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ACI 318-19: What's New for 2019

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ACI 318-19: What's New for 2019

- New Shear Strength Equations; including size-effect factor
- Higher Rebar Grades
- Updated Development Lengths
- New Effective Stiffness for Deflection Calculations
- Seismic Design Details – Shear Walls
- Some Updates to Strut & Tie Method

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Changes to the Concrete Design Standard

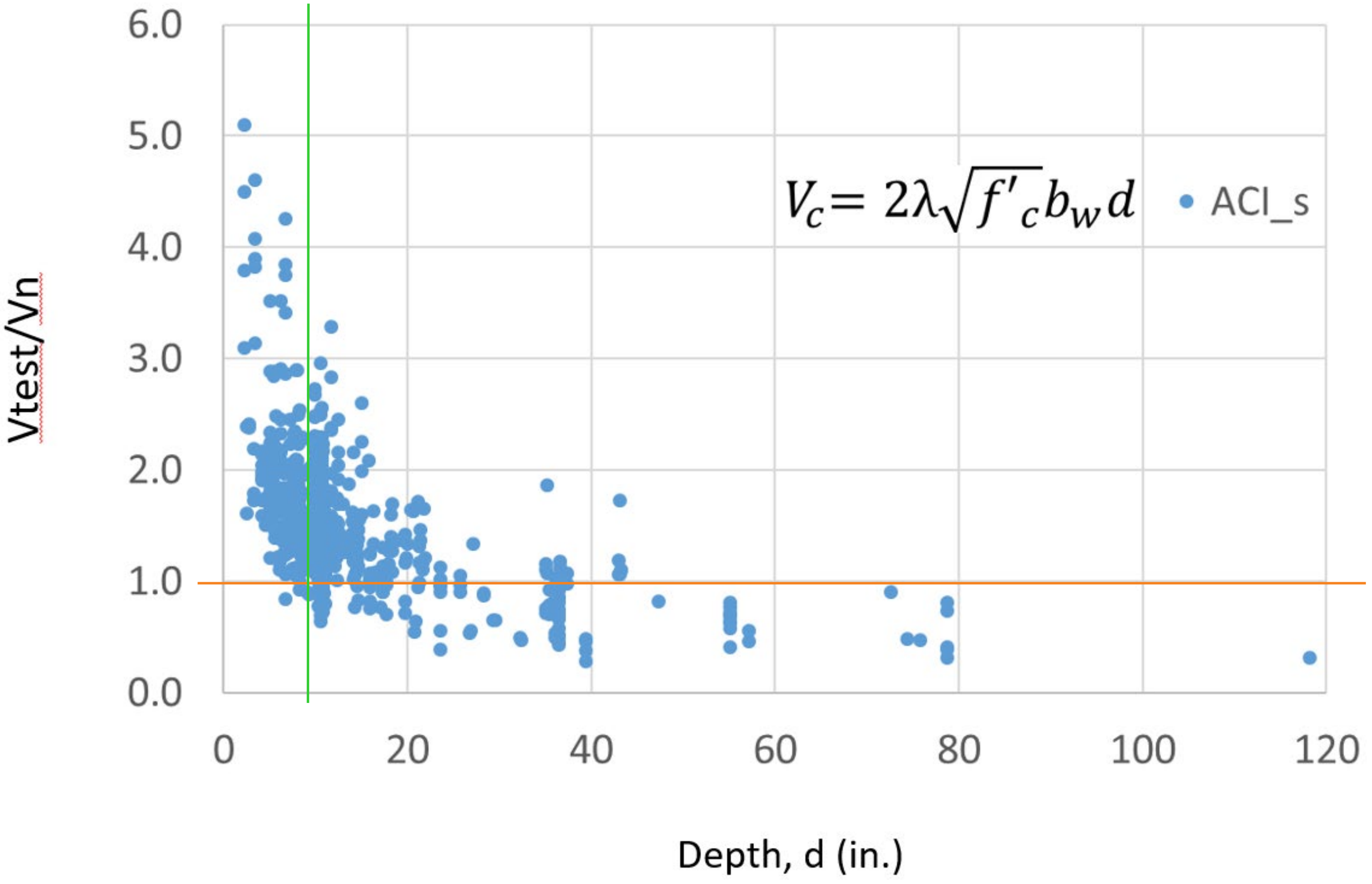
Shear Modifications

Shear equation changes for one-way and two-way shear

- Size Effect
- Low Flexural Reinforcement Ratio
- Axial load (prestress)

- Results gathered and vetted by ACI Comm. 445

Why one-way shear equations changed in 318-19



$$\boxed{A_v \leq A_{v,\min}}$$

Figure: Strength Ratio (V_{test}/V_n)

Why one-way shear equations changed in 318-19

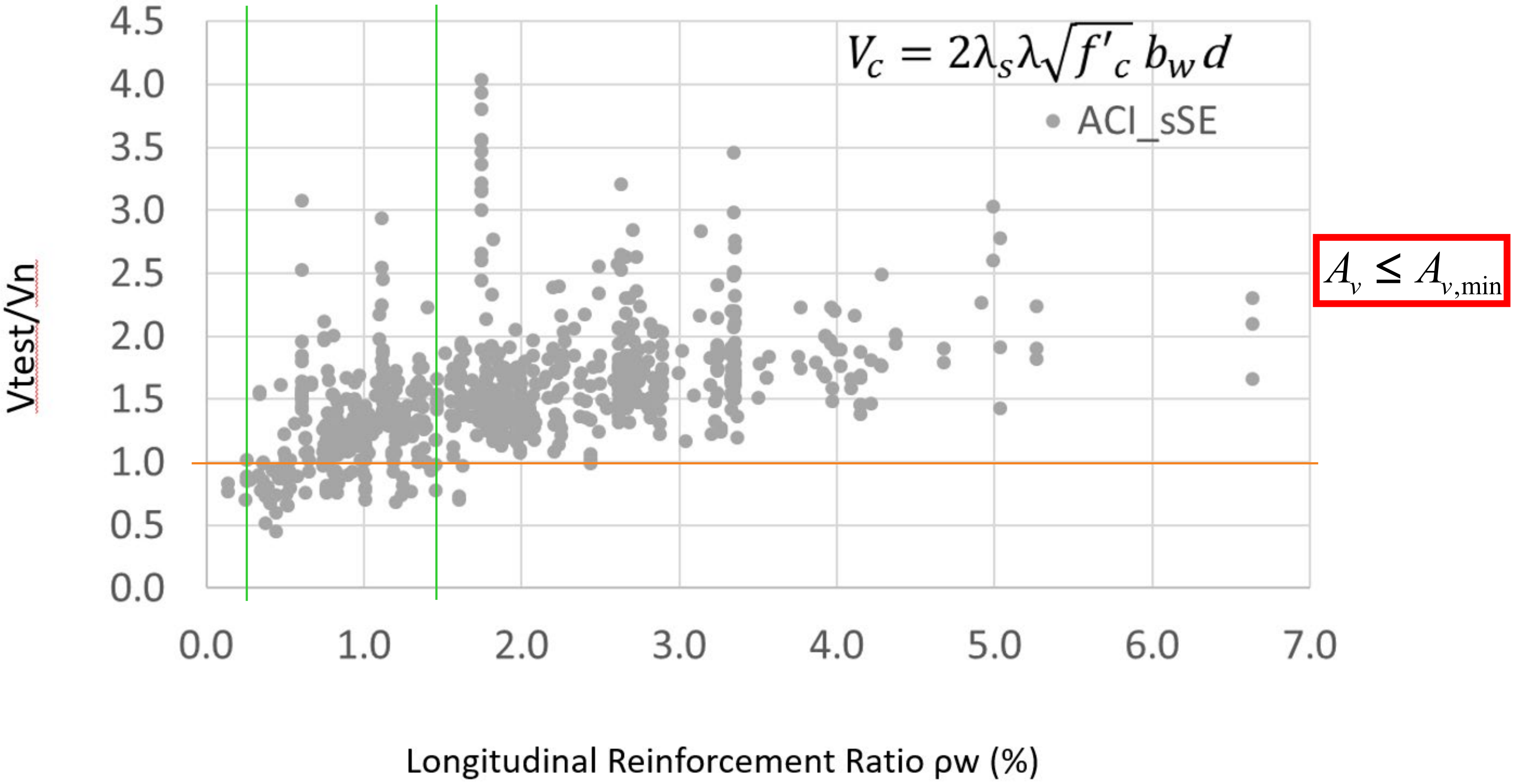


Figure: Strength Ratio (V_{test}/V_n)

Why one-way shear equations changed in 318-19

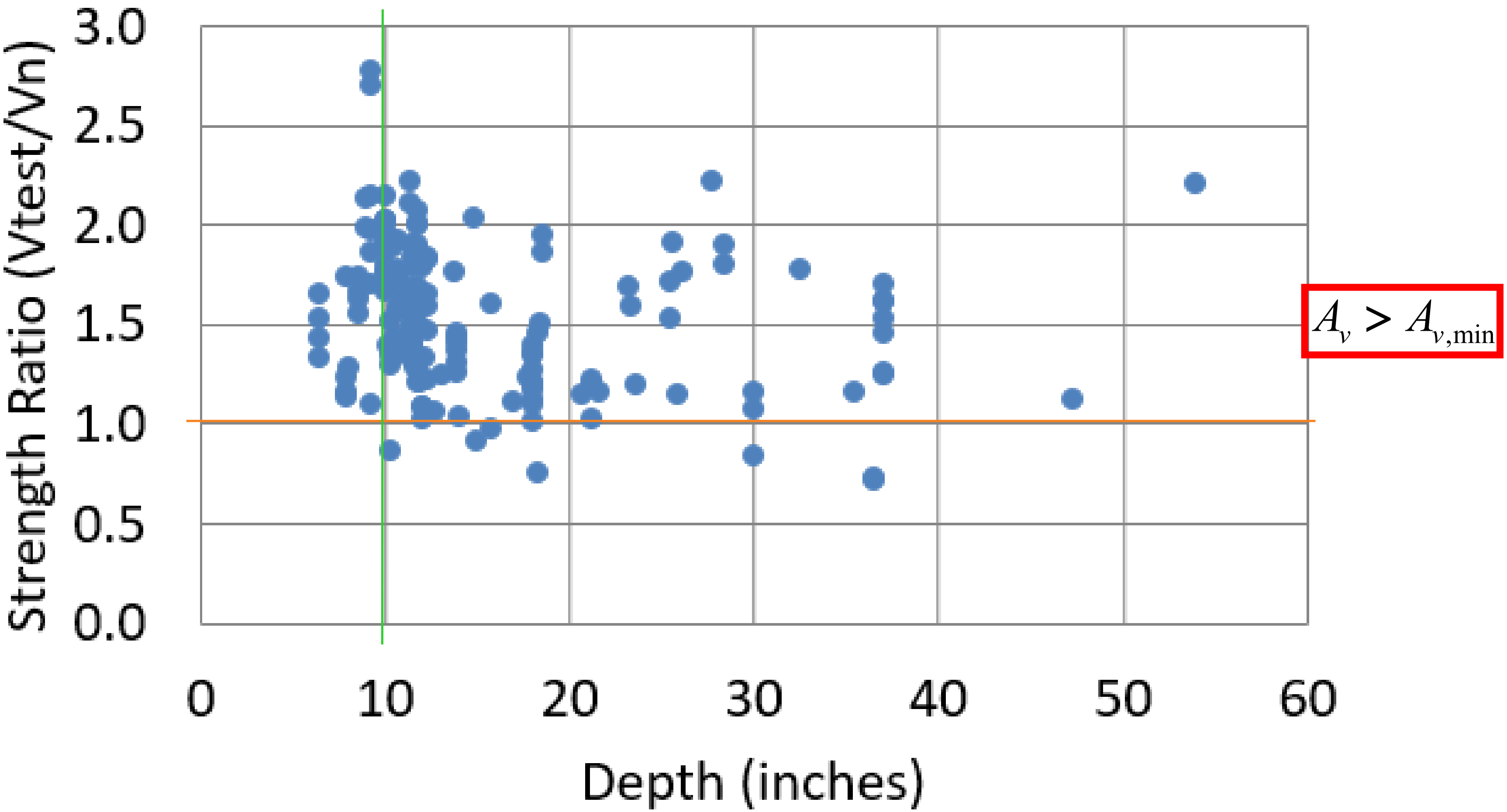


Figure: Strength Ratio (V_{test}/V_n)

One-way shear provision: Modified goals

- Include nonprestressed ~~and prestressed~~
- Include size effect and axial loading
- Include effect of (ρ_w)
- Continue to use $2\sqrt{f'_c}$
- Reduce multiple empirical equations
- Easy to use

ACI 318-19 New one-way shear equations

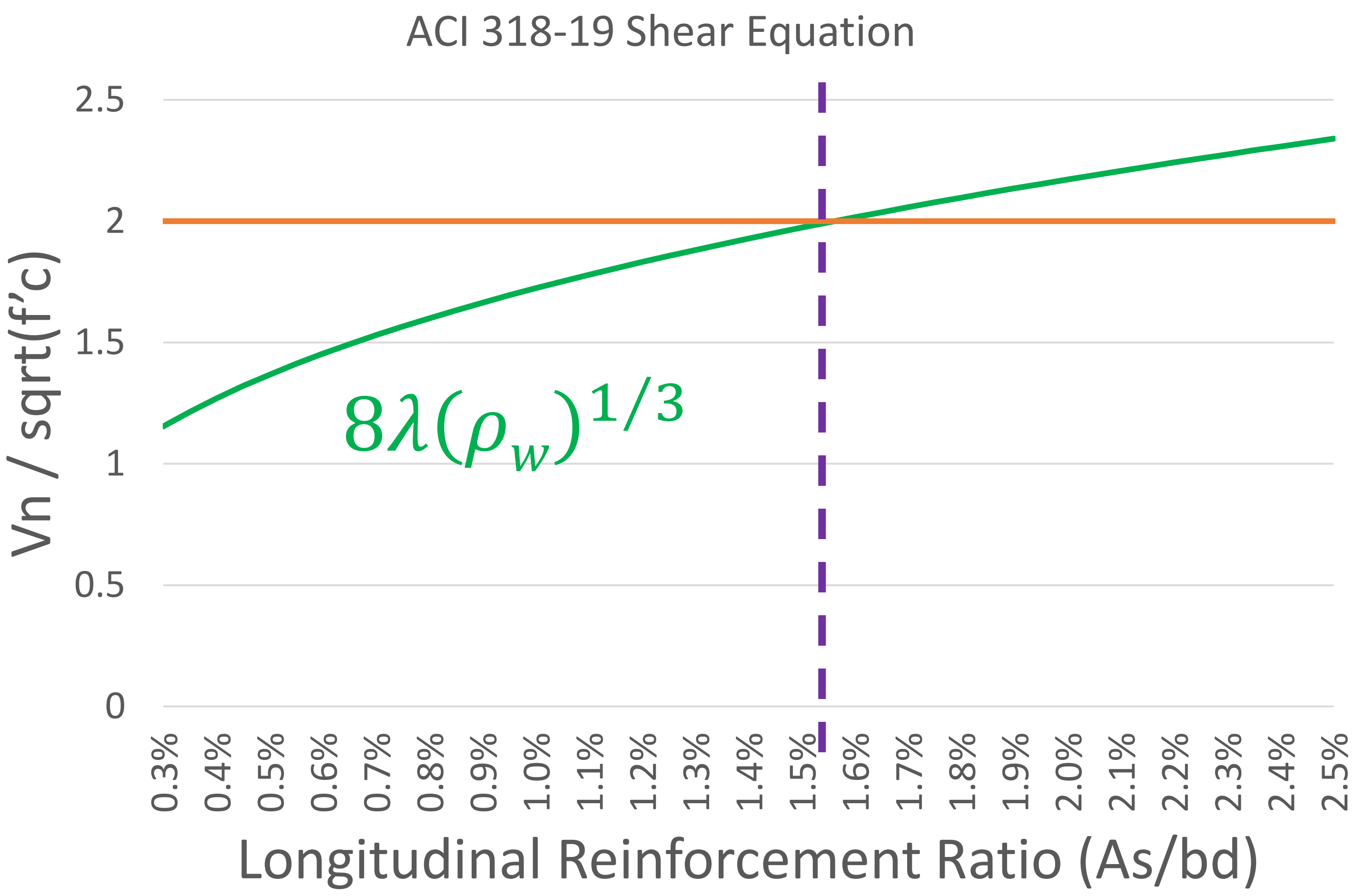
Table 22.5.5.1 - V_c for nonprestressed members

Criteria	**V_c**		
$A_v \geq A_{v,min}$	Either of:	$\left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$	(a)
		$\left[8\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$	(b)
$A_v < A_{v,min}$	$\left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$		(c)

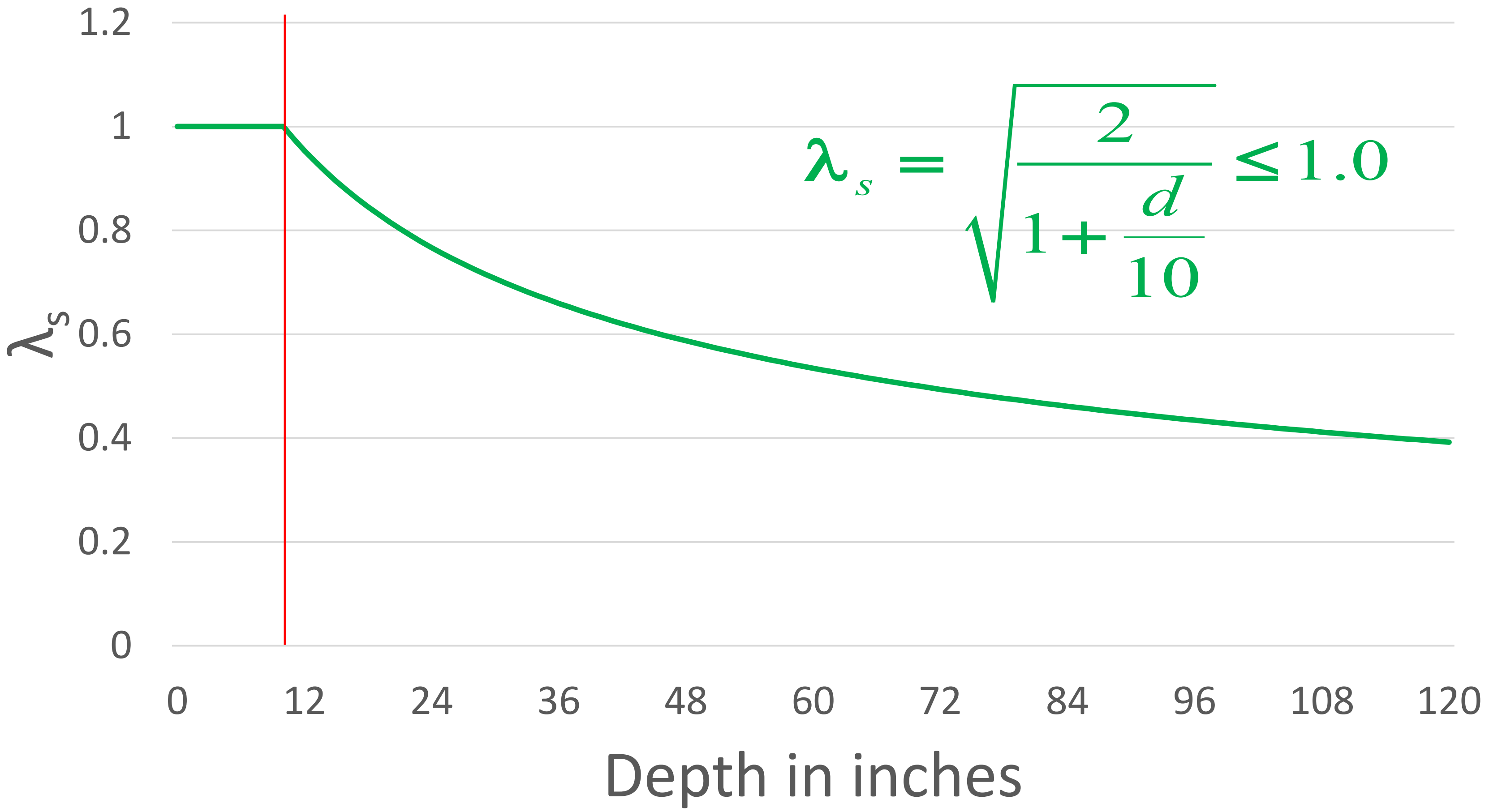
Notes:

1. Axial load, N_u, is positive for compression and negative for tension
2. V_c shall not be taken less than zero.

Effect of ρ_w



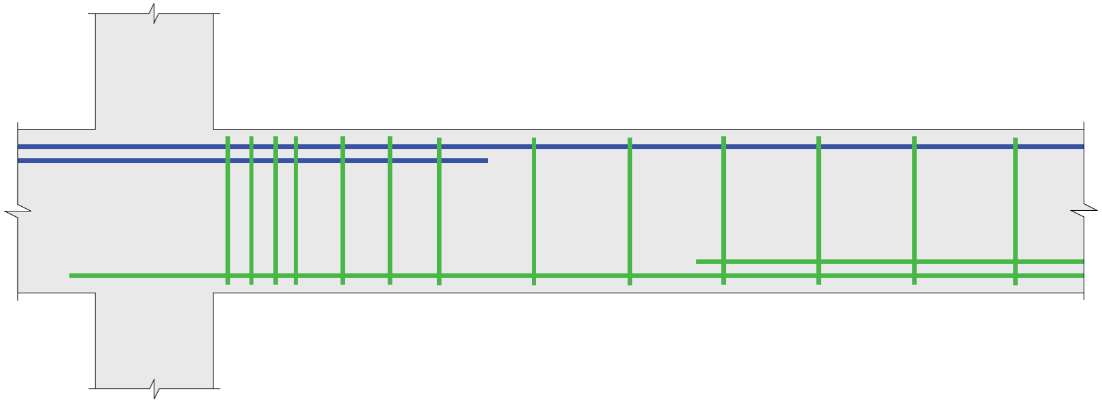
Size Effect: Value for λ_s?



$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \le 1.0$$

Beam discussion

- **Where $A_{v,min}$ installed and $N_u \approx 0$, $V_c \approx (2\sqrt{f'_c})b_w d$,**
  - ACI 318-14 ~ ACI 318-19
- **Provisions encourage use of $A_{v,min}$**



9.6.3.1 - Minimum shear reinforcement

- ACI 318-14
  - $A_{v,min}$ required if $V_u > 0.5\,\phi V_c$

- ACI 318-19
  - $A_{v,min}$ required if $V_u > \phi\lambda\sqrt{f'_c}\,b_w d$

Example: Foundation Shear Check

- ℓ = 12 ft
- h = 30 in.
- $d \sim 25.5$ in.
- f'_c = 4000 psi
- 13-No. 8 bars
- b = 12 ft
- A_v = 0 in.^{2}
- A_s = 10.27 in.^{2}
- Analysis V_u= 231 kip

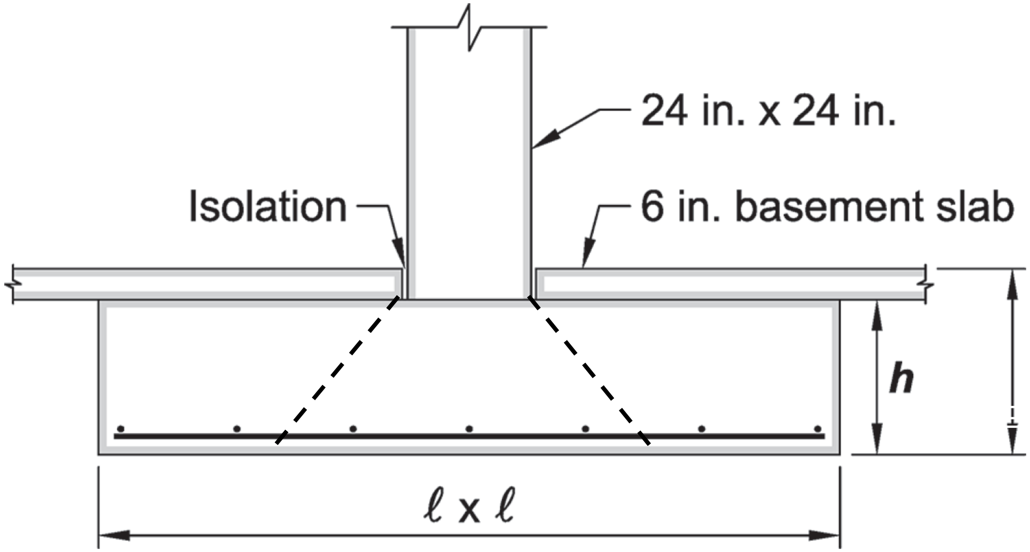


Fig. E1.1—Rectangular foundation plan.

Example: Foundation Shear Check

- **ACI 318-14**

$$\phi V_c = \phi 2 \lambda \sqrt{f_c^{'}} bd$$

$$\phi V_c = (0.75)(2)(1)\sqrt{4000\,psi}\,(144 in.)(25.5 in.)$$

$$\phi V_c = 348 \text{ kip} > 231 \text{ kip} \therefore \text{OK}$$

Example: Foundation Shear Check

- **ACI 318-19**
- $A_v \leq A_{v,min}$
- **Per ACI 318-19 (13.2.6.2), neglect size effect for:**
  - One-way shallow foundations
  - Two-way isolated footings
  - Two-way combined and mat foundations

$$\phi V_c = \phi 8 \lambda (\rho_w)^{1/3} \sqrt{f_c'} bd$$

Example: Foundation Shear Check

- **ACI 318-19**

$$\phi V_c = \phi 8\lambda(\rho_w)^{1/3}\sqrt{f_c'}bd$$

$$\rho_w = \frac{10.27 \text{ in.}^2}{(144 \text{ in.})(25.5 \text{ in.})} = 0.0028$$

$$\phi V_c = (0.75)(8)(1)\left(0.0028\right)^{1/3}\sqrt{4000\,psi}\,(144in.)(25.5in.)$$

$$\phi V_c = 196 \text{ kip} < 231 \text{ kip} \therefore \text{NG}$$

Example: Foundation Shear Check

- **ACI 318-19**
- **Add 6 in. thickness**

$$\phi V_c = \phi 8\lambda(\rho_w)^{1/3}\sqrt{f_c'}bd$$

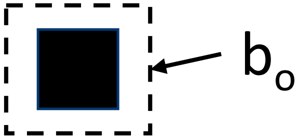
$$\rho_w = \frac{10.27\text{ in.}^2}{(144\text{ in.})(31.5\text{ in.})} = 0.0023$$

$$\phi V_c = (0.75)(8)(1)\left(0.0023\right)^{1/3}\sqrt{4000\text{ psi}}(144\text{ in.})(31.5\text{ in.})$$

$$\phi V_c = 226\text{ kip} > 191\text{ kip} \therefore \text{OK}$$

Why two-way shear provisions changed in 318-19

- First Equation developed in 1963 for slabs with $t < 5$ in. and $\rho > 1\%$
- Two issues similar to one-way shear
  - Size effect
  - Low ρ



$V_c = v_c(b_o d)$

v_c		
Least of (a), (b), and (c):	$4\lambda\sqrt{f_c'}$	(a)
	$\left(2+\frac{4}{\beta}\right)\lambda\sqrt{f_c'}$	(b)
	$\left(2+\frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'}$	(c)

Two-way shear: size effect

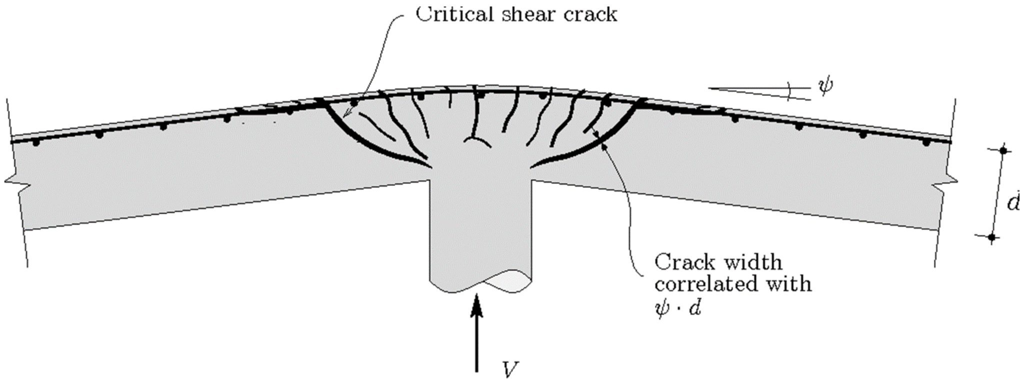
- Table 22.6.5.2—v_c for two-way members without shear reinforcement

$\boldsymbol{v_c}$		
Least of (a), (b), and (c):	$4\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(2+\frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \leq 1$$

Two-way shear: Effect of low ρ

- Only vert. load, cracking ~$2\sqrt{\boldsymbol{f'_c}}$; punching $4\sqrt{\boldsymbol{f'_c}}$
- Aggregate interlock contributes to shear strength
- Low ρ ➔ local bar yielding, crack width increase, allows sliding along shear crack
- Punching loads < $4\sqrt{\boldsymbol{f'_c}}$



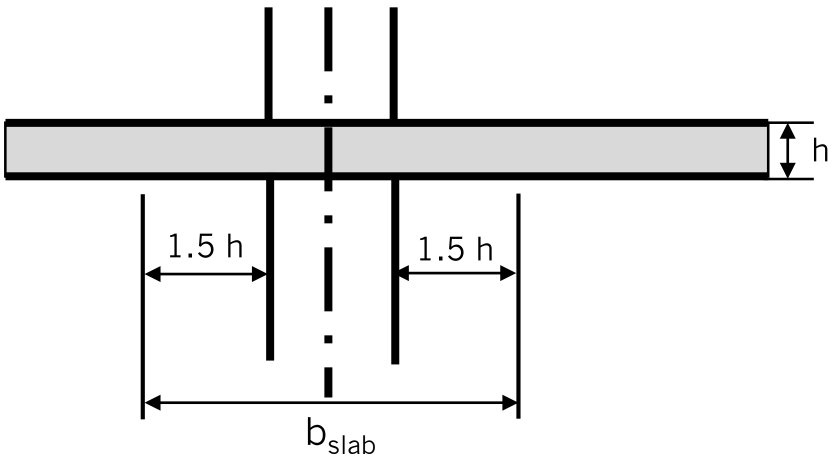
New two-way slab reinforcement limits

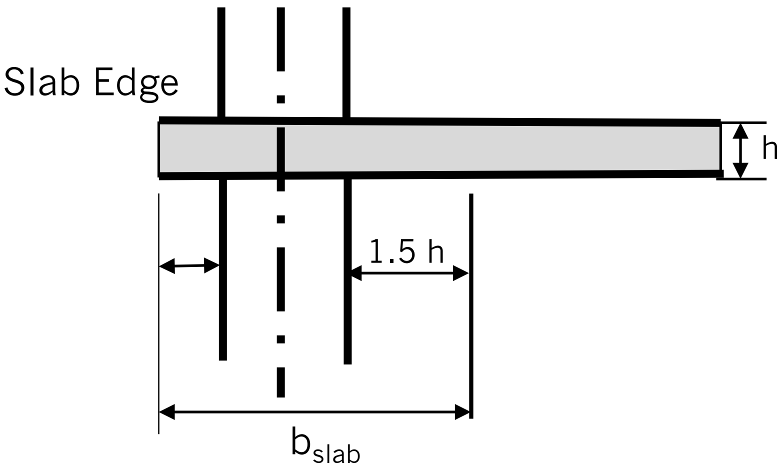
- *Need* $A_{s,min} \geq 0.0018A_g$
- If on the critical section $v_{uv} > \phi 2 \lambda_s \lambda \sqrt{f_c'}$
- Then

$$A_{s,\min} \geq \frac{5 v_{uv} b_{slab} b_o}{\phi \alpha_s f_y}$$

Table 8.4.2.2.3

Definition of b_{slab}





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Changes to the Concrete Design Standard

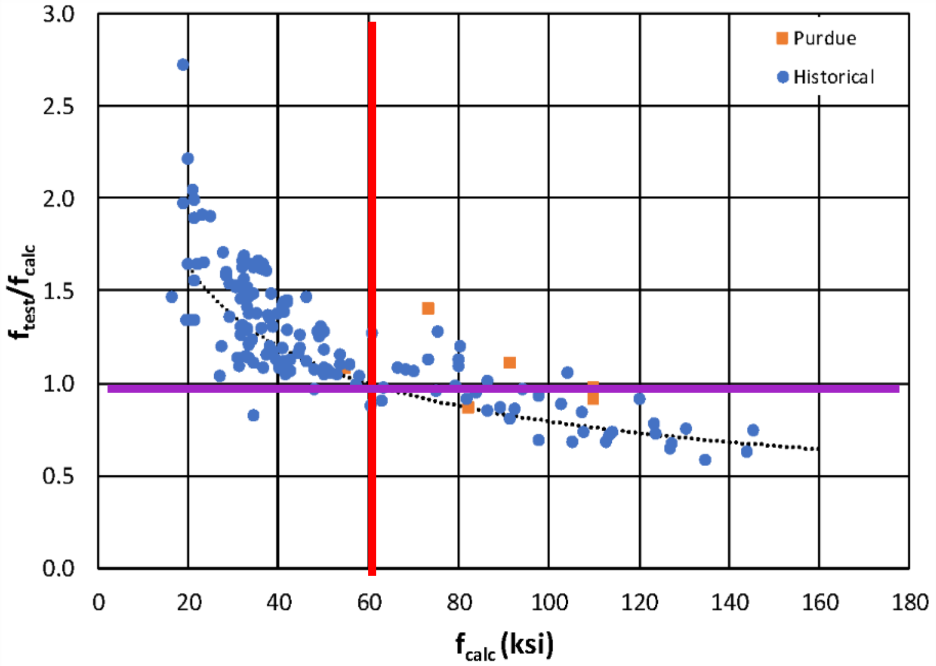
Development Length

Development Length

- Straight Deformed Bars and Deformed Wires in Tension
  - Simple modification to 318-14
  - Accounts for Grade 80 and 100

- Standard Hooks and Headed Deformed Bars
  - Substantial changes from 318-14

Straight Development Length of Deformed Bars in Tension



Unconfined

Figure 1: ACI 318-14 Equation 25.4.2.3a (Unconfined Test Results)

f_{test} = reinforcement stress at the time of failure

f_{calc} = calculated stress: ACI 318-14

Straight Development Length of Deformed Bars in Tension

- Modification in simplified provisions of Table 25.4.2.3
- Ψg : new modification factor based on grade of reinforcement:
- Grade 80, 1.15
- Grade 100, 1.30

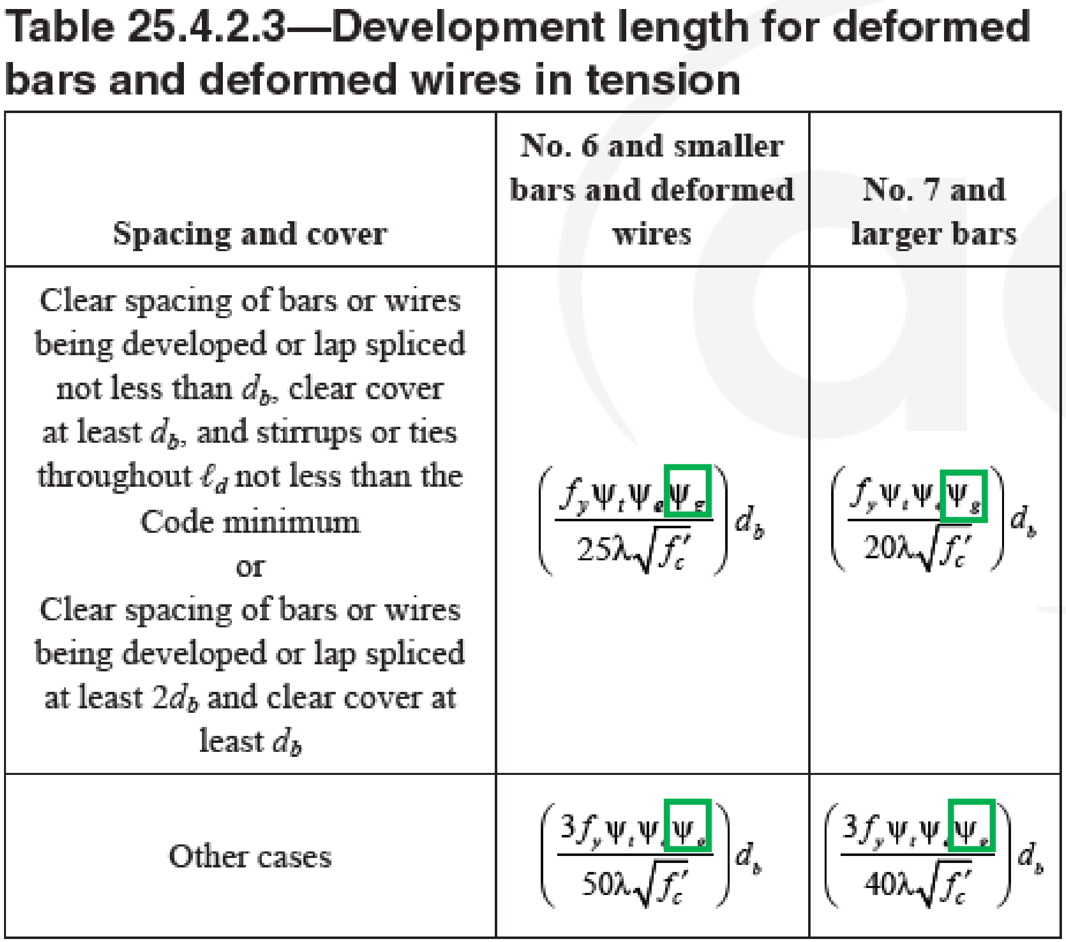


Table 25.4.2.3—Development length for deformed bars and deformed wires in tension

Spacing and cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b, clear cover at least d_b, and stirrups or ties throughout ℓ_d not less than the Code minimum or Clear spacing of bars or wires being developed or lap spliced at least $2d_b$ and clear cover at least d_b	$\left(\frac{f_y \psi_t \psi_e \boxed{\psi_g}}{25\lambda\sqrt{f'_c}}\right)d_b$	$\left(\frac{f_y \psi_t \psi_e \boxed{\psi_g}}{20\lambda\sqrt{f'_c}}\right)d_b$
Other cases	$\left(\frac{3f_y \psi_t \psi_e \boxed{\psi_g}}{50\lambda\sqrt{f'_c}}\right)d_b$	$\left(\frac{3f_y \psi_t \psi_e \boxed{\psi_g}}{40\lambda\sqrt{f'_c}}\right)d_b$

Straight Development Length of Deformed Bars in Tension

- Modification in general development length equation 25.4.2.4(a)

$$\ell_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s\boldsymbol{\psi_g}}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b$$

Modification factors

λ : Lightweight
ψ_t : Casting position
ψ_e : Epoxy
ψ_s : Size
$\boldsymbol{\psi_g}$: Reinforcement grade

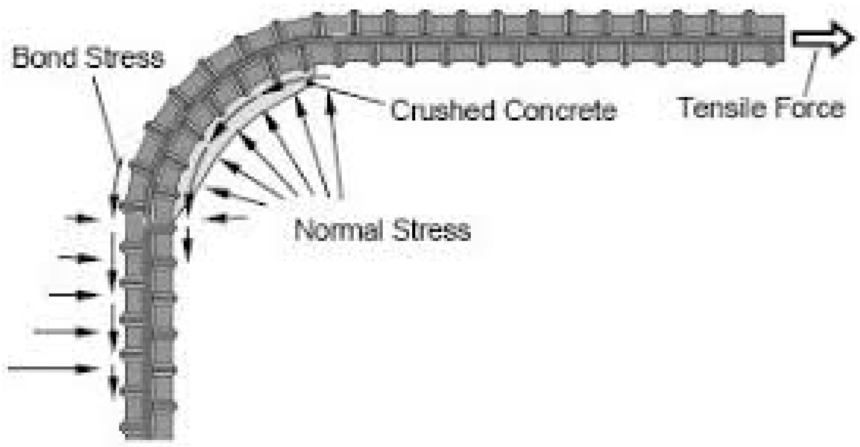
- Provision 25.4.2.2

  $\boldsymbol{K_{tr} \geq 0.5d_b}$ for $\boldsymbol{f_y \geq}$ **80,000 psi** , if longitudinal bar spacing **< 6 in**.

$$K_{tr} = \frac{40A_{tr}}{s \cdot n}$$

Development Length

- Deformed Bars and Deformed Wires in Tension
- Standard Hooks in Tension
- Headed Deformed Bars in Tension

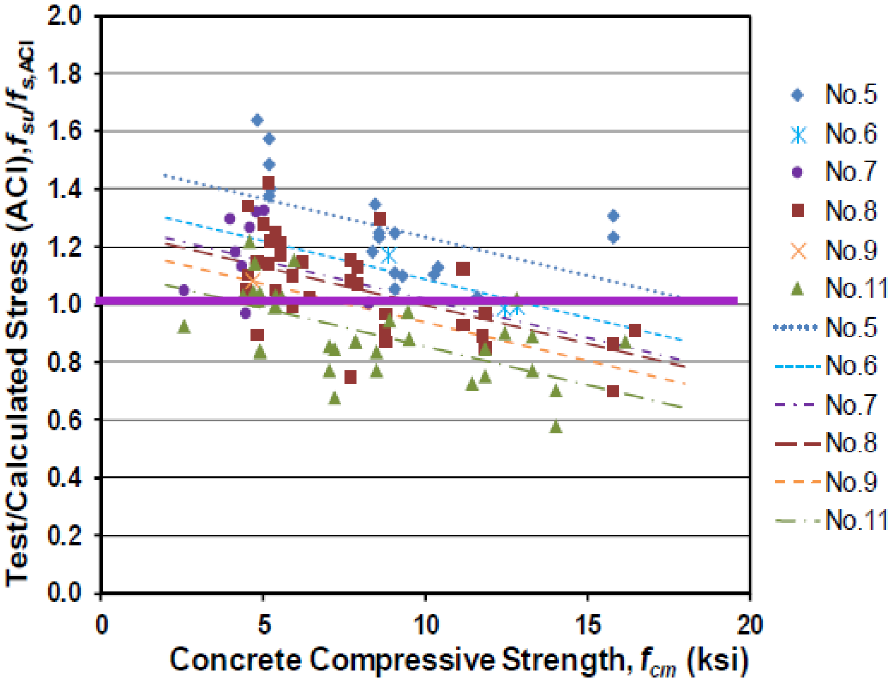


Development Length of Std. Hooks in Tension

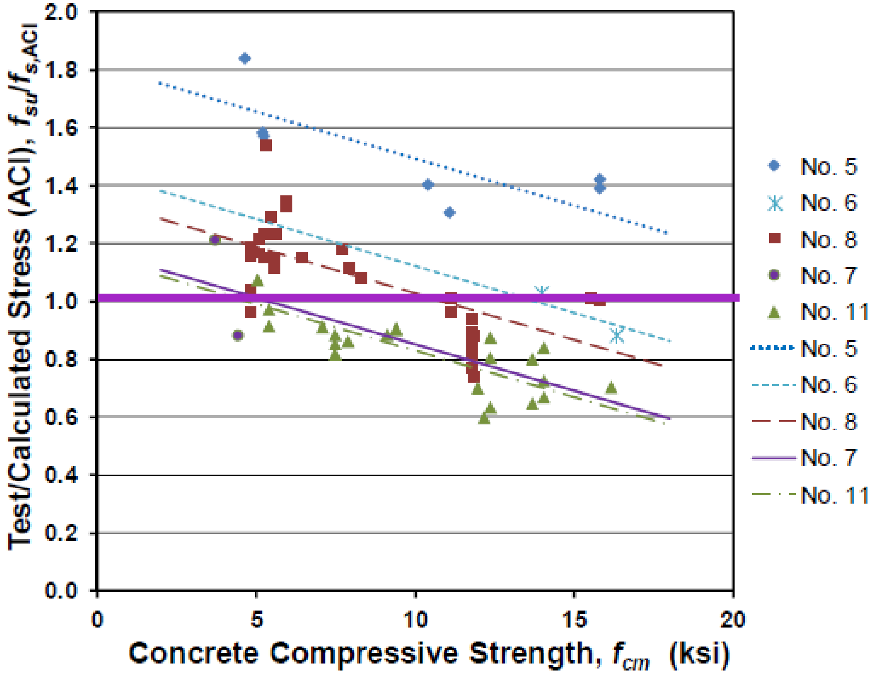
- Failure Modes

- Mostly, front and side failures
    - Dominant front failure (pullout and blowout)
    - **Blowouts** were more sudden in nature

Development Length of Standard Hooks in Tension



Unconfined



Confined

f_{SU} = stress at anchorage failure for the hooked bar

$f_{s,ACI}$ = stress predicted by the ACI development length equation

Development Length of Standard Hooks in Tension

- **25.4.3.1**—Development length of standard hooks in tension is the greater of (a) through (c):

(a) $$\left(\frac{f_y \psi_e \boldsymbol{\psi_r \psi_o \psi_c}}{55\lambda\sqrt{f_c'}}\right) d_b^{\mathbf{1.5}}$$

(b) $8d_b$

(c) 6 in

ACI 318- 14

$$\ell_{dh} = \left(\frac{f_y \psi_e \boldsymbol{\psi_c \psi_r}}{50\lambda\sqrt{f_c'}}\right) d_b$$

- Modification factors

  $\boldsymbol{\psi_r}$: Confining reinforcement (**redefined)**

  $\boldsymbol{\psi_o}$: Location **(new)**

  $\boldsymbol{\psi_c}$: Concrete strength (**new**)

Development Length of Standard Hooks in Tension

Table 25.4.3.2: Modification factors for development of hooked bars in tension

Modification factor	**Condition**	**Value of factor**
318-14 Confining reinforcement, ψ_r	For 90-degree hooks of No. 11 and smaller bars (1) enclosed along ℓ_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $\mathbf{s \leq 3d_b}$, or (2) enclosed along the bar extension **beyond hook** including the bend within ties or stirrups perpendicular to ℓ_{ext} at $\mathbf{s \leq 3d_b}$	0.8
	Other	1.0
318-19 Confining reinforcement, ψ_r	For No.11 and smaller bars with $A_{th} \geq 0.4A_{hs}$ or $s \geq 6d_b$	1.0
	Other	1.6

Development Length of Standard Hooks in Tension

- (1) Confining reinforcement placed parallel to the bar (Typical in beam-column joint)
  - **Two or more** ties or stirrups parallel to ℓ_{dh} enclosing the hooks
  - Evenly distributed with a center-to-center **spacing ≤ $8d_b$**
  - within **$15d_b$ of the centerline** of the straight portion of the hooked bars

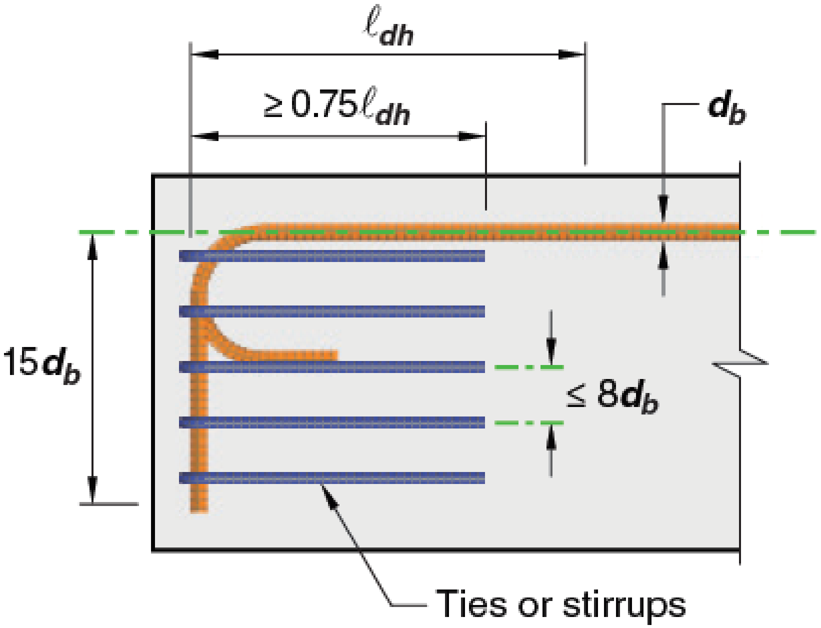


Fig. R25.4.3.3a

Development Length of Standard Hooks in Tension

- (2) Confining reinforcement placed perpendicular to the bar
  - **Two or more** ties or stirrups perpendicular to **ℓ_{dh}** enclosing the hooks
  - Evenly distributed with a center-to-center spacing **≤ 8db**

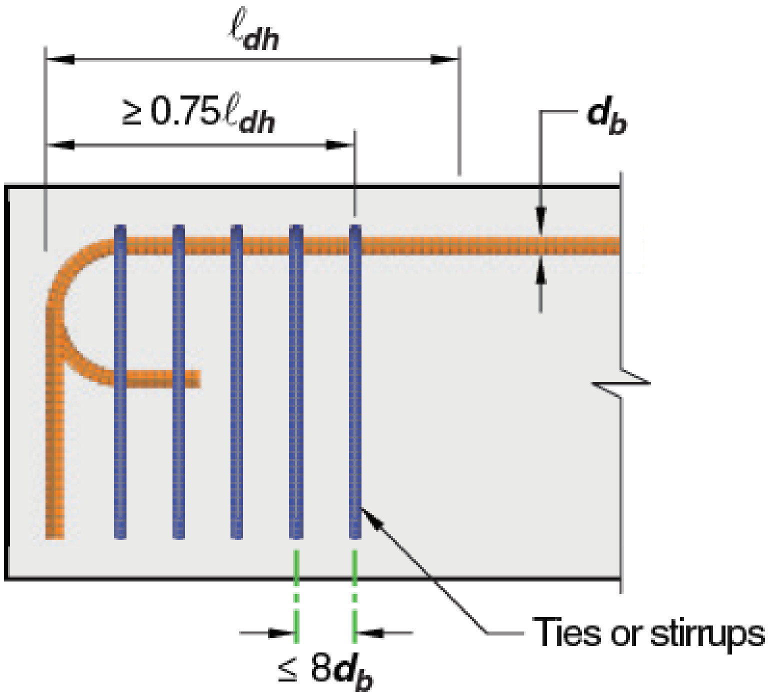


Fig. R25.4.3.3b

Development Length of Std. Hooks in Tension

Table 25.4.3.2: Modification factors for development of hooked bars in tension

Modification factor	**Condition**	**Value of factor**
318-14 Cover ψ_c	For No. 11 bar and smaller hooks with side cover (normal to plane of hook) ≥ 2-1/2 in. and for 90-degree hook with cover on bar extension beyond hook ≥ 2 in.	0.7
	Other	1.0
318-19 Location, ψ_o	For No.11 and smaller diameter hooked bars (1) Terminating inside column core w/ side cover normal to plane of hook ≥ 2.5 in., or (2) with side cover normal to plane of hook ≥ $6d_b$	1.0
	Other	1.25

Development Length of Std. Hooks in Tension

Modification factor	Condition	Value of factor
Concrete strength, ψ_c	For $f'_c < 6000$ psi	$f'_c/15{,}000 + 0.6$
	For $f'_c \geq 6000$ psi	1.0

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Changes to the Concrete Design Standard

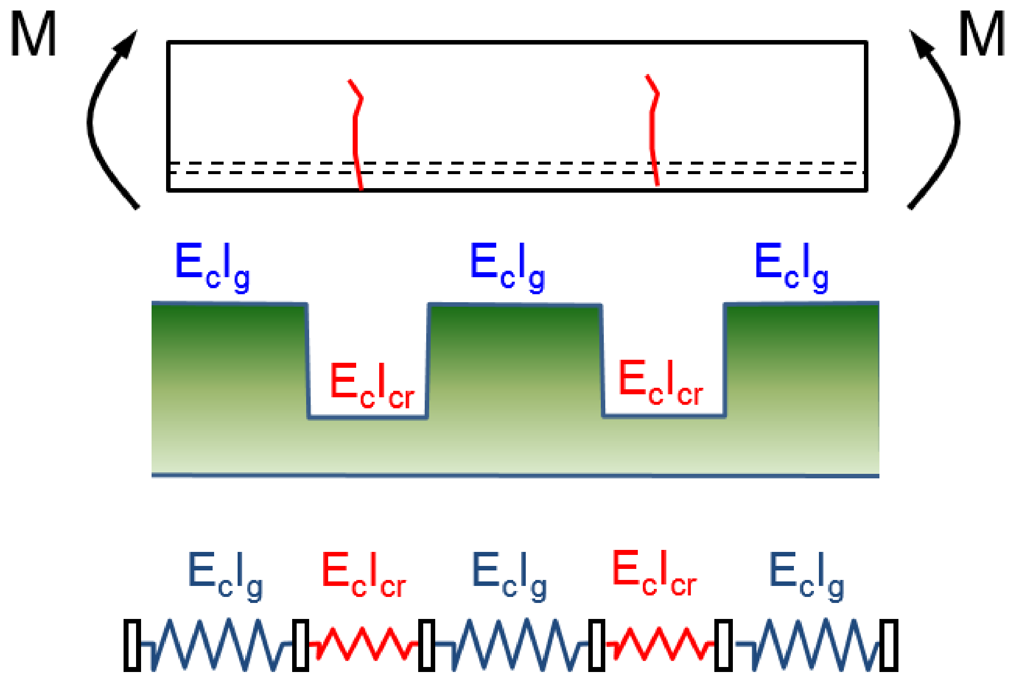
Deflection Equations

Concerns about deflection calculations

- Service level deflections based on Branson's equation underpredicted deflections for ρ below ≈ 0.8%

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

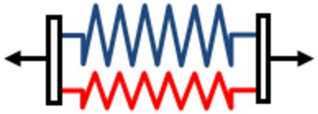
- Reports of excessive slab deflections (Kopczynski, Stivaros)
- High-strength reinforcement may result in lower reinforcement ratios



I_e should be the average of flexibilities

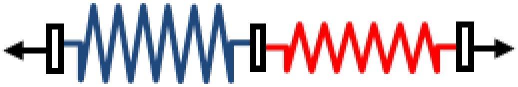
Comparison of Branson's and Bischoff's I_e

- Branson



$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \le I_g$$

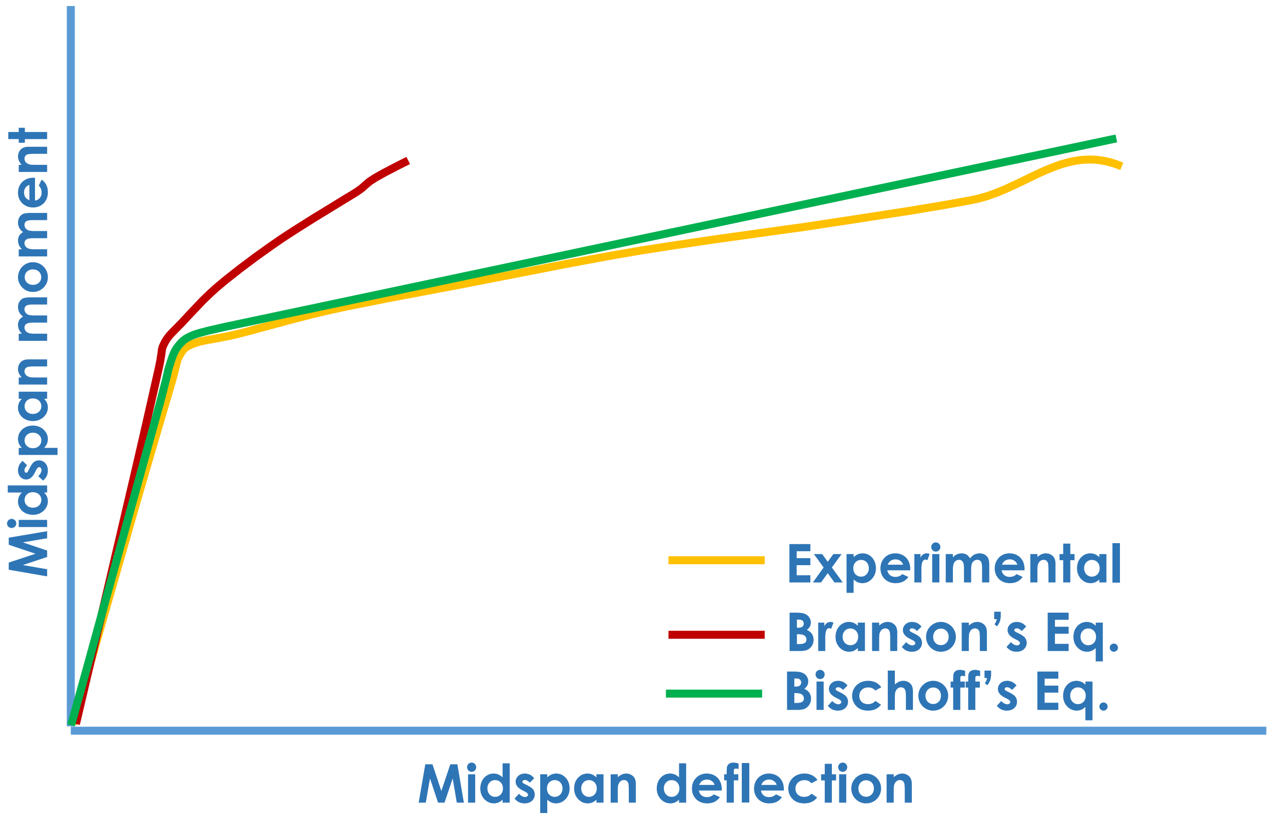
- Bischoff



$$\frac{1}{I_e} = \left(\frac{M_{cr}}{M_a}\right)^2 \frac{1}{I_g} + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^2\right) \frac{1}{I_{cr}} \le \frac{1}{I_g}$$

Branson combines stiffnesses. Bischoff combines flexibilities.

Lightly reinforced



Effective Moment of Inertia

- Table 24.2.3.5 ~ Inverse of Bischoff Eqn.

$$M_a > \ (2/3)M_{cr}, I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a}\right)^2 \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$M_a \leq \ (2/3)M_{cr}, I_e = I_g$$

- 2/3 factor added to account for:
  - restraint that reduces effective cracking moment
  - reduced concrete tensile strength during construction
- Prestressed concrete maintains use of Branson’s Eq. and 1.0 M_a.

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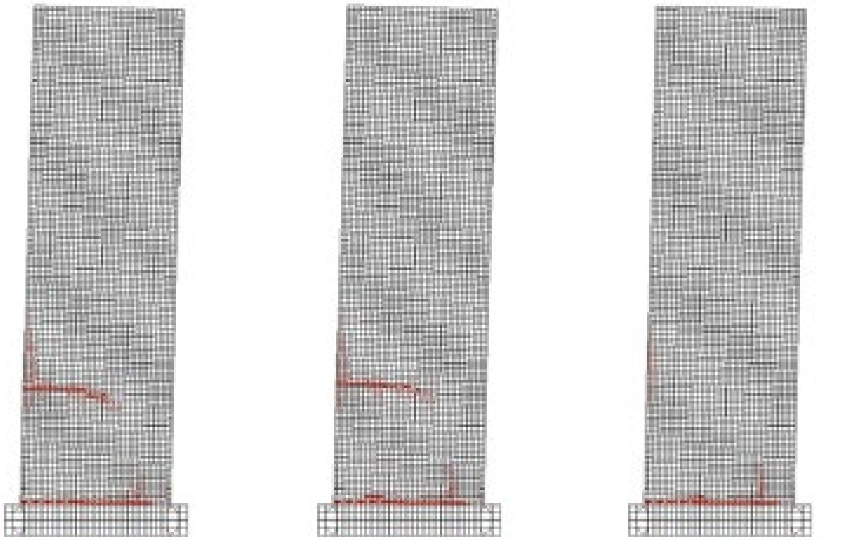
Changes to the Concrete Design Standard

Special Structural Walls

18.10.2.4—Longitudinal reinforcement ratio at ends of walls

$h_w/\ell_w \geq 2.0$

- Failures in Chile and New Zealand
- 1 or 2 large cracks
- Minor secondary cracks



(a) f'_c = 40 MPa (5580 psi)

(b) f'_c = 50 MPa (7250 psi)

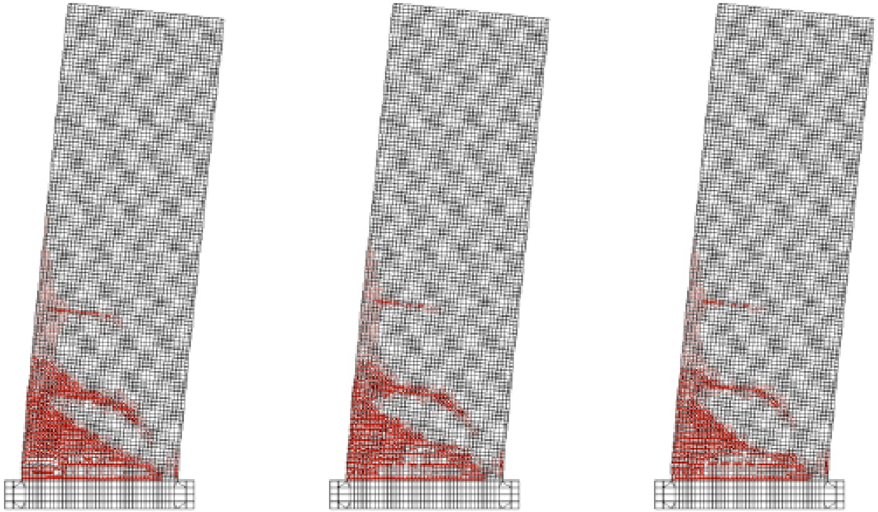
(c) f'_c = 60 MPa (8700 psi)

18.10.2.4—Longitudinal reinforcement ratio at ends of walls

New edge reinforcement ratio

$$\rho = \frac{6\sqrt{f_c'}}{f_y}$$

- Well distributed cracks
- Flexure yielding over longer length

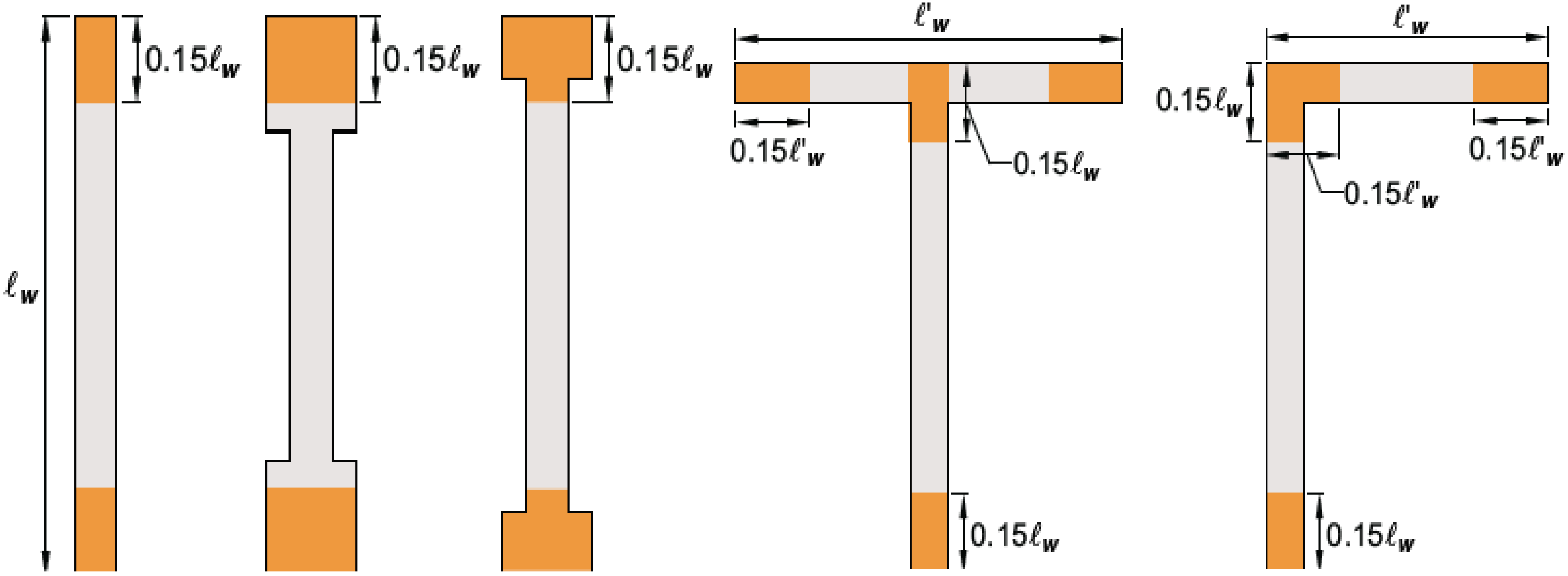


(a) $f'_c = 40$ MPa
(5583 psi)
$\rho_e = 0.64\%$

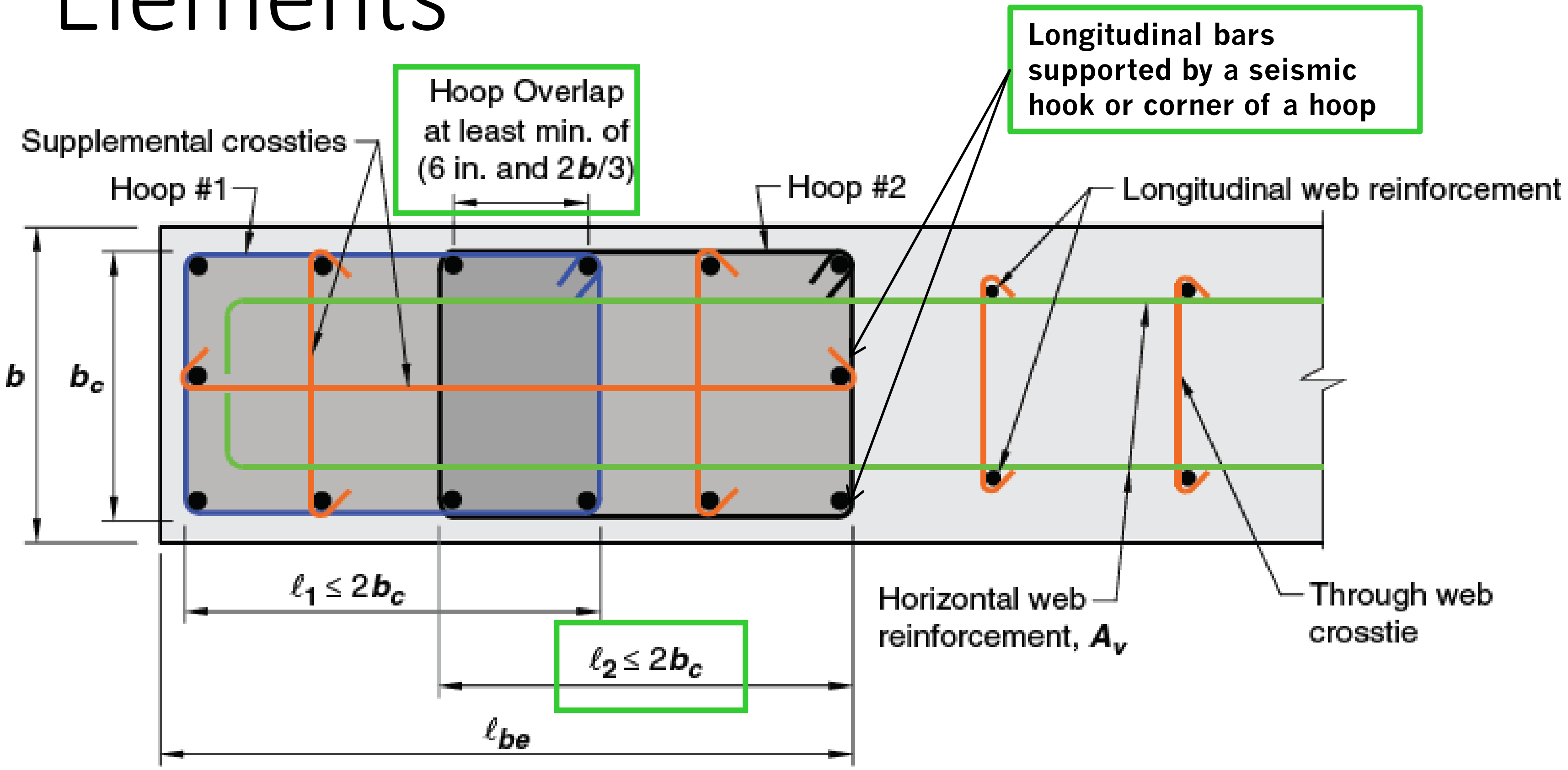
(b) $f'_c = 50$ MPa
(7250 psi)
$\rho_e = 0.71\%$

(c) $f'_c = 60$ MPa
(8700 psi)
$\rho_e = 0.78\%$

18.10.2.4—Longitudinal reinforcement ratio at ends of walls



18.10.6.4(f)—Special Boundary Elements



(b) Overlapping hoops with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement

$$b \ge \sqrt{0.025\ell_w c}$$

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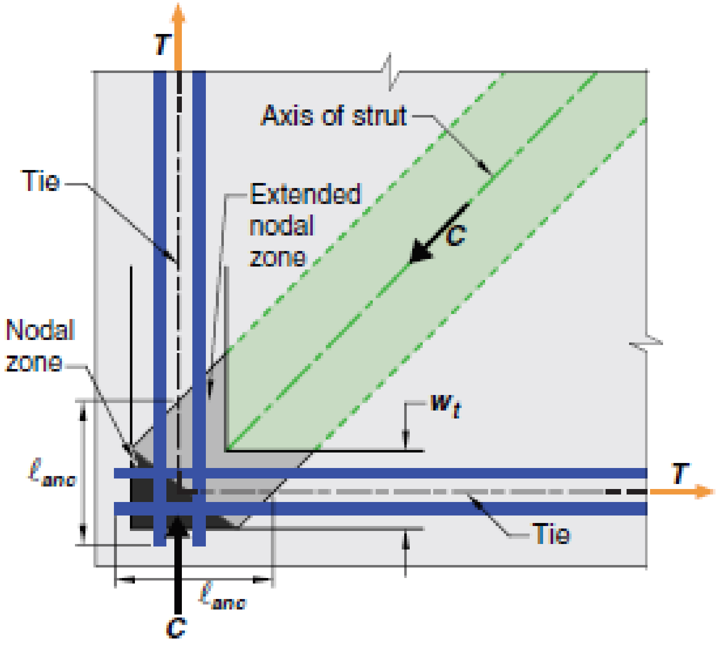
Changes to the Concrete Design Standard

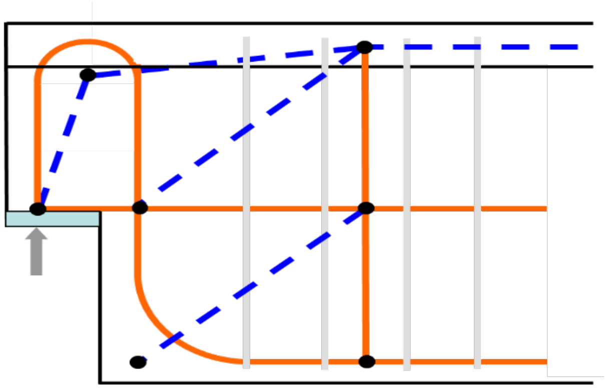
Strut-and-Tie Method

23.10 Curved-bar Nodes

Dapped-end T-beam

Nodal zones are generally too small to allow development



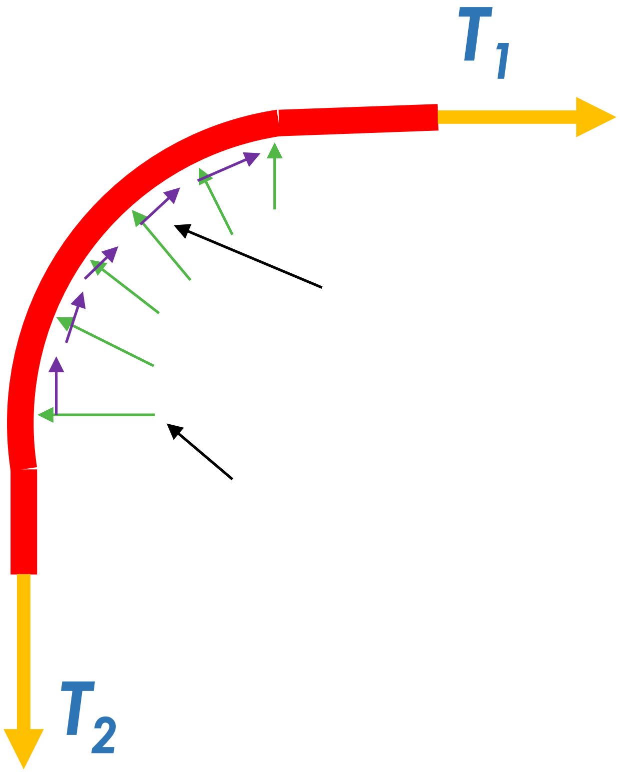


23.10 Curved-bar Nodes

Two issues that need to be addressed:

1. Slipping of bar

2. Concrete crushing

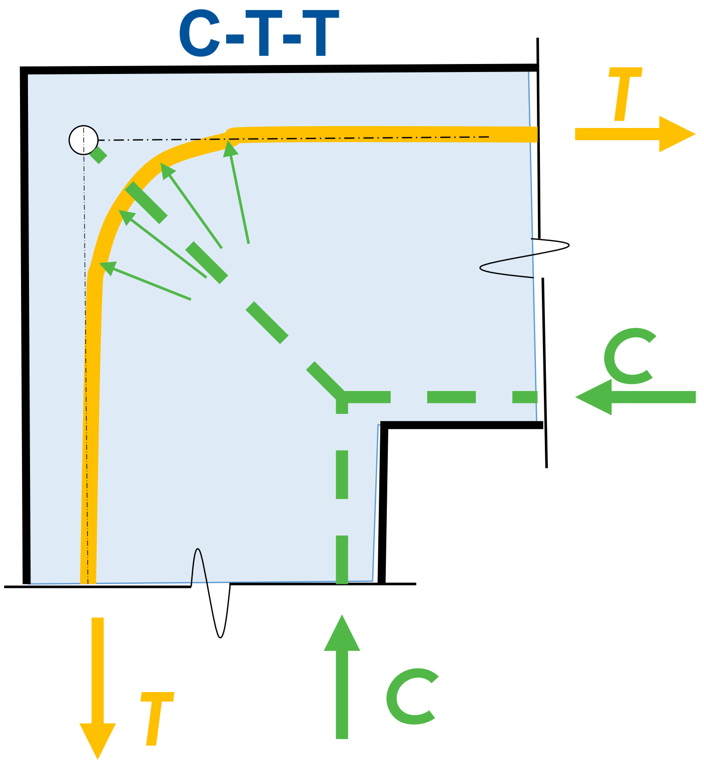


23.10 Curved-bar Nodes

$\theta < 180$ degree bend

$$r_b \geq \frac{2A_{ts}f_y}{b_s f'_c}$$

but not less than half bend diameter of Table 25.3



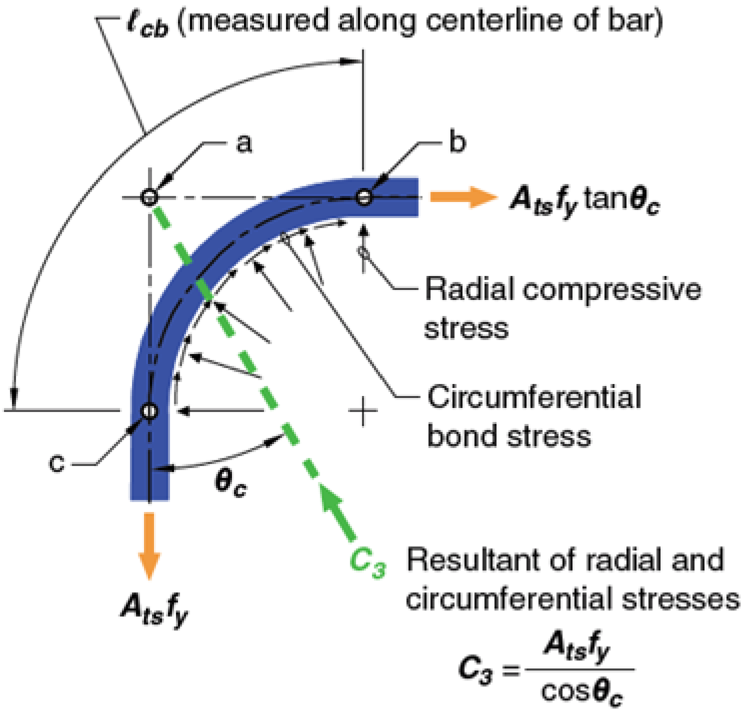
23.10 Curved-bar Nodes

23.10.6 The curved bar must have sufficient to develop difference in force

$\ell_{cb} > \ell_d(1 - \tan\theta_c)$

In terms of r_b

$$r_b > \frac{2\ell_d(1-\tan\theta_c)}{\pi} - \frac{d_b}{2}$$



Thank

You