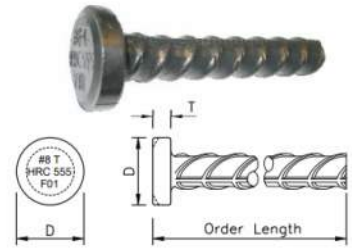
52

52



DIMENSIONS OF HRC 555 SERIES T-HEADS

HRC 555	Bar Size	#4	#5	#6	#7	#8	#9	#10	#11	#14
A706 rebar properties	Diameter [in]	0.500	0.625	0.750	0.875	1.000	1.128	1.270	1.410	1.693
	Area [sq.in.]	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56	2.25
	Yield [lbs]	12,000	18,000	26,400	36,000	47,400	60,000	76,200	93,600	135,000
	Tensile [lbs]	16,000	24,000	35,200	48,000	63,200	80,000	101,600	124,800	180,000
	Elongation [%]	14	14	14	12	12	12	12	12	10
Head	T_{min} [in]	0.25	0.31	0.38	0.44	0.50	0.56	0.64	0.70	1.02
	D[in]	1.14	1.42	1.69	1.97	2.25	2.56	2.87	3.19	3.82
	A_{brg} [sq.in]	0.82	1.27	1.80	2.45	3.18	4.14	5.20	6.43	9.20

ASTM A 706/A 706M Nominal Dimensions and Minimum Tensile Properties.
* Head thickness should be no larger than bar diameter.

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53

ACI 318-08
(ACI 318-14 is same)

12.6 — Development of headed and mechanically anchored deformed bars in tension

12.6.1 — Development length for headed deformed bars in tension, ℓ_{dt}, shall be determined from 12.6.2. Use of heads to develop deformed bars in tension shall be limited to conditions satisfying (a) through (f):

(a) Bar f_y shall not exceed 60,000 psi;

(b) Bar size shall not exceed No. 11;

(c) Concrete shall be normalweight;

(d) Net bearing area of head A_{brg} shall not be less than $4A_b$;

(e) Clear cover for bar shall not be less than $2d_b$; and

(f) Clear spacing between bars shall not be less than $4d_b$.

12.6.2 — For headed deformed bars satisfying 3.5.9, development length in tension ℓ_{dt} shall be $(0.016\psi_e f_y/\sqrt{f'_c})d_b$, where the value of f'_c used to calculate ℓ_{dt} shall not exceed 6000 psi, and factor ψ_e shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases. Where reinforcement provided is in excess of that required by analysis, except where development of f_y is specifically required, a factor of $(A_s \text{ required})/(A_s \text{ provided})$ may be applied to the expression for ℓ_{dt}. Length ℓ_{dt} shall not be less than the larger of $8d_b$ and 6 in.

R12.6 — Development of headed and mechanically anchored deformed bars in tension

The development of headed deformed bars and the development and anchorage of reinforcement through the use of mechanical devices within concrete are addressed in 12.6. As used in 12.6, *development* describes cases in which the force in the bar is transferred to the concrete through a combination of a bearing force at the head and bond forces along the bar, such cases are covered in 12.6.1 and 12.6.2. In contrast, *anchorage* describes cases in which the force in the bar is transferred through bearing to the concrete at the head alone. General provisions for anchorage are given in Appendix D. The limitation on obstructions and interruptions of the deformations is included in 3.5.9 because there is a wide variety of methods to attach heads to bars, some of which involve obstructions or interruptions of the deformations that extend more than $2d_b$ from the bearing face of the head. These systems were not evaluated in the tests used to formulate the provisions in 12.6.2, which were limited to systems that meet the criteria in 3.5.9.

The provisions for headed deformed bars were written with due consideration of the provisions for anchorage in Appendix D and the bearing strength provisions of 10.14.[12.15,12.16] Appendix D contains provisions for headed anchors related to the individual failure modes of concrete breakout, side-face blowout, and pullout, all of which were considered in the formulation of 12.6.2. The restrictions on normalweight concrete, maximum bar size of No. 11, and upper limit of 60,000 psi for f_y are based on the available data from tests.[12.15-12.17]

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54

ACI 318-19

25.4.4 *Development of headed deformed bars in tension*

25.4.4.1 Use of a head to develop a deformed bar in tension shall be permitted if conditions (a) through (f) are satisfied:

(a) Bar shall conform to 20.2.1.6
(b) Bar size shall not exceed No. 11
(c) Net bearing area of head A_{brg} shall be at least $4A_b$
(d) Concrete shall be normalweight
(e) Clear cover for bar shall be at least $2d_b$
(f) Center-to-center spacing between bars shall be at least $3d_b$

25.4.4.2 Development length ℓ_{dt} for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $\left(\dfrac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f_c'}}\right) d_b^{1.5}$ with ψ_e, ψ_p, ψ_o, and ψ_c, given in 25.4.4.3
(b) $8d_b$
(c) 6 in.

Table 25.4.4.3—Modification factors for development of headed bars in tension

Modification factor	Condition	Value of factor
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Parallel tie reinforcement ψ_p	For No. 11 and smaller bars with $A_{tr} \geq 0.3A_{bs}$ or s[1] $\geq 6d_b$[2,3]	1.0
	Other	1.0
Location ψ_o	For headed bars: (1) Terminating inside column core with side cover to bar ≥ 2.5 in.; or (2) With side cover to bar ≥ $6d_b$	1.0
	Other	1.25
Concrete strength ψ_c	For $f_c' < 6000$ psi	$f_c'/15{,}000 + 0.6$
	For $f_c' \geq 6000$ psi	1.0

[1] s is minimum center-to-center spacing of headed bars.
[2] d_b is nominal diameter of headed bar.
[3] Refer to 25.4.4.5.

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ACI 318-19

ACI 318-19 permits use of higher strength bars with possible reductions in development length for headed bars, but *"there's no such thing as a free lunch".*

To qualify for shorter development lengths and, bars must be spaced further apart and increased side cover must be used.

Suggestion: For simplicity, continue using headed bar provisions per ACI 318-08, unless there's a good reason to do otherwise.

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56

Considerations for selecting tension tie steel with Terminators

Suggest using Fy=60 ksi for tension tie bars even when you're using Grade 80 reinforcing steel.

Using Fy=60 ksi reduces strain in the tension struts and reduces potential crack widths in tension zones of slab.

May have to use #10 bars (versus #11) to get proper bar spacings and clear cover to top and bottom of slab.

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ACI 318-08

#10 GR 60

$$\ell_d = \frac{.016(1)(60000)}{\sqrt{6000}}(1.27) = \underline{15.7''} \quad (5d_b = 6.4'')$$

11 GR 60

$$\ell_d = \frac{.016(1)(60000)}{\sqrt{6000}}(1.41) = \underline{17.5''} \quad (5d_b = 7.1'')$$

Note the required minimum bar spacing required to develop the headed bars in tension.

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58

ACI 318-19

#10 GR 60 $\quad 6d_b = 7.6'' \quad 3d_b = 3.8''$

$$l_d = \frac{60000(1)(1.6)(1.25)}{75\sqrt{6000}}(1.27) = 26.2^{IN}$$ (FOR 3.8" SPACING) (FOR $A_s = 3.8''$)

#11 GR 60 $\quad 6d_b = 8.5^{IN} \quad 3d_b = 4.2''$

$$l_d = \frac{60000(1)(1.6)(1.25)}{75(\sqrt{6000})}(1.41) = 29.1^{IN}$$ (FOR 4.2" SPACING) (FOR $A = 4.2''$)

DEVELOPMENT LENGTHS CAN BE CUT IN HALF IF BARS ARE SPACED @ $6d_b$ (1.6 FACTOR) & SIDE COVER > $6d_b$ (1.25 FACTOR)

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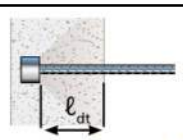
59

59



Tension Development Lengths for Headed Reinforcing Uncoated Bars (ACI)

(Per ACI 318-08)

Bar Size ASTM	f'c = 3,000 psi	f'c = 4,000 psi	f'c = 5,000 psi	f'c = 6,000 psi
#4	9	8	7	6
#5	11	10	9	8
#6	13	12	10	10
#7	16	14	12	11
#8	18	15	14	13
#9	20	17	16	14
#10	23	20	18	16
#11	25	22	19	18

1 inch = 24 milimeters

Notes:

1. Tabulated values are based on a minimum yield strength of 60,000 psi [420MPa]. Lengths are in inches.
2. Tension development lengths of headed bars are calculated per ACI 318-08, Section 12.6.
3. Tabulated values have been rounded up to nearest whole number.

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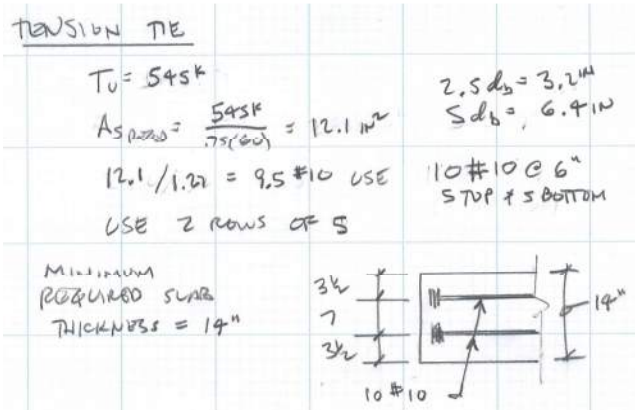
60

Design and detail slab tension strut (ACI 318-08, Section A.4)



Be conservative with large tension forces. Cracks may occur in slabs with tension struts if sufficient steel, properly designed and detailed, is not provided.

Suggestion: Add note on drawings to repair cracks in tension struts via epoxy injection.

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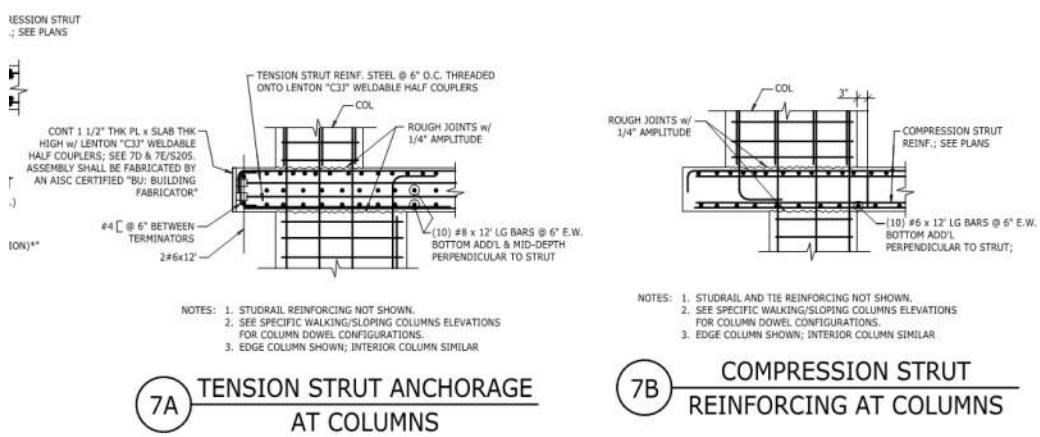
Design tension tie anchor plates



Suggestion: Use continuous tension tie anchor plates at walking column / tension tie nodes occurring at slab edges where tension forces are high (>300k)

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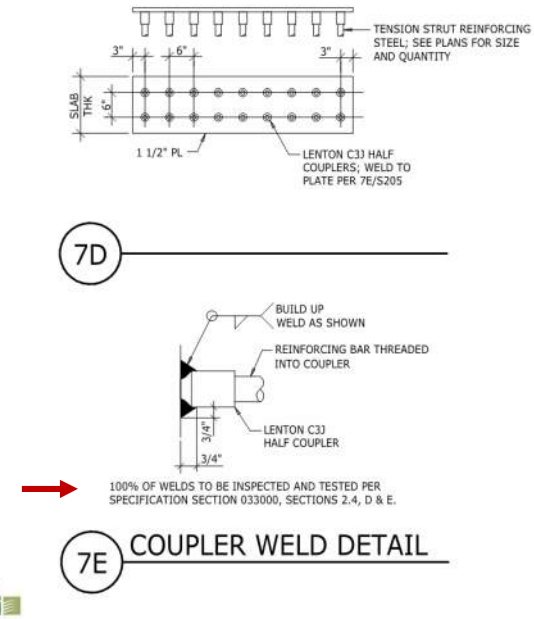
62

62

Design tension tie anchor plates



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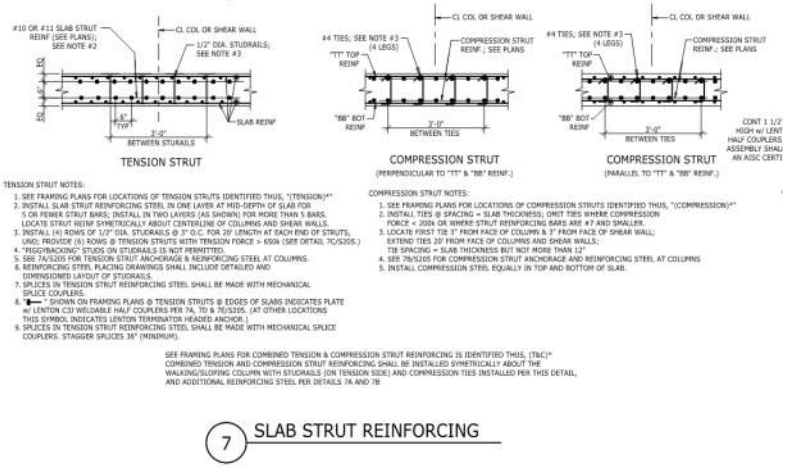
63

Design / detail tension strut reinforcing

Design and detail studrail shear reinforcing in tension zones (because ØVc = 0!).



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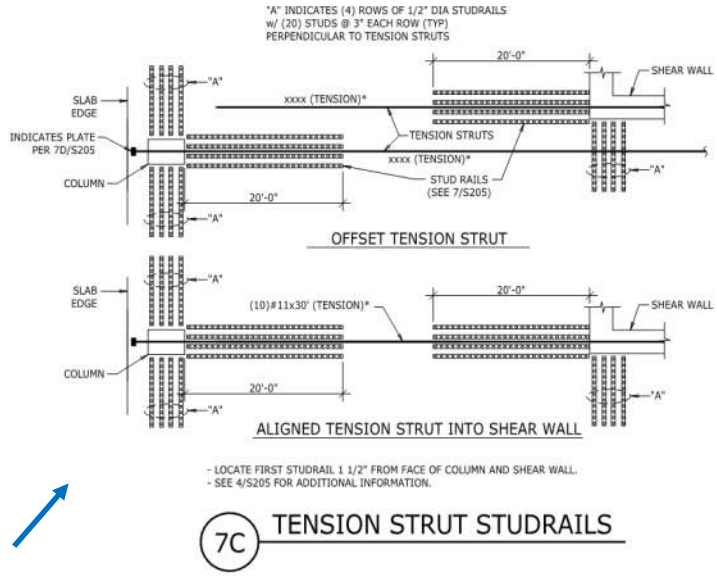
64

Design / detail tension strut reinforcing

ØVc = 0 where slab tension > 500 psi. Provide studrail shear reinforcing to provide required shear strength in tension zones near columns where tension strut originates and near shear walls where tension struts transfer diaphragm loads into the walls.

Can compute reduced ØVc per ACI 318-08 Section 11.2.2.3., but this is not rocket science – and the actual tension forces maybe higher than you think they are – due to shrinkage, restraint to shortening, thermal, etc. – so be conservative and use ØVc = 0.



Will vary from project to project

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65

Design and detail slab compression strut

- Compute slab strut compression strength (ACI 318-08, Section 14.5)
- Compute maximum slab deflection in compression strut (including creep) and double it to account for P-delta effects. Compute additional slab moment using magnified deflection to account P-delta effects. Design compression strut for combined bending and compression.
- Compute combined bending / compression strength

14.5 — Empirical design method

14.5.1 — Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of 14.5 if the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of 14.2, 14.3, and 14.5 are satisfied.

14.5.2 — Design axial strength ϕP_n of a wall satisfying limitations of 14.5.1 shall be computed by Eq. (14-1) unless designed in accordance with 14.4.

$$\phi P_n = 0.55\phi f'_c A_g\left[1-\left(\frac{k\ell_c}{32h}\right)^2\right] \qquad (14\text{-}1)$$

where ϕ shall correspond to compression-controlled sections in accordance with 9.3.2.2 and effective length factor k shall be:

(SLAB) COMPRESSION STRUT

$P = 545^K$ (ASSUME 25' COMPRESSION STRUT SPAN)

STRUT STRENGTH

$\phi P_N = .65(.55)(6^{KSI})(14 \times 12^2)\left[1-\left(\frac{25}{32(1.17)}\right)^2\right]$

$= 198\ K/FT$

REQUIRED WIDTH OF STRUT $= \frac{545}{198} = 2.8\ FT$

ASSUME 1" SLAB DEFLECTION DUE TO D+L

P-Δ MOMENT $= 545^K(.17') = 93\ FT\text{-}K$

ADD'L FLEXURAL STEEL DUE TO P-Δ MOMENT

$A_{P\Delta} = .25(93)/12.5 = 1.86\ IN^2$ (3 #6 ADD'L T&B)

ADDITIONAL T&B BARS TO RESIST P-Δ MOMENT

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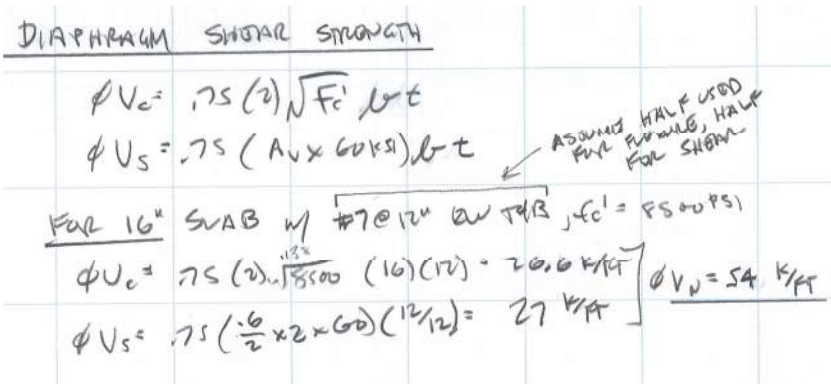
Diaphragm shear strength



Note: Cannot use steel twice. (Cannot use the same steel for both flexure and shear if both are in tension.)

Diaphragm shear strength

- Provide continuous each way top and bottom steel in slabs resisting large diaphragm forces.
- Recommended minimum slab thicknesses,
  - 8” slab for strut forces up to 100k
  - 10” slab for strut forces up to 200k
  - 12” slab for strut forces up to 400k
  - 14” slab for strut forces up to 800k
  - 16” slab for strut forces up to 1,200k
- Use ACI 318-95 deep beam shear strength equation to calculate diaphragm shear strength (per foot)

Diaphragm flexural strength (flexural chord steel)

DIAPHRAGM CHORD STEEL

$M_U = 990^k \times 24' = 23760$ FT-K

$A_{S_{REQD}} = \dfrac{23760 \text{ FT-K}}{86 \text{ FT} (60 \text{ KSI})(.9)} = 5.31 \text{ IN}^2$ $\quad 4\#11 = 6.24 \text{ IN}^2$

Perimeter chord steel to resist global moments in diaphragm.

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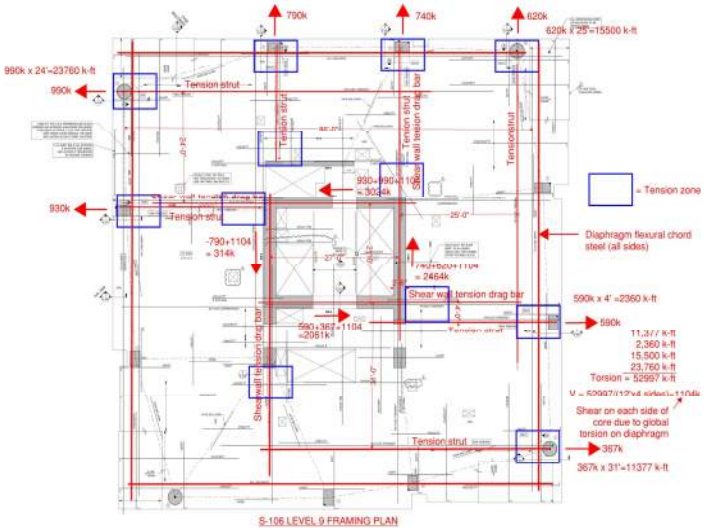
69

Analyze floor diaphragm, determine load path through diaphragm, and design diaphragm / detail reinforcing steel for shear, moment, global diaphragm torsion, tension (studrails in tension zones) and compression



Diaphragm loads get into the shear walls via, tension and compression struts into ends of walls, and by shear friction.

Extend tension strut steel sufficiently far into the walls to allow full load transfer from the strut steel into the wall. (Conservatively, extend strut steel through full length of wall.)

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Identify load paths and address all anomalies in load path (slab openings, steps in slabs, etc.)

70

70

Review, analyze, and design tension & compression load transfer into shear walls

- Loads can get into shear walls by tension, compression or shear.
- Analysis is similar to strut design at interface with walking columns.
- Detail tension steel into shear walls to transfer that force into the wall.
  - Check shear strength of wall and verify that tension strut steel extends sufficiently far into the wall to engage a sufficient length of wall in shear. (Embedding steel into shear wall for lap splice length may be insufficient.)
  - Suggest anchoring tension strut steel across full length of wall (not just 5' or 10' into wall). (example: What is shear strength of wall per foot? How much shear is being transferred into the wall by the tension strut?)
  - Identify rough joints at interface between walls and top and bottom of slabs.

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Draw elevations for all walking columns

Provide,

- Full height elevation of walking columns with portions of columns above and below
- Indicate orientation (i.e., VIEW LOOKING NORTH, etc)
- Column identification (Identify walking columns on plan and on column schedule)
- Column dimensions and reinforcing steel
- Column axial load at top of walk and horizontal forces into diaphragm
- Two sections (minimum) looking down through column showing all reinforcing steel. One section above walking column, one section near bottom of walking column. (Cut sections where column steel is lapping with steel from above / below.)
- Provide enough information so that it is easy to understand how the reinforcing steel is to be configured.
- Identify rough joints at interface between columns and slabs.

Do not rely on walking column typical details. Actual conditions usually negate the ability to use those details.

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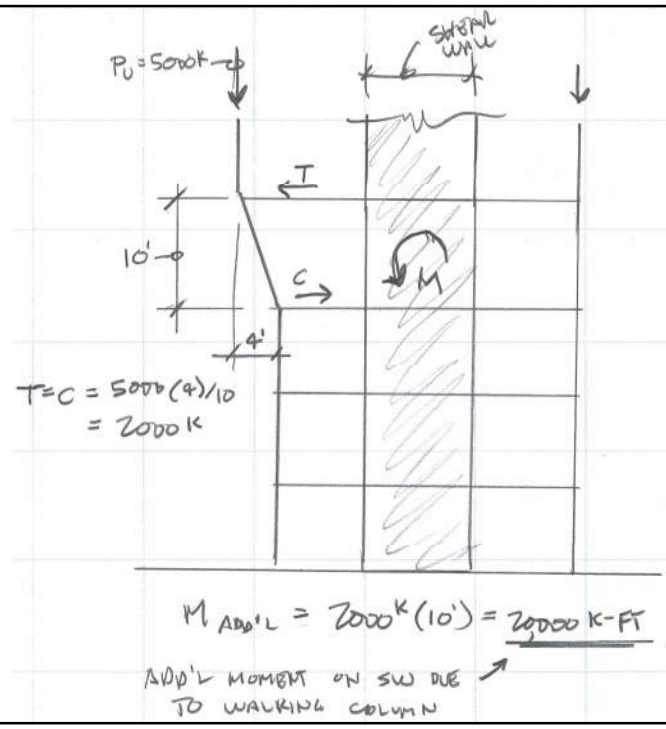
72

Calculate additional moments imposed on shear walls due to walking columns



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73

Things to watch out for,

- Reinforcing steel congestion (in nodes and strut connections to shear walls)
- Additional MEP sleeves and openings, or field modifications at known sleeves and openings
- Steps in slabs
- Large openings in compression and tension struts
- Failure to properly install terminators, anchor plates, splice couplers.
- Cracks in tension strut slabs
- Inadequate development of tension strut steel
- Insufficient detailing of column dowels at ends of walking/sloping columns resulting in incorrect placement of vertical reinforcing steel.

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74

Summary

- Use strut-and-tie methodology for design of walking columns
- Compute strut strength in walking column
- Compute and design for tension and compression diaphragm tie/strut forces in slab
- Draw elevations for walking columns and show on drawings (to better communicate with the rebar detailer and the engineer in your office reviewing the rebar placing drawings).
- Design / detail node reinforcing at slab / walking column joints
- ØVc = 0 in slabs near nodes imposing significant tension in the slab! Provide studrails.
- Tension forces in nodes occurring at slab edges require careful attention to detailing to develop the tension tie reinforcing steel. (See ACI 318 requirements for strut and tie design.) There are no alternative load paths at these nodes.
- Provide, design, and detail rational load paths for transfer of diaphragm strut forces to shear walls.

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75

75

Summary

- Provide, design, and detail rational load paths for transfer of diaphragm strut forces to shear walls.
- Show diaphragm strut forces on framing plans and identify diaphragm strut steel
- Design and detail slab reinforcing steel to resist diaphragm shear and to connect the diaphragm and slab struts to the shear walls.
- Provide top and bottom steel in diaphragms with substantial tie/strut forces (Reinforcing steel cannot be used twice. (example: If you need #5@12” EW T&B for gravity loads and #5@12” EW T&B for horizontal forces in the diaphragm, then you need #7@12” EW T&B in the slab.)
- Slab tension steel extending into parallel shear walls must extend sufficiently far into the walls to permit load transfer into the walls. (Extending #11 tension bars into a shear wall with #5@12” HEF will not work.)
- Pay close attention to walking columns, slab diaphragm design, nodes, load transfer to shear walls, punching shear in slabs near tension tie nodes (including tension load transfer into shear walls).
- Be conservative.

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76

76

Thank you!

Questions?

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77