Case Study in Reinforced Concrete
adapted from Simplified Design of Concrete Structures, James Ambrose, 7th ed.

Building description

The building is a three-story office building intended for speculative rental. Figure 17.37 presents a full-building section and a plan of the upper floor. The exterior walls are permanent. The design is a rigid perimeter frame to resist lateral loads.

Loads (UBC 1994)

*Live Loads:*
  - Roof: 20 lb/ft$^2$
  - Floors:
    - Office areas: 50 lb/ft$^2$ (2.39 kPa)
    - Corridor and lobby: 100 lb/ft$^2$ (4.79 kPa)
    - Partitions: 20 lb/ft$^2$ (0.96 kPa)

*Wind:* map speed of 80 mph (190 km/h); exposure B

*Assumed Construction Loads:*
  - Floor finish: 5 lb/ft$^2$ (0.24 kPa)
  - Ceilings, lights, ducts: 15 lb/ft$^2$ (0.72 kPa)
  - Walls (average surface weight):
    - Interior, permanent: 10 lb/ft$^2$ (0.48 kPa)
    - Exterior curtain wall: 15 lb/ft$^2$ (0.72 kPa)

Materials

Use $f'_c = 3000$ psi (20.7 MPa) and grade 60 reinforcement ($f_y = 60$ ksi or 414 MPa).

Structural Elements/Plan

*Case 1* is shown in Figure 17.44 and consists of a flat plate supported on interior beams, which in turn, are supported on girders supported by columns. We will examine the slab, and a four-span interior beam.

*Case 2* will consider the bays with flat slabs, no interior beams with drop panels at the columns and an exterior rigid frame with spandrel (edge) beams. An example of an edge bay is shown to the right. We will examine the slab and the drop panels.

For both cases, we will examine the exterior frames in the 3-bay direction.
Case 1:

Slab:

The slabs are effectively 10 ft x 30 ft, with an aspect ratio of 3, making them one-way slabs. Minimum depths (by ACI) are a function of the span. Using Table 7.3.1.1 for one way slabs the minimum is \( \frac{l_n}{28} \) with 5 inches minimum for fire rating. We’ll presume the interior beams are 12” wide, so

\[ l_n = 10 \text{ ft} - 1 \text{ ft} = 9 \text{ ft} \]

minimum \( t \) (or \( h \)) = \( \frac{9\times 12^{n/p}}{28} = 3.86\text{ in} \)

Use 5 in. (fire rating governs)

dead load from slab = \( \frac{150^{lb/ft^3} \cdot 5^{in}}{12^{in/ft}} = 62.5 \text{ lb/ft}^2 \)

total dead load = (5 + 15 + 62.5) lb/ft\(^2\) + 2” of lightweight concrete topping with weight of 18 lb/ft\(^2\) (0.68 KPa) (presuming interior wall weight is over beams & girders)

dead load = 100.5 lb/ft\(^2\)

live load (worst case in corridor) = 100 lb/ft\(^2\)

total factored distributed load (ASCE-7) of 1.2D+1.6L:

\[ w_u' = 1.2(100.5 \text{ lb/ft}^2) + 1.6(100 \text{ lb/ft}^2) = 280.6 \text{ lb/ft}^2 \]

**Maximum Positive Moments** from Figure 2-3, end span (integral with support) for a 1 ft wide strip:

\[ M_u (\text{positive}) = \frac{w_u \ell^2_n}{14} = \frac{w_u' \cdot 1\text{ ft} \cdot \ell^2_n}{14} = \frac{(280.6^{lb/ft^2})(1\text{ft})(9^{ft})^2}{14} \cdot \frac{1k}{1000lb} = 1.62 \text{ k-ft} \]
Maximum Negative Moments from Figure 2-5, end span (integral with support) for a 1 ft wide strip:

\[
M_u(\text{negative}) = \frac{w_u \ell_n^2}{12} = \frac{w_u \cdot 1 \text{ ft} \cdot \ell_n^2}{12} = \frac{(280.6^b/\beta^2)(1\text{ft})(9^b)^2}{12} \cdot \frac{1k}{1000lb} = 1.89 \text{ k-ft}
\]

The design aid (Figure 3.8.1) can be used to find the reinforcement ratio, \(\rho\), knowing \(R_n = M_u/\phi^2\) with \(M_n = M_u/\phi\), where \(\phi = 0.9\) (because \(\rho_{0.005} = 0.0135\)). We can presume the effective depth to the centroid of the reinforcement, \(d\), is 1.25" less than the slab thickness (with 3⁄4" cover and 1⁄2 of a bar diameter for a #8 (1") bar) = 3.75".

\[
R_n = \frac{1.89^{k-\rho}}{(0.9)^{(12^a/3.75^b)^2}} \cdot 12^{in/\rho} \cdot 1000^{lb/k} = 149.3 \text{ psi}
\]

so \(\rho\) for \(f'_c = 3000\text{ psi}\) and \(f_y = 60,000\text{ psi}\) is the minimum. For slabs, \(A_s\) minimum is 0.0018bt for grade 60 steel.

\[
A_s = 0.0018(12\text{in})(5\text{in}) = 0.108 \text{ in}^2/\text{ft}
\]
Pick bars and spacing off Table 3-7. Use #3 bars @ 12 in ($A_s = 0.11 \text{ in}^2$).

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</table>

Check the moment capacity. $d$ is actually 5 in – 0.75 in (cover) – $\frac{1}{2}$ (3/8 in bar diameter) = 4.06 in

$$a = A_s f_y / 0.85f'_c$$
$$b = 0.11 \text{ in}^2 / (60 \text{ ksi}) = 0.22 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d-a/2) = 0.9 (0.11 \text{ in}^2) / (60 \text{ ksi}) (4.06 \text{ in} - \frac{0.22 \text{ in}}{2}) / \left( \frac{1}{12 \text{ in} / \text{ft}} \right) = 1.96 \text{ k-ft} > 1.89 \text{ k-ft needed}$$

(OK)

Maximum Shear: Figure 2-7 shows end shear that is $w_u l_u / 2$ except at the end span on the interior column which sees a little more and you must design for 15% increase:

$$V_{u\text{-max}} = 1.15 w_u l_u / 2 = 1.15 \left( \frac{280.6 \text{ lb/ft}^2 \cdot \frac{1}{12 \text{ ft}}} \right) \left( \frac{1}{12 \text{ in}} \right) = 1452 \text{ lb}$$ (for a 1 ft strip)

$$V_u \text{ at } d \text{ away from the support} = V_{u\text{-max}} - w(d) = 1452 \text{ lb} - \left( \frac{280.6 \text{ lb/ft}^2 \cdot \frac{1}{12 \text{ ft}}} \right) \left( \frac{1}{12 \text{ in}} \right) = 1357 \text{ lb}$$

Check the one way shear capacity: $\phi V_c = \phi 2 \lambda \sqrt{f'_c} bd$ ($\phi = 0.75$, $\lambda = 1.0$ for normal weight concrete):

$$\phi V_c = 0.75(2)(1.0)\sqrt{5000 \psi i(12'')(4.06''}) = 4003 \text{ lb}$$

Is $V_u$ (needed) < $\phi V_c$ (capacity)? YES: 1357 lb ≤ 4003 lb, so we don't need to make the slab thicker.

Interior Beam (effectively a T-beam):

Tributary width = 10 ft for an interior beam.

$$\text{dead load} = (100.5 \text{ lb/ft}^2)(10 \text{ ft}) = 1005 \text{ lb/ft (14.7 kN/m)}$$
Reduction of live load is allowed, with a live load element factor, $K_{LL}$, of 2 for an interior beam for its tributary width assuming the girder is 12” wide. The live load is 100 lb/ft$^2$:

$$L = L_0(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}) = 100 \text{ lb/ft}^2(0.25 + \frac{15}{\sqrt{2(30 \text{ ft} - 1 \text{ ft})/(10 \text{ ft})}}) = 87.3 \text{ lb/ft}^2$$

(Reduction Multiplier = 0.873 > 0.5)

live load = 87.3 lb/ft$^2$(10ft) = 873 lb/ft (12.7 kN/m)

Estimating a 12” wide x 24” deep beam means the additional dead load from self weight ($w = \gamma \cdot A$ in units of load/length) can be included. The top 5 inches of slab has already been included in the dead load:

dead load from self weight = $150/\gamma$ (12 in wide) (24 - 5 in deep) ($\frac{1}{12} \text{ ft}^2$) = 237.5 lb/ft (3.46 kN/m)

$w_u = 1.2(1005 \text{ lb/ft} + 237.5 \text{ lb/ft}) + 1.6(873 \text{ lb/ft}) = 2888 \text{ lb/ft} (4.30 \text{ kN/m})$

The effective width, $b_E$, of the T part is the smaller of $\frac{L}{4}$, $b_w + 16\ell$, or center-center spacing

$b_E = \text{min}\{29 \text{ ft}/4 = 7.25 \text{ ft} = 87 \text{ in}, 12 \text{ in} + 16\times5 \text{ in} = 92 \text{ in}, 10 \text{ ft} = 120 \text{ in}\} = 87 \text{ in}$

The clear span for the beam is

$l_n = 30 \text{ ft} - 1 \text{ ft} = 29 \text{ ft}$

**Maximum Positive Moments** from Figure 2-3, end span (integral with support):

$$M_u (\text{positive}) = \frac{w_u \ell^2 n}{14} = \frac{2888 \text{ lb/ft}^2 (29 \text{ ft})^2}{14} \frac{1 \text{k}}{1000 \text{ lb}} = 173.5 \text{ k-ft}$$

**Maximum Negative Moments** from Figure 2-4, end span (integral with support):

$$M_u (\text{negative}) = \frac{w_u \ell^2 n}{10} = \frac{2888 \text{ lb/ft}^2 (29 \text{ ft})^2}{10} \frac{1 \text{k}}{1000 \text{ lb}} = 242.9 \text{ k-ft}$$
Figure 3.8.1 can be used to find an approximate $\rho$ for top reinforcement if $R_n = M_n / bd^2$ and we set $M_n = M_u / \phi$. We can presume the effective depth is 2.5" less than the 24" depth (for 1.5" cover and 1/2 bar diameter for a #10 (10/8") bar + #3 stirrups (3/8" more)), so $d = 21.5"$.

$$R_n = \frac{1000^{\frac{3}{2}} \cdot 242.9^{k-f} \cdot 0.9 \cdot (12^{\text{in}} / 21.5^{\text{in}})^2}{12^{\text{in}} / 2} = 584 \text{ psi}$$

so $\rho$ for $f'_c = 3000 \text{ psi}$ and $f_y = 60,000 \text{ psi}$ is about 0.011 (and less than $\rho_{\text{max}} = 0.0135$)

Then we pick bars and spacing off Table 3-7 to fit in the effective flange width in the slab.

For bottom reinforcement (positive moment) the effective flange is so wide at 87 in, that it resists a lot of compression, and needs very little steel to stay under-reinforced ($a$ is between 0.6" and 0.5"). We’d put in bottom bars at the minimum reinforcement allowed and for tying the stirrups to.

**Maximum Shear:** $V_{\text{max}} = w_u l/2$ normally, but the end span sees a little more and you must design for 15% increase. But for beams, we can use the lower value of $V$ that is a distance of $d$ from the face of the support

$$V_{\text{u-design}} = 1.15 w_u l/2 - w_u d = \frac{1.15(2888^{\text{lb/ft}})(29^{\text{in}})}{2} - \frac{2888^{\text{lb/ft}}(21.5^{\text{in}})}{12^{\text{in}} / 2} = 42,983 \text{ lb} = 43.0 \text{ k}$$

Check the one way shear capacity = $\phi v V_c = \phi v 2 \lambda \sqrt{f'_c} bd$, where $\phi v = 0.75$ and $\lambda = 1.0$

$$\phi v V_c = 0.75(2)(1.0)\sqrt{3000 \text{ psi}}(12^{\text{in}})(21.5^{\text{in}}) = 21,197 \text{ lb} = 21.2 \text{ k}$$

Is $V_u$ (needed) < $\phi v V_c$ (capacity)?

NO: 43.0 k is greater than 21.2 k, so stirrups are needed

$$\phi v V_s = V_u - \phi v V_c = 43.0 \text{ k} - 21.2 \text{ k} = 21.8 \text{ k} \text{ (max needed)}$$

Using #3 bars (typical) with two legs means $A_v = 2(0.11^{\text{in}^2}) = 0.22^{\text{in}^2}$.

To determine required spacing, use Table 3-8. For $d = 21.5"$ and $\phi v V_s \leq \phi v 4 \lambda \sqrt{f'_c} bd$ (where $\phi v 4 \lambda \sqrt{f'_c} bd = 2 \phi v V_c = 2(21.2 \text{ k}) = 42.4 \text{ k}$), the maximum spacing is $d/2 = 10.75 \text{ in}$. or 24”.

$$s_{\text{required}} = \frac{\phi v A_v f'_c d}{V_s - \phi v V_c} = \frac{\phi v A_v f'_c d}{\phi v V_c} = \frac{0.75 \cdot 0.22^{\text{in}} \cdot 60^{\text{ksi}} \cdot 21.5^{\text{in}}}{21.8} = 9.75 \text{ in}, \text{ so use 9 in}.$$
The required spacing where stirrups are needed for crack control \((\phi V_c \geq V_u > \frac{1}{2} \phi V_c)\) is
\[
s_{\text{required}} = \frac{A_{f_m}}{50b_w} = \frac{0.22\text{in}^2(60,000\text{psi})}{50(12\text{ in})} = 22\text{ in}
\]
and the maximum spacing is \(d/2 = 10.75\text{ in. or 24". Use 10 in.}\).

A recommended minimum spacing for the first stirrup is 2 in. from the face of the support. A distance of one half the spacing near the support is often used.

**Spandrel Girders:**

Because there is a concentrated load on the girder, the approximate analysis can’t technically be used. If we converted the maximum moment (at midspan) to an equivalent distributed load by setting it equal to \(w_ud^2/8\) we would then use:
Maximum Positive Moments from Figure 2-3, end span (integral with support):

\[ M_{u+} = \frac{w_u \ell_n^2}{14} \]

Maximum Negative Moments from Figure 2-4, end span (column support):

\[ M_{u-} = \frac{w_u \ell_n^2}{10} \text{ (with } \frac{w_u \ell_n^2}{16} \text{ at end)} \]

Column:

An exterior or corner column will see axial load and bending moment. We’d use interaction charts for \( P_u \) and \( M_u \) for standard sizes to determine the required area of steel. An interior column sees very little bending. The axial loads come from gravity. The factored load combination is 1.2D + 1.6L + 0.5L_r.

The girder weight, presuming 1’ x 4’ girder at 150 lb/ft^3 = 600 lb/ft

Top story: presuming 20 lb/ft^2 roof live load, the total load for an interior column (tributary area of 30’x30’) is:

\[ \begin{align*}
\text{DL}_{\text{roof}} & : \ 1.2 \times 100.5 \text{ lb/ft}^2 \times 30 \text{ ft} \times 30 \text{ ft} = 108.5 \text{ k} \\
* \text{assuming the same dead load and materials as the floors} \\
\text{DL}_{\text{beam}} & : \ 1.2 \times 237.5 \text{ lb/ft} \times 30 \text{ ft} \times 3 \text{ beams} = 25.6 \text{ k} \\
\text{DL}_{\text{girder}} & : \ 1.2 \times 600 \text{ lb/ft} \times 30 \text{ ft} = 21.6 \text{ k} \\
\text{LL}_r & : \ 0.5 \times 20 \text{ lb/ft}^2 \times 30 \text{ ft} \times 30 \text{ ft} = 9.0 \text{ k} \\
\text{Total} & = 164.7 \text{ k}
\end{align*} \]

Lower stories:

\[ \begin{align*}
\text{DL}_{\text{floor}} & : \ 1.2 \times 100.5 \text{ lb/ft}^2 \times 30 \text{ ft} \times 30 \text{ ft} = 108.5 \text{ k} \\
\text{DL}_{\text{beam}} & : \ 1.2 \times 237.5 \text{ lb/ft} \times 30 \text{ ft} \times 3 \text{ beams} = 25.6 \text{ k} \\
\text{DL}_{\text{girder}} & : \ 1.2 \times 600 \text{ lb/ft} \times 30 \text{ ft} = 21.6 \text{ k} \\
\text{LL}_{\text{floor}} & : \ 1.6 \times (0.873) \times 100 \text{ lb/ft}^2 \times 30 \text{ ft} \times 30 \text{ ft} = 125.7 \text{ k} \quad \text{(includes reduction)} \\
\text{Total} & = 281.4 \text{ k}
\end{align*} \]

2nd floor column sees \( P_u = 164.7+281.4 = 446.1 \text{ k} \)

1st floor column sees \( P_u = 446.1+281.4 = 727.5 \text{ k} \)

Look at the example interaction diagram for an 18” x 18” column (Figure 5-20 – ACI 318-02) using \( f'_c = 4000 \text{ psi} \) and \( f_y = 60,000 \text{ psi} \) for the first floor having \( P_u = 727.5 \text{ k} \), and \( M_u \) to the column being approximately 10% of the beam negative moment = 0.1*242.9 k-ft = 24.3 k-ft: (See maximum negative moment calculation for an interior beam.) The chart indicates the capacity for the reinforcement amounts shown by the solid lines.

For \( P_u = 727.5 \text{ k} \) and \( M_u = 24.3 \text{ k-ft} \), the point plots below the line marked 4-#11 (1.93% area of steel to an 18 in x 18 in area).
Lateral Force Design:

The wind loads from the wind speed, elevation, and exposure we’ll accept as shown in Figure 17.42 given on the left in psf. The wind is acting on the long side of the building. The perimeter frame resists the lateral loads, so there are two with a tributary width of \( \frac{1}{2} [(30\text{ft}) \times (4 \text{ bays}) + 2\text{ ft for beam widths and cladding}] = \frac{122\text{ft}}{2} = 61\text{ft} \)

The factored combinations with dead and wind load are:
- \(1.2D + 1.6L + 0.5W\)
- \(1.2D + 1.0W + L + 0.5L_r\)

The tributary height for each floor is half the distance to the next floor (top and bottom):

**FIGURE 17.42** Building Five: How wind loads affect the lateral bracing system.
Exterior frame (bent) loads:

$$H_1 = 195^\frac{h}{l/\beta} (61^\beta) = 11,895 \text{ lb} = 11.9 \text{ k/bent}$$

$$H_2 = \frac{234^\frac{h}{l/\beta} (61^\beta)}{1000^\frac{h}{l/\beta}} = 14.3 \text{ k/bent}$$

$$H_3 = \frac{227^\frac{h}{l/\beta} (61^\beta)}{1000^\frac{h}{l/\beta}} = 13.8 \text{ k/bent}$$

Using Multiframe, the axial force, shear and bending moment diagrams can be determined using the load combinations, and the largest moments, shear and axial forces for each member determined.

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(This is the summary diagram of force, shear and moment magnitudes refer to the maximum values in the column or beams, with the maximum moment in the beams being negative over the supports, and the maximum moment in the columns being at an end.)

**Axial force diagram:**

![Axial force diagram]

**Shear diagram:**

![Shear diagram]

**Bending moment diagram:**

![Bending moment diagram]

**Displacement:**

![Displacement]
Beam-Column loads for design:

The bottom exterior columns see the largest bending moment on the lee-ward side (left):

\[ P_u = 192.8 \text{ k and } M_u = 165.1 \text{ k-ft} \] (with large axial load)

The interior columns see the largest axial forces:

\[ P_u = 341.7 \text{ k and } M_u = 95.5 \text{ k-ft} \] and \[ P_u = 340.6 \text{ k and } M_u = 104.4 \text{ k-ft} \]

Refer to an interaction diagram for column reinforcement and sizing.

Case 2

Slab:

The slabs are effectively 30 ft x 30 ft, making them two-way slabs. Minimum thicknesses (by ACI) are a function of the span. Using Table 4-1 for two way slabs, the minimum is the larger of \( l_n/36 \) or 4 inches. Presuming the columns are 18” wide, \( l_n = 30 \text{ ft} - (18 \text{ in})/(12 \text{ ft/in}) = 28.5 \text{ ft} \),

\[ h = l_n/36 = (28.5 \times 12)/36 = 9.5 \text{ in} \]

The table also says the drop panel needs to be \( l/3 \) long = 28.5 ft/3 = 9.5 ft, and that the minimum depth must be \( 1.25h = 1.25(9.5 \text{ in}) = 12 \text{ in} \).

For the strips, \( l_2 = 30 \text{ ft} \), so the interior column strip will be \( 30 \text{ ft}/4 + 30 \text{ ft}/4 = 15 \text{ ft} \), and the middle strip will be the remaining 15 ft.

\[ \text{dead load from slab} = \frac{150 \text{ lb/ft}^3 \times 9.5 \text{ in}}{12 \text{ in}^2} = 118.75 \text{ lb/ft}^2 \]

total dead load = 5 + 15 + 118.75 lb/ft\(^2\) + 2” of lightweight concrete topping @ 18 lb/ft\(^2\) (0.68 KPa)

(presuming interior wall weight is over beams & girders)
total dead load = 156.75 lb/ft²
live load with reduction, where live load element factor, $K_{LL}$, is 1 for a two way slab:

$L = L_0(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}) = 100 \frac{lb}{ft^2}(0.25 + \frac{15}{\sqrt{1(30 ft)(30 ft)}}) = 75$ lb/ft²

total factored distributed load:

$w_u = 1.2(156.75 \text{ lb/ft}^2) + 1.6(75 \text{ lb/ft}^2) = 308.1 \text{ lb/ft}^2$

total panel moment to distribute:

$M_o = \frac{w_u l_4^2 l_m^2}{8} = \frac{308.1 lb/ft^2 (30 ft)(28.5 ft)^2}{8} \cdot \frac{1k}{1000 lb} = 938.4$ k-ft

Column strip, end span:

Maximum Positive Moments from Table 4-3, (flat slab with spandrel beams):

$M_{u+} = 0.30M_o = 0.30 \cdot (938.4 \text{ k-ft}) = 281.5$ k-ft

Maximum Negative Moments from Table 4-3, (flat slab with spandrel beams):

$M_{u-} = 0.53M_o = 0.53 \cdot (938.4 \text{ k-ft}) = 497.4$ k-ft

Middle strip, end span:

Maximum Positive Moments from Table 4-3, (flat slab with spandrel beams):

$M_{u+} = 0.20M_o = 0.20 \cdot (938.4 \text{ k-ft}) = 187.9$ k-ft

Maximum Negative Moments from Table 4-3, (flat slab with spandrel beams):

$M_{u-} = 0.17M_o = 0.17 \cdot (938.4 \text{ k-ft}) = 159.5$ k-ft

Design as for the slab in Case 1, but provide steel in both directions distributing the reinforcing needed by strips.
Shear around columns: The shear is critical at a distance $d/2$ away from the column face. If the drop panel depth is 12 inches, the minimum $d$ with two layers of 1” diameter bars would be $12" - \frac{3}{4}"$ (cover) - (1") - $\frac{1}{2}(1")$ = about 9.75 in (to the top steel).

The shear resistance is $\phi V_c = \phi_4 \lambda \sqrt{f'_c} b_o d$, $\phi = 0.75$ and $\lambda = 1.0$ where $b_o$, is the perimeter length.

The design shear value is the distributed load over the tributary area outside the shear perimeter, $V_u = w_u (\text{tributary area - } b_1 \times b_2)$ where b’s are the column width plus $d/2$ each side.

\[ b_1 = b_2 = 18" + 9.75"/2 + 9.75"/2 = 27.75 \text{ in} \]

\[ b_1 \times b_2 = (27.75\text{in})^2 \cdot \left(\frac{1\text{ft}}{12\text{in}}\right)^2 = 5.35 \text{ ft}^2 \]

\[ V_u = (284.7\sqrt{1.5})(30^{0.6} \cdot 30^{0.6} - 4.97^{0.6}) \cdot \frac{1k}{1000lb} = 254.7 \text{ k} \]

Shear capacity:

\[ b_o = 2(b_1) + 2(b_2) = 4(27.75 \text{ in}) = 111 \text{ in} \]

\[ \phi V_c = 0.75 \cdot 4(1.0)\sqrt{3000 \text{ psi} \cdot 111" \cdot 9.75"} = 177,832 \text{ lb} = 177.8 \text{ ksi} < V_u! \]

The shear capacity is not large enough. The options are to provide shear heads or a deeper drop panel, or change concrete strength, or even a different system selection...

There also is some transfer by the moment across the column into shear.

**Deflections:**

Elastic calculations for deflections require that the steel be turned into an equivalent concrete material using $n = \frac{E_s}{E_c}$. $E_c$ can be measured or calculated with respect to concrete strength.

For normal weight concrete (150 lb/ft$^3$): $E_c = 57,000 \sqrt{f'_c}$

\[ E_c = 57,000 \sqrt{3000 \text{ psi}} = 3,122,019 \text{ psi} = 3122 \text{ ksi} \]

\[ n = 29,000 \text{ psi}/3122 \text{ ksi} = 9.3 \]
Deflection limits are given in Table 24.2.2

<table>
<thead>
<tr>
<th>Member</th>
<th>Condition</th>
<th>Deflection to be considered</th>
<th>Deflection limitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat roofs</td>
<td>Not supporting or attached to nonstructural elements likely to be damaged by large deflections</td>
<td>Immediate deflection due to maximum of L, S, and R</td>
<td>≤180[1]</td>
</tr>
<tr>
<td>Floors</td>
<td></td>
<td>Immediate deflection due to L</td>
<td>≤360</td>
</tr>
<tr>
<td>Roof or floors</td>
<td>Supporting or attached to nonstructural elements</td>
<td>That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time dependent deflection due to any additional live load[2]</td>
<td>≤480[3]</td>
</tr>
<tr>
<td></td>
<td>Likely to be damaged by large deflections</td>
<td></td>
<td>≥240[4]</td>
</tr>
<tr>
<td></td>
<td>Not likely to be damaged by large deflections</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[1] Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time-dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

[2] Time-dependent deflection shall be calculated in accordance with 24.2.4, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be calculated on basis of accepted engineering data relating to time deflection characteristics of members similar to those being considered.

[3] Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.

[4] Limit shall not exceed tolerance provided for nonstructural elements.