ACI318-19 CODE REQUIREMENTS FOR DESIGN OF CONCRETE FLOOR SYSTEMS[1]

This Technical Note details the requirements of ACI318-19 for design of concrete floor systems, with emphasis on post-tensioning and their implementation in the ADAPT Builder Platform programs.

The implementation follows the ACI Code's procedure of calculating a "Demand," referred to as "design value" for each design section, and a "Resistance," for the same section, referred to as "design capacity." "Design value" and "design capacity" are generic terms that apply to displacements as well as actions. For each loading condition, or instance defined in ACI Code, the design is achieved by making the "resistance" exceed the associated demand "Design Value". Where necessary, reinforcement is added to meet this condition.

The implementation is broken down into the following steps:

- Serviceability limit state
- Strength limit state
- Initial condition (transfer of prestressing)
- Reinforcement requirement and detailing

In each instance, the design consists of one or more of the following checks:

- Bending of section
- Punching shear (two-way shear)
- Beam shear (one-way shear)
- Minimum reinforcement

In the following, the values in square brackets "[ ]" are defaults of the program. They can be changed by the user.

REFERENCES

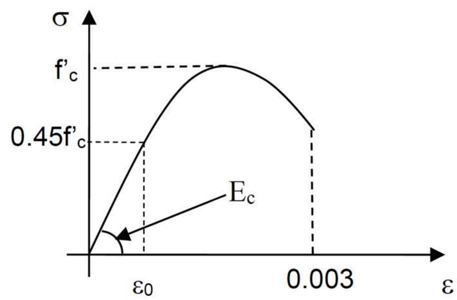
1. ACI-318-19

MATERIAL AND MATERIAL FACTORS

Concrete[2]

- Cylinder strength at 28 days, as specified by the user
  f'_c = characteristic compressive cylinder strength at 28 days [psi, MPa]

- Parabolic stress/strain curve with the maximum stress at f'_c and maximum strain at 0.003. Strain at limit of proportionality is not defined.



- Modulus of elasticity of concrete is automatically calculated and displayed by the program using f'_c, w_c, and the following relationship[3] of the code. User is given the option to override the code value and specify a user defined substitute.

$$E_c = {w_c}^{1.5} 33\sqrt{f'_c} \quad \text{US}$$
$$E_c = {w_c}^{1.5} 0.043\sqrt{f'_c} \quad \text{SI}$$

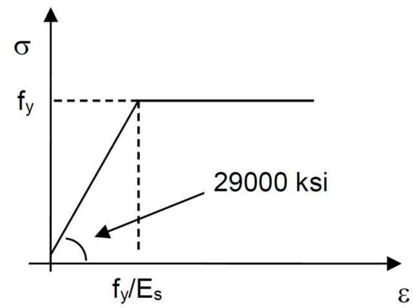
Where,

Ec = modulus of elasticity at 28 days [psi, MPa]
f'c = characteristic cylinder strength at 28 days [psi; MPa]
wc = density of concrete [150 lb/ft^3; 2400 kg/m^3]

Nonprestressed Steel[4]

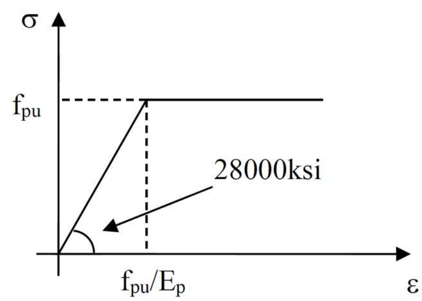
- Bilinear stress/strain diagram with the horizontal branch at f_y
- Modulus of elasticity (E_s) is user defined [29000 ksi; 200,000 MPa]
- No limit has been set for the ultimate strain of the mild steel in analysis and design

[2] ACI318-19, Section 22.2.2.3
[3] ACI318-19, Section 19.2.2.1
[4] ACI318-19, Section 20.2.2.1 and 20.2.2.2



Prestressing Steel

- A bilinear stress-strain curve is assumed
- Modulus of elasticity is user defined [28000 ksi; 190,000 MPa



LOADING

Selfweight determined based on geometry and unit weight of concrete, and other loads are user defined.

SERVICEABILITY

- **Load combinations**
  Total load combinations:
  - DL+1.0 LL+1.0 PT

  Sustained load combinations:
  - DL+0.3 LL+1.0 PT

The above combinations are the default settings of the program. User has the option to change them or create additional load combinations for service evaluation.

- **Stress checks**[5]
  Code stipulated stress limitations are used as the default allowables. However, the user can edit the default values.

[5] ACI318-19, Section 24.5

"Total load" condition:

- Concrete
  - Maximum compressive stress 0.60 f'_c. If calculated stress at any location exceeds the allowable, the program identifies the location graphically on the screen and notes it in its tabular reports.
  - The maximum allowable hypothetical tensile stress for one-way slabs and beams depends on the selection of design in one of the three classes of uncracked (U), transition (T) or cracked (C):

    Class U: $f_t \leq 7.5\sqrt{f'_c}$ $[US]$, $f_t \leq 0.62\sqrt{f'_c}$ $[SI]$

    Class T: $7.5\sqrt{f'_c} < f_t \leq 12\sqrt{f'_c}$ $[US]$, $0.62\sqrt{f'_c} < f_t \leq 1\sqrt{f'_c}$ $[SI]$

    Class C: $f_t > 12\sqrt{f'_c}$ $[US]$, $f_t > 1\sqrt{f'_c}$ $[SI]$

  - The program does not explicitly handle Class C. For one-way slabs and beams designed for Class C, additional serviceability requirements are required per Table R24.5.2.1 and are not defaulted to in the software when allowable stress settings are based on Class C. The user should modify stiffness properties and ensure proper cracked deflection combinations are considered to meet the necessary requirements.
  - For two-way slabs design only Class U (uncracked) is permitted:

    Class U with $f_t \leq 6\sqrt{f'_c}$ $[US]$, $f_t \leq 0.5\sqrt{f'_c}$ $[SI]$

- Nonprestressed Reinforcement
  - None specified - no check made
- Prestressing steel
  - None specified - no check made

"Sustained load" condition:

- Concrete
  - Maximum compressive stress 0.45 f'_c. If stress at any location exceeds, the program displays that location with a change in color (or broken lines for black and white display), along with a note on the text output.
  - The maximum allowable hypothetical tensile stress:

    Class U: $f_t \leq 7.5\sqrt{f'_c}$ $[US]$, $f_t \leq 0.62\sqrt{f'_c}$ $[SI]$

    Class T: $7.5\sqrt{f'_c} < f_t \leq 12\sqrt{f'_c}$ $[US]$, $0.62\sqrt{f'_c} < f_t \leq 1\sqrt{f'_c}$ $[SI]$

    Class C: $f_t > 12\sqrt{f'_c}$ $[US]$, $f_t > 1\sqrt{f'_c}$ $[SI]$

    The program does not explicitly handle Class C. For one-way slabs and beams designed for Class C, additional serviceability requirements are required per Table R24.5.2.1 and are not defaulted to in the software when allowable stress settings are based on Class C. The user should modify stiffness properties and ensure proper cracked deflection combinations are considered to meet the necessary requirements.

Two- way slab systems: Class U with $f_t \leq 6\sqrt{f'_c}$ $[US]$, $f_t \leq 0.5\sqrt{f'_c}$ $[SI]$

ADAPT uses $6\sqrt{f'_c}$ $[US]$, $0.5\sqrt{f'_c}$ $[SI]$ as its default for two-way systems
ADAPT uses $7.5\sqrt{f'_c}$ $[US]$, $0.62\sqrt{f'_c}$ $[SI]$ as its default for one-way systems

- Nonprestressed Reinforcement
  - None specified – no check made
- Prestressing steel
  - None specified - no check made

STRENGTH

- **Load combinations**[6]

  The following are the load combinations for gravity design of floor systems
  - D +1.0 Hyp
  - 1.2 D + 1.6 L + 0.5 (L_r or S or R) + 1.0 Hyp
  - 1.2 D + 1.6 (L_r or S or R) + 1.0 L + 1.0 Hyp

  The program designates reserved (default) load cases Dead and Live load. Therefore, loads classified as L_r, S and R should be input as additional load cases by the user.

- **Design of members for bending**[7]

  General

  The following describes the simplified procedure of ACI318 for the design of prestressed concrete sections. The relationships for the simplified procedure are given in Section 20.3.2. These are reproduced at the end of this Technical Note for ease of reference. The simplified procedure given in the code is restricted to the cases, where the effective stress in prestressing steel (f_{se}) after allowance for immediate and long-term losses is not less than 50% of its guaranteed ultimate strength ($0.5f_{pu}$)

  It uses code specified formulas for the determination of stress in prestressing steel at strength limit state (f_{se}). The rigorous design is based on strain compatibility, which is applied to bonded prestressed systems in ADAPT when the ACI code is used

[6] ACI318-19, Section 5.3.1
[7] ACI318-19, Section 22.2.1

Material and Stresses

The stress-strain relationship of the materials used is shown in Fig. 1 for the general case.

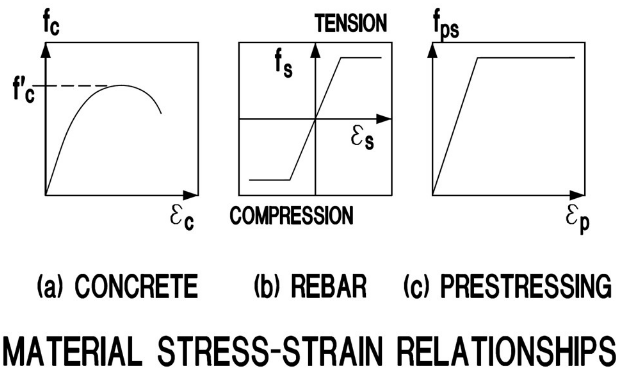


Figure 1

At ultimate condition the stresses are idealized using the following assumptions and relationships:

Concrete

- Plane sections remain plane
- Maximum concrete strain in compression is limited to 0.003
- Tensile capacity of the concrete is neglected
- For ductility of members designed in bending the maximum depth of the neutral axis "c" is limited to:

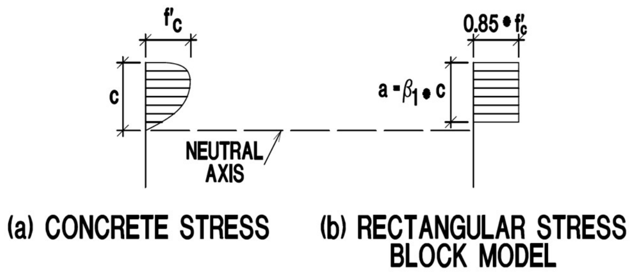
  $$\frac{c}{d_t} \leq 0.375$$

  Where, d_t is the distance from compression fiber to the farthest reinforcement. Where necessary, compression reinforcement is added to enforce the above requirement.
- If a section is made up of more than one concrete material, the entire section is designed using the concrete properties of lowest strength in that section.
- Rectangular concrete stress block with maximum stress equal to $0.85f'_c$ and the depth of stress block from the extreme compression fiber, a, equal to $\beta_1 c$ is used. See Fig. 2,

  where:

  $$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000 \geq 0.65 \quad \text{US}$$

  $$\beta_1 = 0.85 - 0.05(f'_c - 28)/7 \geq 0.65 \quad \text{SI}$$



CONCRETE STRESS AT LIMIT STATE

Figure 2

Non-Prestressed Reinforcement

Non-prestressed reinforcement, regardless of its yield stress, is referred to as rebar. Rebar may be added to supplement prestressing in developing the required moment resistance.

The stress-strain relationship for rebar is idealized as shown in Figure 1b. If the strain in concrete at the location of the rebar is less than the elastic limit of the rebar material, the rebar will not develop its yield stress. In this case, the calculation uses the stress obtained from the stress-strain diagram of the rebar material.

Prestressing

The stress developed in the prestressing steel at nominal strength is given by f_{ps}. If the effective stress in prestressing (f_{se}) (after allowance for short- and long-term losses) is not less than $0.5*f_{pu}$, the ACI simplified relationships may be used to estimate fps.

For grouted tendons, the code uses a parameter λ_p, for the calculation of stress in prestressing steel at strength limit state. λ_p is a constant depending on the material of prestressing tendon.

$\lambda_p = 0.55$ for f_{py}/f_{pu} not less than 0.80
$\lambda_p = 0.40$ for f_{py}/f_{pu} not less than 0.85
$\lambda_p = 0.28$ for f_{py}/f_{pu} not less than 0.90

Geometry

The general geometry of the section considered is shown in Fig. 3 for a T-section. Inverted L or rectangular sections are treated as special conditions of a T-section in which one, or both of the overhangs are reduced to zero. I-sections at ultimate strength are also treated as T-sections since the contribution of concrete in tension zone is disregarded.

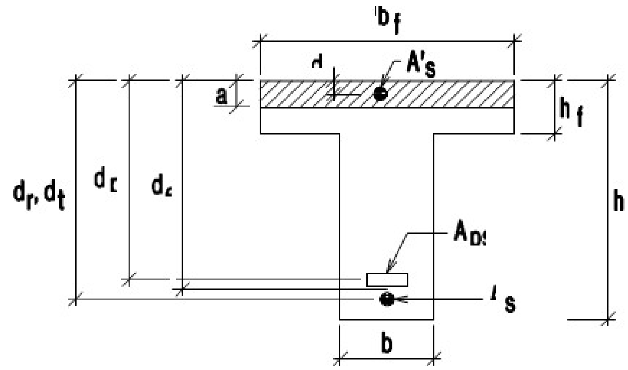


Figure 3

- For flanged sections, the following procedure is adopted:
  - If x_u is within the flange, the section is treated as a rectangle
  - If x_u exceeds the flange thickness, uniform compression is assumed over the flange. The stem is treated as a rectangular section.

- **Design Requirements**
  - The design moment (M_u) must be less than the moment which the section can develop, the nominal moment (M_n), reduced by a strength reduction factor (ϕ). The expression ϕM_n is referred to as design capacity.

$$M_u < \Phi M_n$$

  - The section should possess a minimum ductility. In this context, ductility is defined as the ratio of rotation of a section at failure (θ_u at location of plastic hinge) to rotation of the section at its elastic limit (θ_y at onset of plasticity). Figure 4a illustrates the definition of ductility as expressed by μ.
  - Experiments have established that ductility (μ) is primarily a function of the amount and position of prestressing and reinforcement in a section, as well as a section's geometry. The ductility of a section is controlled by the ratio c/d_t and the strength reduction factor ϕ. The minimum ductility required by the code is achieved through the limitation imposed on the ratio c/d_t. For the basic strength reduction factor (ϕ = 0.9) the ratio of c/dt is limited to 0.375.

$$c_{max} = 0.375 d_t$$
$$a_{max} = \beta_1 c_{max}$$

  - In a somewhat similar manner, the Canadian code (CSA-A23.3), the British code (BS 8110), and the European code (EC2) implement the ductility requirement by limiting the maximum depth of the neutral axis (c) to a fraction of d or h.

- Based on the threshold specified for the depth of the neutral axis (c), prior to triggering a reduction in the strength reduction factor (ϕ), six design conditions are identified. These are illustrated in Fig. 5.

CASE 1: PRESTRESSING ADEQUATE

Case 1 is the condition in which the available prestressing is in excess of that required to resist the design moment, M_u, with adequate ductility ($c < 0.375d_t$). Clearly, the section is satisfactory as is. No additional rebar is required.

CASE 2: PRESTRESSING PLUS TENSION REBAR

In Case 2, the available prestressing is not adequate to resist the design moment M_u. Rebar A_s is required to supplement the prestressing A_{ps}. The combined areas of A_{ps} and A_s result in ($a < a_{max}$). The larger circle shown in the figure around the rebar represents the maximum area of rebar ($A_{s,max}$) that would bring the section to its ductility threshold of ($a = a_{max}$).

CASE 3: PRESTRESSING AND TENSION REBAR NOT ADEQUATE

By increasing the applied moment of Case 2, a condition is reached for which the prestressing and the maximum rebar derived from the ductility relationships are no longer adequate to develop the required design capacity ϕM_n. In this case, the balance of design capacity must be generated by a force couple resulting from addition of tension and compression rebar. Generally, the area of the added compression steel (A'_s) will be equal to the added tension steel in excess of $A_{s,max}$. The exception is when the bars are positioned such that one or both of them would not yield.

CASE 4: OVER-REINFORCED SECTION, DESIGN BASED ON COMPRESSION ZONE

In Case 4, the amount of available prestressing is excessive, in that ($c > 0.375\ d_t$). As a result the ductility threshold is exceeded. The section can still be considered a satisfactory design, provided that the design moment (M_u) is less than the design capacity of the compression zone of the section. The design capacity of the compression zone is determined with the depth of the neutral axis assumed at c_{max}. The relationships are (Figs. 6 and 7):

$$\Phi M_n = \Phi\left\{\left[C_c(d_p - \frac{a}{2})\right] + \left[C_s(d_p - d')\right] + \left[C_f(d_p - \frac{a}{2})\right]\right\}$$

For a T-section where "a" falls in the stem use the following formula:

$$\Phi M_n = \Phi\left\{\left[C_c(d_p - \frac{a}{2})\right] + \left[C_s(d_p - d')\right] + \left[C_f(d_p - \frac{h_f}{2})\right]\right\}$$

In the above relationships, C_s refers to the component of the force from the compression steel, if available.

CASE 5: PRESTRESSING AND COMPRESSION REBAR

As in Case 4, the design capacity of the compression zone, ϕM_n, is not adequate to resist the design moment, M_u. That is to say $M_u > \phi M_n$. In this case, rebar must be added to the compression zone. The design capacity of the section is based on the force developed in the compression zone with a_{max}. The expressions in the section "Relationships" below apply. The following gives the compression force in its expanded form.

$$C = A'_s * f_{com} + 0.85(0.5b\beta_1 d_r - A_{s'})f'_c$$

CASE 6: PRESTRESSING AND COMPRESSION REBAR NOT ADEQUATE

Case 6 is one in which the prestressing alone is in excess of that required to satisfy a_{max} criterion, and the maximum compression rebar permissible. An acceptable design can be achieved by resisting the excess moment through addition of rebar for equal tension and compression forces such as to maintain the expressions in the section "Relationships" below.

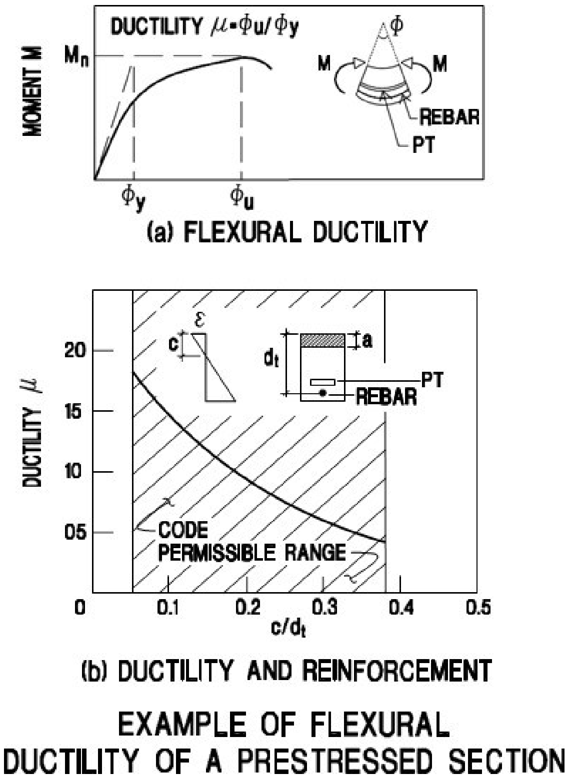


Figure 4

- **Relationships**
  - For given geometry, material properties, and amount of prestressing, the design is achieved by obtaining the minimum amount of rebar that develops the required design capacity, ϕM_n.
  - The forces at the strength limit state are defined in Figs. 6 and 7.
  - Tension equals compression, T = C, where:

$$T = T_p + T_s$$
$$T_p = A_{ps} f_{ps}$$
$$T_s = A_s f_{ten}$$
$$C = C_f + C_c + C_s$$
$$C_c = 0.85(b * a - A'_s) f'_c$$
$$C_s = A'_s f_{com}$$
$$C_f = 0.85\left(b_f - b\right) * a * f'_c$$

  - $M_u < \Phi M_n$
  - $c < 0.375d$
  - For flanged sections, as shown in Fig. 7, the contribution of flange overhang

to the compression zone is represented by an equivalent compressive force located at the height of the centroid of overhang's compression block. The force developed by the overhang is:

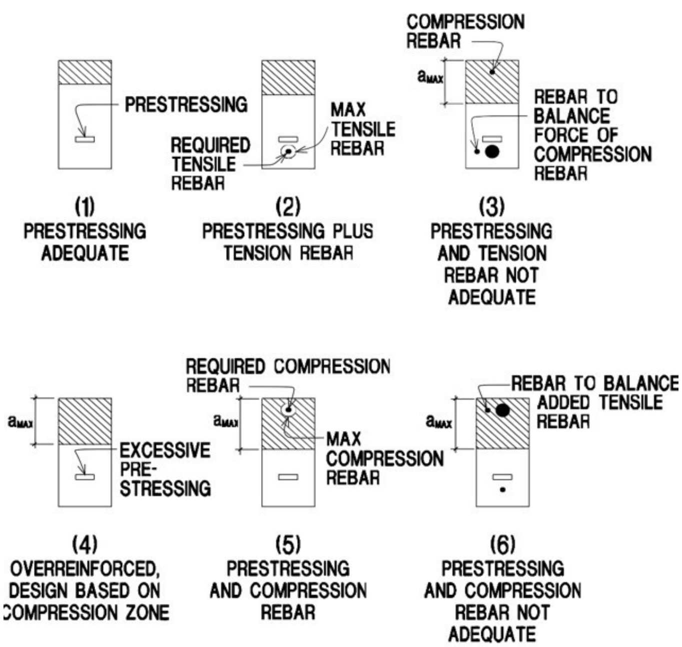
$$C_f = 0.85(b_f - b) * a * f'_c$$

- At every section of a flexural post-tensioned member with bonded tendons, the following will be satisfied[8]:

$$M_n \geq 1.2M_{cr}$$

Where,

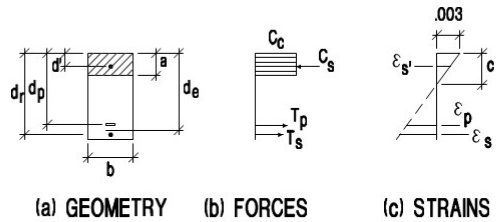
M_{cr} = cracking moment = S*(f_p + f_t)
S = section modulus
f_p = stress due to post-tensioning
f_t = tensile strength of the concrete



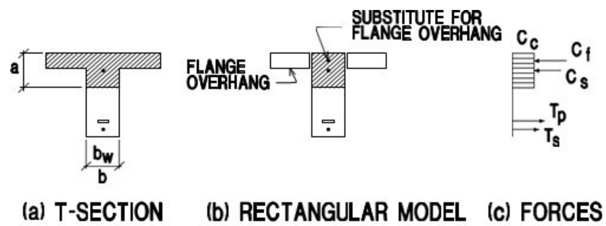
DESIGN CONDITIONS

Figure 5

[8] ACI318-19, Section 7.6.2.1



FORCES AND STRAINS AT LIMIT STATE
Figure 6



FORCES IN FLANGED SECTIONS
Figure 7

- **One-way Shear**[9]
    - The design is based on the following:

$$\Phi V_n \geq V_u$$
$$V_n = V_c + V_s$$

where,

V_n = factored shear resistance
V_u = factored shear force due to design loads
V_c = shear resistance attributed to the concrete
V_s = shear resistance provided by shear reinforcement
b_w = width of the web [in]
d = effective shear depth [in]
$\sqrt{f'_c} \leq 100psi, 8.3\ MPa$

- Design shear strength of concrete, V_c
    - Non-prestressed members[10]
        - For members where $A_v \geq A_{v,min}$:

[9] ACI318-19, Section 22.5
[10] ACI318-19, Section 22.5.5

$$Vc = \left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d \qquad \text{US}$$
$$Vc = \left[0.17\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d \qquad \text{SI}$$

- For members where $A_v < A_{v,min}$:

$$Vc = \left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d \qquad \text{US}$$
$$Vc = \left[0.66\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d \qquad \text{SI}$$

Where,

$$V_{c,\max} = 5\lambda\sqrt{f'_c} b_w d$$
$$\frac{N_u}{6A_g} \leq 0.05\sqrt{f'_c}$$

λ = a modification factor for concrete strength
1 for normal weight concrete
0.85 for sand light-weight concrete
0.75 for all-light-weight concrete

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

Where N_u/A_g is in psi, MPa

- Prestressed members

$$V_c = \left[0.6\lambda\sqrt{f'_c} + 700\frac{V_u d_p}{M_u}\right] b_w d$$
$$2\lambda\sqrt{f'_c} b_w d \leq V_c \leq 5\lambda\sqrt{f'_c} b_w d$$

Where $\frac{V_u d_p}{M_u} \leq 1$

- Shear reinforcement, A_v[11]

  - If $V_u - \Phi V_c > V_{n,max}$[12] — shear is excessive. Revise the section or increase the concrete strength

    Where $V_{n,max}$=

    $$V_{n,max} = 8\sqrt{f'_c} b_w d \qquad \text{US}$$

    $$V_{n,max} = 0.66\sqrt{f'_c} b_w d \qquad \text{SI}$$

  - If $V_u < 0.5\Phi V_c$ — no shear reinforcement is required

  - If $0.5\Phi V_c < V_u < \Phi V_c$ — $A_v = A_{v,min}$[13]

  - If $V_u > \Phi V_c$ — A_v[14] $= \frac{(V_u - \Phi V_c)s}{\Phi f_{yt} d} \geq A_{v,min}$

  - For nonprestressed beams, if $\Phi\lambda\sqrt{f'_c} b_w d \leq V_u < \Phi V_c$, $A_{v,min}$ is not required in the conditions per ACI318-19 Table 9.6.3.1
  - For prestressed beams, if $0.5\Phi V_c \leq V_u < \Phi V_c$, $A_{v,min}$ is not required in the conditions per ACI318-19 Table 9.6.3.1, where $A_{v,min}$,

  - Nonprestressed members:

    $$A_{v,min} = 0.75\sqrt{f'_c}\frac{b_w s}{f_{yt}} \geq 50\frac{b_w s}{f_{yt}} \qquad \text{US}$$

    $$A_{v,min} = 0.062\sqrt{f'_c}\frac{b_w s}{f_{yt}} \geq 0.35\frac{b_w s}{f_{yt}} \qquad \text{SI}$$

  - Prestressed members:

    $$A_{v,min} = \min\left\{\max\left[0.75\sqrt{f'_c}\frac{b_w s}{f_{yt}}, 50\frac{b_w s}{f_{yt}}\right], \frac{A_{ps} f_{pu}}{80 f_{yt} d}\sqrt{\frac{d}{b_w}}\right\} \qquad \text{US}$$

    $$A_{v,min} = \min\left\{\max\left[0.062\sqrt{f'_c}\frac{b_w s}{f_{yt}} \geq 0.35\frac{b_w s}{f_{yt}}\right], \frac{A_{ps} f_{pu}}{80 f_{yt} d}\sqrt{\frac{d}{b_w}}\right\} \qquad \text{SI}$$

    Where,

    s = longitudinal spacing of vertical stirrups [in,mm].

    f_{yt} = characteristic strength of the stirrup [psi, MPa]

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[11] ACI318-19, Section 22.5.8.5
[12] ACI318-19, Section 22.5.1.2
[13] ACI318-19, Section 9.6.3.4
[14] ACI318-19, Section 22.5.8.5.3

- Maximum spacing of the legs of shear reinforcement, $s_{v,max}$[15]:
- Nonprestressed members:

US: $S_{v,max}$ =

d/2 ≤24in if $(V_u - \Phi V_c) < \Phi 4\sqrt{f'_c} b_w d$

d/4 ≤12in if $\Phi 4\sqrt{f'_c} b_w d < (V_u - \Phi V_c) < \Phi 8\sqrt{f'_c} b_w d$

SI: $S_{v,max}$ =

d/2 ≤600 mm if $(V_u - \Phi V_c) < \Phi 0.33\sqrt{f'_c} b_w d$

d/4 ≤300mm if $\Phi 0.33\sqrt{f'_c} b_w d < (V_u - \Phi V_c) < \Phi 0.66\sqrt{f'_c} b_w d$

Prestressed members:

US: $S_{v,max}$ =

0.75h ≤24in if $(V_u - \Phi V_c) < \Phi 4\sqrt{f'_c} b_w d$

0.375h ≤12in if $\Phi 4\sqrt{f'_c} b_w d < (V_u - \Phi V_c) < \Phi 8\sqrt{f'_c} b_w d$

SI: $S_{v,max}$ =

0.75h ≤600mm if $(V_u - \Phi V_c) < \Phi 0.33\sqrt{f'_c} b_w d$

0.375h ≤300mm if $\Phi 0.33\sqrt{f'_c} b_w d < (V_u - \Phi V_c) < \Phi 0.66\sqrt{f'_c} b_w d$

- **Two-way shear**
  - **Categorization of columns**
    - Based on the geometry of the floor slab at the vicinity of a column, each column is categorized into to one of the following options:
      1. **Interior column:** Each face of the column is at least four times the slab thickness away from a slab edge
      2. **Edge column:** One side of the column normal to the axis of the moment is less than four times the slab thickness away from the slab edge
      3. **Corner column:** Two adjacent sides of the column are less than four times the slab thickness from slab edges parallel to each
      4. **End column:** One side of the column parallel to the axis of the moment is less than four times the slab thickness from a slab edge
    - In cases 2, 3 and 4, the column is assumed to be at the edge of the slab, however, the software shear design contains a user option to extend the section the modeled free edge of the slab. When this option is not used, the

[15] ACI318-19, Section 9.7.6.2.2

overhang of the slab beyond the face of the column is not included in the calculations. Hence, the analysis performed is somewhat conservative.

- **Stress calculation**[16]
  - The maximum factored shear stress is calculated for several critical perimeters around the columns based on the combination of the direct shear and moment**:**

$$v_{u1} = \frac{V_u}{A} + \frac{\gamma M_u c}{J_c}$$

$$v_{u2} = \frac{V_u}{A} + \frac{\gamma M_u c'}{J_c}$$

Where,
V_u = absolute value of the direct shear and
M_u = absolute value of the unbalanced column moment about the center
of geometry of the critical section
c and c' = distances from centroidal axis of critical section to the perimeter of
the critical section in the direction of the analysis
A = area of concrete of assumed critical section,
γ = ratio of the moment transferred by shear and
J_c = moment of inertia of the critical section about the axis of moment.

  - The implementation of the above in ADAPT is provided with the option of allowing the user to consider the contribution of the moments separately or combined. ACI318 however recommends that due to the empirical nature of its formula, punching shear check should be performed independently for moments about each of the principal axis[17]. For a critical section with dimension of b_1 and b_2 and column dimensions of c_1, c_2 and average depth of d, A_c, J_c, c, γ and M_u are:

1. Interior column:

$$A = 2(b_1 + b_2)d$$

[16] ACI318-19, Section 8.4.4.2
[17] "Concrete Q&A – Checking Punching Shear Strength by the ACI Code," Concrete International, November 2005, pp 76.

$$c = \frac{b_1}{2}$$

$$J_c = \frac{b_1 d^3}{6} + \frac{d{b_1}^3}{6} + \frac{{b_1}^2 b_2 d}{2}$$

$$\gamma = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$M_u = abs(M_{u,direct})$$

2. End column: (b_1 is perpendicular to the axis of moment)

$$A = (2b_1 + b_2)d$$

$$c = \frac{{b_1}^2}{2b_1 + b_2}$$

$$J_c = \frac{b_1 d^3}{6} + \frac{d{b_1}^3}{6} + 2b_1 d\left(\frac{b_1}{2} - c\right)^2 + b_2 d c^3$$

$$\gamma = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$M_u = abs\left[M_{u,direct} - V_u\left(b_1 - c - \frac{c_1}{2}\right)\right]$$

3. Corner Column:

$$A = (b_1 + b_2)d$$

$$c = \frac{{b_1}^2}{2b_1 + 2b_2}$$

$$J_c = \frac{b_1 d^3}{12} + \frac{d{b_1}^3}{12} + b_1 d\left(\frac{b_1}{2} - c\right)^2 + b_2 d c^3$$

$$\gamma = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$M_u = abs\left[M_{u,direct} - V_u\left(b_1 - c - \frac{c_1}{2}\right)\right]$$

4. Edge column: (b_1 is perpendicular to the axis of moment)

$$A = (b_1 + 2b_2)d$$

$$c = \frac{b_1}{2}$$

$$J_c = \frac{b_1 d^3}{12} + \frac{d {b_1}^3}{12} + 2 b_2 d c^2$$

$$\gamma = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}$$

$$M_u = abs\left[M_{u,direct}\right]$$

- **Allowable stress[18]:**
  - For nonprestressed member and prestressed member where columns are less than $4h_s$ from a slab edge:

$$v_c = min\begin{Bmatrix} 4\lambda_s\lambda\sqrt{f'_c} \\ \left(2+\frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f'_c} \\ \left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c} \end{Bmatrix} \qquad \text{US}$$

$$v_c = min\begin{Bmatrix} 0.33\lambda_s\lambda\sqrt{f'_c} \\ 0.17\left(1+\frac{2}{\beta}\right)\lambda_s\lambda\sqrt{f'_c} \\ 0.083\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c} \end{Bmatrix} \qquad \text{SI}$$

Where,

β	= the ratio of the larger to the smaller side of the critical section
f'_c	= the strength of the concrete
α_s	= 40 for interior columns
	= 30 for edge and end columns
	= 20 for corner columns
b_o	= the perimeter of the critical section.

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

  - For prestressed members where columns are more than $4h_s$ from a slab edge:

$$v_c = \left(\beta_p\lambda\sqrt{f'_c} + 0.3 f_{pc}\right) + V_p$$

[18] ACI318-19, Section 22.6.5

Where,

$$\beta_p = min\left(3.5, \left(\frac{\alpha_s d}{b_o} + 1.5\right)\right) \quad \text{US}$$

$$\beta_p = min\left(0.29, 0.083\left(\frac{\alpha_s d}{b_o} + 1.5\right)\right) \quad \text{SI}$$

Where,

α_s = 40 for interior columns
= 30 for edge and end columns
= 20 for corner columns

b_0 = the perimeter of the critical section

f_{pc} = the average value of f_{pc} for the two directions ≤ 500 psi (3.5 MPa) ≥ 125 psi (0.9 MPa)

V_p = the factored vertical component of all prestress forces crossing the critical section. ADAPT conservatively assumes it as zero.

$\sqrt{f'_c} \leq 70\ psi, 0.5\ MPa$

- **Critical sections**[19]
  - The closest critical section to check the stresses is d/2 from the face of the column where 'd' is the effective depth of the slab or drop panel/cap. Subsequent sections checked are located d/2 away from the previous critical section. The program checks rectilinear critical sections to the location where no reinforcement is required. At that same critical section, the program then checks the least critical section (octagonal-shaped) for stress.
  - If the calculated stress exceeds the code-required limit

$$v_{max} = \Phi_v 2\lambda\sqrt{f'_c} \quad \text{US}$$

$$v_{max} = \Phi_v 0.17\lambda\sqrt{f'_c} \quad \text{SI}$$

    the program adds reinforcement and continues to check octagonal- shaped critical sections until the allowable stress is met.
  - If drop panels or cap components are modeled, stresses are also checked at d/2 from the face of the drop panel/cap in which 'd' is the effective depth of the slab. Subsequent sections are d/2 away from the previous critical section.
- **Stress check**: Calculated stresses are compared against the allowable stress[20]

[19] ACI318-19, Section 8.4.4 & 22.6.4
[20] ACI318-19, Section 22.6.5

If $v_u < \Phi_v v_c$ no punching shear reinforcement is required

If $v_u > \Phi_v v_{n,max}$ punching stress is excessive; revise the section

If $\Phi_v v_{n,max} > v_u > \Phi_v v_c$ provide punching shear reinforcement

For stirrups[21]:

$\Phi_v v_{n,max}$ = $\Phi_v 6\sqrt{f'_c}$ US

= $\Phi_v 0.5\sqrt{f'_c}$ SI

For studs[22]:

$\Phi_v v_{n,max}$ = $\Phi_v 8\sqrt{f'_c}$ US

= $\Phi_v 0.66\sqrt{f'_c}$ SI

- Where ϕ_v is the shear factor and $v_{n,max}$ is the maximum shear stress that can be carried out by the critical section including the stresses in shear reinforcement.
- Stress check is performed until no shear reinforcement is required. For drop panels or caps, stresses are checked within the drop panel or cap until the stress is less than the permissible stress and then checked outside the drop panel or cap region until the stress is less than the permissible value. In the case of a drop cap, if the first critical section at 0.5d from face of column (where 'd' is calculated from the depth of the drop cap) is located beyond the drop cap, the program uses the effective depth calculated for the slab for all critical sections checked.
- Vu shall not exceed[23]:

  $v_{max} = \Phi_v 2\lambda\sqrt{f'_c}$ US

  $v_{max} = \Phi_v 0.17\lambda\sqrt{f'_c}$ SI

  at the critical section located d/2 outside the outermost line of shear reinforcement that surround the column.

- **Shear reinforcement**
  - Where needed, shear reinforcement is provided according to the following[24]:

    $$A_v = \frac{(v_u - \Phi_v v_c)us}{\Phi_v f_v \sin(\alpha)}$$

[21] ACI318-19, Section 22.6.6.3
[22] ACI318-19, Section 22.6.6.3
[23] ACI318-19, Section 22.6.6.1
[24] ACI318-19, Section 22.6.7 & 22.6.8

For studs[25]: $A_v \geq A_{v,min} = \frac{2\sqrt{f'_c}us}{f_y}$

Where[26],

$$v_c = 2\lambda\sqrt{f'_c}\,[US], 0.17\lambda\sqrt{f'_c}\,[SI] \text{ for stirrups}$$

$$v_c = 3\lambda\sqrt{f'_c}\,[US], 0.25\lambda\sqrt{f'_c}\,[SI] \text{ for studs}$$

α = angle of shear reinforcement with the plane of slab

u = periphery of the critical section

s = spacing between the critical sections

- If required, shear reinforcement will be extended to the section where:

$$v_u \leq \Phi_v 2\lambda\sqrt{f'_c} \qquad \text{US}$$

$$v_u \leq \Phi_v 0.17\lambda\sqrt{f'_c} \qquad \text{SI}$$

- **Arrangement of shear reinforcements:**
  - Shear reinforcement can be in the form of shear studs or shear stirrups (links). In case of shear links, the number of shear links (N_{shear_links}) in a critical section and distance between the links ($Dist_{shear\_links}$) are given by:

$$N_{shear\_links} = \frac{A_v}{A_{shear\_link}}$$

$$Dist_{shear\_links} = \frac{u}{N_{shear\_link}}$$

  - The first layer of stirrups is provided at d/2 from the column face and the successive layers are at d/2 from the previous layer. The spacing between the adjacent stirrup legs in the first line of shear reinforcement shall not exceed 2d measured in a direction parallel to the column face.[27]
  - If shear studs are used, the number of shear studs per rail (N_{shear_studs}) and the distance between the studs ($Dist_{shear\_studs}$) are given by:

$$N_{shea\ \_studs} = \frac{A_v}{A_{shear\_stud} N_{rails}}$$

[25] ACI318-19, Section 22.6.8.3

[26] ACI318-19, Section 22.6.6.1

[27] ACI318-19, Section 8.7.6.3

$$Dist_{shear\_studs} = \frac{d_{slab}}{2N_{shear\_studs}}$$

- The spacing between the column face and the first peripheral line of shear reinforcement shall not exceed d/2. The spacing between adjacent shear reinforcement elements, measured on the perimeter of the first peripheral line of shear reinforcement, shall not exceed 2d. The spacing between peripheral lines of shear reinforcement, measured in a direction perpendicular to any face of the column, shall be constant[28].
- The default graphical and tabular output for stirrup or heads stud reinforcement report number of studs or vertical stirrup legs at a distance from face of column. Multiple quantities and spacings may be reported (e.g. 10@2", 5@4", 10@8"). The program gives an option for the reinforcement to be reported as uniform. In the case this option is used, the program calculates the total length for which reinforcement is required based on the default layout and applies the minimum spacing to this length. (e.g. 60@2").
- The program gives the option to utilize and limit the code required spacing[29] between stud rails and adds enough rails per column side to meet this requirement.

- **Minimum Two-Way Shear Resistance for Seismic Drift[30]:**
  - For slab-to-columns joints not specifically designed as part of the seismic-force-resisting system in Seismic Design Categories D, E, and F, the ACI318 code requires minimum shear resistance so as to reduce the likelihood of two-way shear failure where the design story drift ratio exceeds the design value.
  - This option is included in the software. If selected, the program ensures that at a distance of 4h (where 'h' is the slab thickness) from each column face, the following amount of shear reinforcement is required such that:

$$v_s \geq 3.5\sqrt{f'_c}$$

  - In the following expression used to calculate A_v, when the quantity $(v_u - \Phi v_c) \leq v_s$, the program uses $3.5\sqrt{f'_c}$ for critical sections up to 4h.

$$A_v = \frac{(v_u - \Phi_v v_c)us}{\Phi_v f_v \sin(\alpha)}$$

INITIAL CONDITION

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28 ACI318-19, Section 8.7.6.3
29 ACI318-19, Section 8.7.7.1.2
30 ACI318-19, Section 18.14.5.1

- **Load combinations**
  - ADAPT uses the following default values. User can modify these values.
  - 1.0 SW +1.15 PT
- **Allowable stresses**[31]
  - Tension:
    - At ends of simply supported members: $6\sqrt{f'_{ci}}[US], 0.5\sqrt{f'_{ci}}\ [SI]$
    - All others (program default): $3\sqrt{f'_{ci}}\ [US], 0.25\sqrt{f'_{ci}}\ [SI]$
  - Compression: $0.6f'_{ci}$
    - If the tensile stress exceeds the threshold, reinforcement is added in the tensile zone
- **Reinforcement**
  - Reinforcement will be provided for initial condition if tensile stress exceeds allowable stress. Rebar is provided based on ACI code and will be placed on tension side:

$$A_s = \frac{N_c}{0.5f_y}$$

Where:

As = Area of reinforcement

Nc = tensile force in the concrete based on the uncracked section

fy = Yield Stress of the steel but not more than 60 ksi

DETAILING

- **Reinforcement requirement and placing**
  - Nonprestressed member:
    - Minimum tension rebar
      - Beam[32]:

$$A_{s,min} = \frac{3\sqrt{f'_c}b_w d}{f_y} > \frac{200b_w d}{f_y} \quad \text{US}$$

$$A_{s,min} = \frac{0.25\sqrt{f'_c}b_w d}{f_y} > \frac{1.4b_w d}{f_y} \quad \text{SI}$$

where,

b_w = width of the web [in,mm]

$f_{y,max}$ = 80,000 psi

For statically determinate members with flange in tension:

b_w = minimum of {$2b_w$, width of the flange} [in,mm]

- Minimum rebar requirement will be waived if A_s provided is at least 1/3 greater than

[31] ACI318-19, Section 24.5.3

[32] ACI318-19, Section 9.6.1 & 9.6.2

that required by analysis.

- Slab[33]:

$$A_{s,min} = 0.0018A_g$$

$$s_{max} = \min\,(5h, 18in) \qquad \text{US}$$

$$s_{max} = \min\,(5h, 450mm) \qquad \text{SI}$$

- Prestressed member:
  - One way system with unbonded tendon:

    $$A_{s,min} = 0.004A_{ct}$$

    Where,

    A_{ct} = Area of the part of cross-section between the flexural tension face and the center of gravity of cross-section

  - Two way system with unbonded tendon:
    - Positive moment areas if tensile stress exceeds $2\sqrt{f'_c}$:

    $$A_{s,min} = \frac{N_c}{0.5f_y}$$

    - Negative moment areas at column supports:

    $$A_{s,min} = 0.00075A_{cf}$$

    - Where A_{cf} = larger gross cross-sectional area of the design strips in two orthogonal directions

[33] ACI318-19, Section 7.6.1, 7.6.2, 8.6.1, 8.6.2

APPENDIX

This appendix includes additional information directly relevant to the design of concrete structures, but not of a type to be included in the program.

- **Effective width of the flange**[34]
  - For T-Beams
    - Effective overhanging flange width on each side of web is the smallest of:
      - Ln/8
      - 8 times the flange thickness
      - ½ of the clear distance to the next web – not checked in program
  - For L-Beams
    - Effective overhanging flange width on each side is the smallest of:
      - Ln/12
      - 6 times the flange thickness
      - ½ of the clear distance to the next web – not checked in program
- **Analysis**
  - Arrangement of loads[35]:
  - Continuous beams and one-way slabs:
    - factored dead load on all spans with full factored live load on two adjacent spans
    - factored dead load on all spans with full factored live load on alternate spans
  - Two-way slabs[36]:
    - If the ratio of live over dead load exceeds 0.75, live load is skipped as in the following combination:
      - factored dead load on all spans with 3/4th of the full factored live load on the panel and on alternate panels; and
      - factored dead load on all spans with 3/4th of the factored live load on adjacent panels only.
- **Redistribution of moment**[37]
  - Redistribution is only permitted when the net tensile strain, ε_t, is not less than 0.0075.
  - Percentage of redistribution = $1000\varepsilon_t\% \leq 20\%$ where ε_t = net tensile strain in extreme layer of longitudinal tension steel at nominal strength.
- **Deflection**
  - Maximum permissible computed deflections are based on Table 24.2.2[38].

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[34] ACI318-19, Section 6.3.2
[35] ACI318-19, Section 6.4
[36] ACI318-19, Section 6.4
[37] ACI318-19, Section 6.6.5
[38] ACI318-19, Section 24.2.2

NOTATION

A	depth of compression block
A_{ps}	area of prestressing
A_s	area of tension steel
A'_s	area of compression steel
A_t	area of concrete in tension zone
B	width of rectangular beam, or stem of T-section
b_f	flange width of T-section
c	depth of neutral axis from compression fiber
C	total compression force
C_c	compression force due to concrete
C_f	compression force in flange overhang of T- or inverted L-beam
C_s	compression force due to compression reinforcement
D	dead load
d'	distance from extreme compression fiber to centroid of compression reinforcement
d_c	distance from extreme compression fiber to centroid of total compression block
d_p	distance from extreme compression fiber to centroid of prestressed reinforcement
d_r	distance from extreme compression fiber to tensile rebar
f'_c	28-day compressive strength of concrete
f_{com}	stress in compression rebar at its centroid
f_{ps}	stress in prestress reinforcement at nominal strength
f_{pu}	specified tensile strength of prestressing tendons
f_{py}	specified yield strength of prestressing tendons
f_{se}	effective stress in prestress reinforcement (after allowance for all prestress losses)
f_{ten}	stress in tension rebar at its centroid
f_y	specified yield strength of nonprestressed reinforcement
h	overall thickness of member
h_f	thickness of flange or T- or inverted L-beam
I	moment of inertia of section about centroidal axis
L	live load
M_u	factored moment
M_n	nominal moment
s	spacing of stirrups
T	total tension force
T_p	tension force due to prestressing
T_s	tension force due to tensile rebar
v_c	concrete shear strength
v_u	design shear stress
β_1	factor = a/c
Υ	factor for type of prestressing material
ε_c	concrete strain

ε_p	strain in tendons at centroid
ε'_s	strain in compressive steel at centroid
λ	modification factor reflecting the mechanical properties of the concrete
ϕ	strength reduction factor
μ	ductility factor
ρ	ratio of nonprestressed tension reinforcement = $A_s/b*d_r$
ρ'	ratio of nonprestressed compression reinforcement = $A_s'/b*d_r$
ρ_p	ratio of prestressed reinforcement = $A_{ps}/b*d_p$;
ω	$\rho*f_v/f'_c$;
ω'	$\rho'*f_v/f'_c$;
ω_p	ρ_p*f_{ps}/f'_c