



Two-Way Flat Plate Concrete Floor System Analysis and Design (ACI 318-14)







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The concrete floor slab system shown below is for an intermediate floor to be designed considering partition weight = 20 psf, and unfactored live load = 40 psf. Flat plate concrete floor system does not use beams between columns or drop panels and it is usually suited for lightly loaded floors with short spans typically for residential and hotel buildings. The lateral loads are independently resisted by shear walls. The two design procedures shown in <u>ACI 318-</u><u>14</u>: Direct Design Method (DDM) and the Equivalent Frame Method (EFM) are illustrated in detail in this example. The hand solution from EFM is also used for a detailed comparison with the analysis and design results from <u>spSlab</u> engineering software program by <u>StructurePoint</u>.





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Code

Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14) Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10) International Code Council, 2012 International Building Code, Washington, D.C., 2012

References

- Notes on ACI 318-11 Building Code Requirements for Structural Concrete, Twelfth Edition, 2013 Portland Cement Association, Example 20.1
- Concrete Floor Systems (Guide to Estimating and Economizing), Second Edition, 2002 David A. Fanella
- Simplified Design of Reinforced Concrete Buildings, Fourth Edition, 2011 Mahmoud E. Kamara and Lawrence C. Novak
- spSlab Engineering Software Program Manual v10.00, STRUCTUREPOINT, 2024
- Contact <u>Support@StructurePoint.org</u> to obtain supplementary materials (<u>spSlab</u> model: DE-Two-Way-Flat-Plate-ACI-318-14.slbx)

Design Data

Floor-to-Floor Height = 9 ft (provided by architectural drawings)

Superimposed Dead Load, SDL = 20 psf for framed partitions, wood studs plaster 2 sides

ASCE/SEI 7-10 (Table C3-1)

Live Load, LL = 40 psf for Residential floors

ASCE/SEI 7-10 (Table 4-1)

 f_c ' = 4,000 psi (for slabs)

 f_c ' = 6,000 psi (for columns)

 $f_y = 60,000 \text{ psi}$

Required fire resistance rating = 2 hours



1. Notations

This section (based on ACI 318-14 provisions) defines notation and terminology used in this design example:

- a = depth of equivalent rectangular stress block, in.
- A_b = area of an individual bar or wire, in.²
- A_g = gross area of concrete section, in.² For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s)
- A_s = area of nonprestressed longitudinal tension reinforcement, in.²
- A_{st} = total area of nonprestressed longitudinal reinforcement including bars or steel shapes, and excluding prestressing reinforcement, in.²
- $A_{s,min}$ = minimum area of flexural reinforcement, in.²
- b = width of compression face of member, in.
- b_o = perimeter of critical section for two-way shear in slabs and footings, in.
- b_w = web width or diameter of circular section, in.
- b_1 = dimension of the critical section b_o measured in the direction of the span for which moments are determined, in.
- b_2 = dimension of the critical section b_o measured in the direction perpendicular to b_1 , in.
- c = distance from extreme compression fiber to neutral axis, in.
- c_c = clear cover of reinforcement, in.
- c_1 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.
- c_2 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1 , in.
- C = cross-sectional constant to define torsional properties of slab and beam
- d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.
- d_b = nominal diameter of bar, wire, or prestressing strand, in.
- E_c = modulus of elasticity of concrete, psi
- E_{cs} = modulus of elasticity of slab concrete, psi
- E_s = modulus of elasticity of reinforcement and structural steel, excluding prestressing reinforcement, psi
- f_c' = specified compressive strength of concrete, psi

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- f_r = modulus of rupture of concrete, psi
- f_y = specified yield strength for nonprestressed reinforcement, psi
- h = overall thickness, height, or depth of member, in.
- I_{cr} = moment of inertia of cracked section transformed to concrete, in.⁴
- I_e = effective moment of inertia for calculation of deflection, in.⁴
- I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.⁴
- K_t = torsional stiffness of member; moment per unit rotation
- l = span length of beam or one-way slab; clear projection of cantilever, in.
- l_n = length of clear span measured face-to-face of supports, in.
- l_1 = length of span in direction that moments are being determined, measured center-to-center of supports, in.
- l_2 = length of span in direction perpendicular to l_1 , measured center-to-center of supports, in.
- M_a = maximum moment in member due to service loads at stage deflection is calculated, in.-lb
- M_{cr} = cracking moment, in.-lb
- M_u = factored moment at section, in.-lb
- M_o = total factored static moment, in.-lb
- n = number of items, such as, bars, wires, monostrand anchorage devices, anchors, or shearhead arms
- P_n = nominal axial compressive strength of member, lb
- P_o = nominal axial strength at zero eccentricity, lb
- P_u = factored axial force; to be taken as positive for compression and negative for tension, lb
- q_{Du} = factored dead load per unit area, lb/ft²
- q_{Lu} = factored live load per unit area, lb/ft²
- q_u = factored load per unit area, lb/ft²
- v_c = stress corresponding to nominal two-way shear strength provided by concrete, psi
- v_u = maximum factored two-way shear stress calculated around the perimeter of a given critical section, psi
- V_c = nominal shear strength provided by concrete, lb
- V_n = nominal shear strength, lb
- V_s = nominal shear strength provided by shear reinforcement, lb
- V_u = factored shear force at section, lb

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- w_c = density, unit weight, of normalweight concrete or equilibrium density of lightweight concrete, lb/ft³
- y_t = distance from centroidal axis of gross section, neglecting reinforcement, to tension face, in.
- α_s = constant used to calculate V_c in slabs and footings
- β = ratio of long to short dimensions: clear spans for two-way slabs, sides of column, concentrated load or reaction area; or sides of a footing
- β_1 = factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis
- γ_f = factor used to determine the fraction of M_{sc} transferred by slab flexure at slab-column connections
- γ_{v} = factor used to determine the fraction of M_{sc} transferred by eccentricity of shear at slab-column connections
- ε_t = net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature
- λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength
- λ_{Δ} = multiplier used for additional deflection due to long-term effects
- ξ = time-dependent factor for sustained load
- $\rho' = \text{ratio of } A_s' \text{ to } bd$
- ϕ = strength reduction factor





2. Preliminary Member Sizing

2.1 Slab Minimum Thickness – Deflection

ACI 318-14 (Table 8.3.1.1)

In this example, deflection will be calculated and checked to satisfy project deflection limits. Minimum member thickness and depths from ACI 318-14 will be used for preliminary sizing.

Using ACI 318-14 minimum slab thickness for two-way construction without interior beams in Table 8.3.1.1.

Exterior Panels:
$$h = \frac{l_n}{30} = \frac{200}{30} = 6.67$$
 in.

But not less than 5 in.

<u>ACI 318-14 (8.3.1.1(a))</u>

ACI 318-14 (Table 8.3.1.1)

Interior Panels: $h = \frac{l_n}{33} = \frac{200}{33} = 6.06$ in.

But not less than 5 in.

ACI 318-14 (8.3.1.1(a))

Where $l_n = \text{length of clear span in the long direction} = 216.00 - 16.00 = 200.00 \text{ in.}$

Try 7 in. slab for all panels (self-weight = 87.5 psf)

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2.2. Slab Shear Strength – One Way Shear

Evaluate the average effective depth (Figure 2):

$$d_l = t_{slab} - c_{clear} - d_b - \frac{d_b}{2} = 7 - 0.75 - 0.50 - \frac{0.50}{2} = 5.50$$
 in

$$d_t = t_{slab} - c_{clear} - \frac{d_b}{2} = 7 - 0.75 - \frac{0.50}{2} = 6.00$$
 in

$$d_{avg} = \frac{d_l + d_t}{2} = \frac{5.50 + 6.00}{2} = 5.75$$
 in

Where:

$$c_{clear} = 3/4$$
 in. for # 4 steel bar

ACI 318-14 (Table 20.6.1.3.1)

 $d_b = 0.50$ in. for # 4 steel bar



Figure 2 - Average Effective Depth for Flat Plate

 Factored dead load,
 $q_{Du} = 1.20 \times (87.5 + 20) = 129.00 \text{ psf}$

 Factored live load,
 $q_{Lu} = 1.60 \times 40 = 64.00 \text{ psf}$

 ACI 318-14 (5.3.1)

 Total factored load,
 $q_u = 193.00 \text{ psf}$

Check the adequacy of slab thickness for beam action (one-way shear)

ACI 318-14 (22.5)



At an interior column:

Consider a 12-in. wide strip. The critical section for one-way shear is located at a distance d, from the face of support (see Figure 3).

Tributary area for one-way shear is:

$$A_{Tributary} = \left[\left(\frac{18}{2} \right) - \left(\frac{16}{2 \times 12} \right) - \left(\frac{5.75}{12} \right) \right] = 7.85 \text{ ft}^2$$
$$V_u = q_u \times A_{Tributary} = 0.193 \times 7.85 = 1.52 \text{ kips}$$
$$V_c = 2 \times \lambda \times \sqrt{f_c'} \times b_w \times d$$

ACI 318-14 (Eq. 22.5.5.1)

where $\lambda = 1$ for normal weight concrete

$$\phi V_c = 0.75 \times 2 \times 1.0 \times \sqrt{4,000} \times 12 \times \frac{5.75}{1,000} = 6.55 \text{ kips} > V_u = 1.52 \text{ kips}$$

Slab thickness of 7 in. is adequate for one-way shear.



Figure 3 - Critical Section for One-Way Shear



2.3. Slab Shear Strength – Two Way Shear

Check the adequacy of slab thickness for punching shear (two-way shear) at an interior column (Figure 4):

Tributary area for two-way shear is:

$$A_{Tributary} = (18 \times 14) - \left(\frac{16 + 5.75}{12}\right)^2 = 248.71 \text{ ft}^2$$

 $V_u = q_u \times A_{Tributary} = 0.193 \times 248.71 = 48.00$ kips

 $V_c = 4 \times \lambda \times \sqrt{f'_c} \times b_o \times d$ (For square interior column)

ACI 318-14 (Table 22.6.5.2(a))

$$V_c = 4 \times 1.0 \times \sqrt{4,000} \times (4 \times 21.75) \times \frac{5.75}{1,000} = 126.55$$
 kips

$$\phi V_c = 0.75 \times 126.55 = 94.92$$
 kips $> V_u = 48.00$ kips

Slab thickness of 7 in. is adequate for two-way shear.



Figure 4 – Critical Section for Two-Way Shear



2.4. Column Dimensions – Axial Load

Check the adequacy of column dimensions for axial load:

Tributary area for interior column is:

 $A_{Tributary} = (18 \times 14) = 252.00 \text{ ft}^2$

 $P_u = q_u \times A_{Tributary} = 0.193 \times 252 = 48.64$ kips

 $P_n = 0.80 \times P_o = 0.80 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st})$ (For square interior column)

ACI 318-14 (22.4.2)

 $P_n = 0.80 \times (0.85 \times 6,000 \times (16 \times 16 - 0) + 0) = 1,044.48$ kips

 $\phi P_n = 0.65 \times 1,044.48 = 678.91 \text{ kips} > P_u = 48.64 \text{ kips}$

Column dimensions of 16 in. \times 16 in. are adequate for axial load.



3. Two-Way Slab Analysis and Design

ACI 318 states that a slab system shall be designed by any procedure satisfying equilibrium and geometric compatibility, provided that strength and serviceability criteria are satisfied. Distinction of two-way systems from one-way systems is given by <u>ACI 318-14 (R8.10.2.3 & R8.3.1.2)</u>.

ACI 318 permits the use of Direct Design Method (DDM) and Equivalent Frame Method (EFM) for the gravity load analysis of orthogonal frames and is applicable to flat plates, flat slabs, and slabs with beams. The following sections outline the solution per DDM, EFM, and spSlab software respectively.

3.1. Direct Design Method (DDM)

Two-way slabs satisfying the limits in <u>ACI 318-14 (8.10.2)</u> are permitted to be designed in accordance with the DDM.

3.1.1. Direct Design Method Limitations

There is a minimum of three continuous spans in each direction	<u>ACI 318-14 (8.10.2.1)</u>
Successive span lengths are equal	<u>ACI 318-14 (8.10.2.2)</u>
Long-to-short span ratio is 1.29 < 2	<u>ACI 318-14 (8.10.2.3)</u>
Columns are not offset	<u>ACI 318-14 (8.10.2.4)</u>
Loads are uniformly distributed over the entire panel	<u>ACI 318-14 (8.10.2.5)</u>
Service live-to-dead load ratio of $0.37 < 2.0$	<u>ACI 318-14 (8.10.2.6)</u>
Slab system is without beams and this requirement is not applicable	<u>ACI 318-14 (8.10.2.7)</u>

Since all the criteria are met, Direct Design Method can be utilized.

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3.1.2. Design Moments

a) Calculate the total factored static moment:

$$M_o = \frac{q_u \times \ell_2 \times \ell_n^2}{8} = \frac{0.193 \times 14 \times 16.67^2}{8} = 93.82 \text{ ft-kips}$$
 ACI 318-14 (8.10.3.2)

b) Distribute the total factored moment, M_o , in an interior and end span:

ACI 318-14 (8.10.4)

Table 1 – Distribution of M_o along the span							
I	ocation	Total Design Strip Moment, M _{DS} (ft-kips)					
	Exterior Negative	$0.26 \times M_o = 24.39$					
Exterior Span	Positive	$0.52 \times M_o = 48.79$					
	Interior Negative	$0.70 \times M_o = 65.67$					
Interior Span	Positive	$0.35 \times M_o = 32.84$					

c) Calculate the column strip moments.

ACI 318-14 (8.10.5)

That portion of negative and positive total design strip moments not resisted by column strips shall be proportionally assigned to corresponding two half-middle strips. <u>ACI 318-14 (8.10.6.1)</u>

Table 2 – Lateral Distribution of the Total Design Strip Moment, MDS									
Location		Total Design Strip Moment, M _{DS} (ft-kips)	Column Strip Moment, (ft-kips)	Moment in Two Half Middle Strips, (ft-kips)					
	Exterior Negative [*]	24.39	$1.00 \times M_{DS} = 24.39$	$0.00\times M_{DS}=0.00$					
Exterior Span	Positive	48.79	$0.60 \times M_{DS} = 29.27$	$0.40 \times M_{DS} = 19.52$					
	Interior Negative [*]	65.67	$0.75 \times M_{DS} = 49.25$	$0.25 \times M_{DS} = 16.42$					
Interior Span	Positive	32.84	$0.60 \times M_{DS} = 19.70$	$0.40 \times M_{DS} = 13.14$					
* All negative moments are at face of support.									

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3.1.3. Flexural Reinforcement Requirements

a) Determine flexural reinforcement required for column and middle strips at all critical sections.

The following calculation is for the exterior span exterior negative location of the column strip.

 $M_u = 24.39$ ft-kips

Use average $d_{avg} = 5.75$ in.

To determine the area of steel, assumptions have to be made whether the section is tension or compression controlled, and regarding the distance between the resultant compression and tension forces along the slab section (*jd*). In this example, tension-controlled section will be assumed so the reduction factor ϕ is equal to 0.9, and *jd* will be taken equal to 0.95*d*. The assumptions will be verified once the area of steel is finalized.

Assume $jd = 0.95 \times d = 5.46$ in.

Column strip width, $b = \frac{14 \times 12}{2} = 84$ in.

Middle strip width, $b = \frac{14 \times 12}{2} = 84$ in.

$$A_s = \frac{M_u}{\phi \times f_v \times jd} = \frac{24.39 \times 12,000}{0.90 \times 60,000 \times 0.95 \times 5.46} = 0.99 \text{ in.}^2$$

Recalculate 'a' for the actual $A_s = 0.99$ in.²:

$$a = \frac{A_s \times f_y}{0.85 \times f_c' \times b} = \frac{0.99 \times 60,000}{0.85 \times 4,000 \times 84} = 0.208 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{0.208}{0.85} = 0.245 \text{ in.}$$

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{0.245}\right) \times 5.75 - 0.003 = 0.067 > 0.005$$

Therefore, the assumption that section is tension-controlled is valid.

$$A_{s} = \frac{M_{u}}{\phi \times f_{y} \times \left(d - \frac{a}{2}\right)} = \frac{24.39 \times 12,000}{0.90 \times 60,000 \times \left(5.75 - \frac{0.208}{2}\right)} = 0.96 \text{ in.}^{2}$$



$A_{s,min} = 0.018 \times 84 \times 7 = 1.06 \text{ in.}^2 > 0.96 \text{ in.}^2$

Maximum spacing $s_{max} = 2 \times h = 14$ in. < 18 in.

Provide 6 - #4 bars with $A_s = 1.20$ in.² and $s = \frac{84}{6} = 14$ in. $\leq s_{\text{max}}$

Based on the procedure outlined above, values for all span locations are given in Table 3.

Table 3 – Required Slab Reinforcement for Flexure (DDM)											
S	pan Location	M _u (ft-kips)	<i>b</i> (in.)	d (in.)	A _{s,req} (in. ²)	$A_{s,min}$ (in. ²)	Reinforcement Provided	A _{s,provided} (in. ²)			
	End Span										
	Exterior Negative	24.39	84	5.75	0.96	1.06	6-#4	1.20			
Column Strip	Positive	29.27	84	5.75	1.16	1.06	6 – #4	1.20			
	Interior Negative	49.25	84	5.75	1.98	1.06	10-#4	2.00			
	Exterior Negative	0.00	84	5.75	0.00	1.06	6 – #4	1.20			
Middle Strip	Positive	19.52	84	5.75	0.77	1.06	6 – #4	1.20			
	Interior Negative	16.42	84	5.75	0.64	1.06	6 – #4	1.20			
	Interior Span										
Column Strip Positive 19.70 84 5.75 0.77				0.77	1.06	6-#4	1.20				
Middle Strip	Positive	13.14	84	5.75	0.51	1.06	6-#4	1.20			

<u>ACI 318-14 (24.4.3.2)</u>

ACI 318-14 (8.7.2.2)

b) Calculate additional slab reinforcement at columns for moment transfer between slab and column

The factored slab moment resisted by the column ($\gamma_f \times M_u$) shall be assumed to be transferred by flexure. Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist this moment. The fraction of slab moment not calculated to be resisted by flexure shall be assumed to be resisted by eccentricity of shear. ACI 318-14 (8.4.2.3)

Portion of the unbalanced moment transferred by flexure is $\gamma_f \times M_u$ ACI 318-14 (8.4.2.3.1)

Where:

$$\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{b_1}{b_2}}}$$
ACI 318-14 (8.4.2.3.2)

- b_1 = Dimension of the critical section b_0 measured in the direction of the span for which moments are determined in ACI 318, Chapter 8 (see Figure 5).
- b_2 = Dimension of the critical section b_o measured in the direction perpendicular to b_1 in ACI 318, Chapter 8 (see Figure 5).
- b_b = Effective slab width = $c_2 + 3 \times h$

c_{CD}

 c_{AB}

Z

c₂ b_2 B

c2

 b_2

 b_1

D

w Critical shear perimeter for interior column

 c_{CD}

zc_{AB}

Edge of slab

 b_2

w



w

Figure 5 – Critical Shear Perimeters for Columns

Edge of slab c_{CD} C₁ b_1 7 Z c_{AB} c₂ \mathbf{B} Ŵ

Critical shear perimeter for exterior column



D

R

 b_1

Ζ

C



ACI 318-14 (8.4.2.3.3)



The additional reinforcement calculation for the column strip of end span – exterior negative location is provided below.

$$\begin{aligned} M_u &= 24.39 \text{ ft-kips} \\ A_{s,provided} &= 1.20 \text{ in.}^2 \\ b_1 &= c_1 + \frac{d}{2} = 16 + \frac{5.75}{2} = 18.88 \text{ in.} \\ b_2 &= c_2 + d = 16 + 5.75 = 22.75 \text{ in.} \\ b_b &= c_2 + 3 \times h = 16 + 3 \times 7 = 37 \text{ in.} \\ \gamma_f &= \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{18.88}{22.75}}} = 0.62 \\ A_s &= \frac{0.85 \times f_c' \times b_b}{f_y} \times \left(d - \sqrt{d^2 - \frac{2 \times \gamma_f \times M_u}{\phi \times 0.85 \times f_c' \times b_b}} \right) \\ A_s &= \frac{0.85 \times 4,000 \times 37}{60,000} \times \left(5.75 - \sqrt{5.75^2 - \frac{2 \times 0.62 \times 24.39}{0.9 \times 0.85 \times 4,000 \times 37}} \right) = 0.60 \text{ in.}^2 \\ A_{s,provided \text{ within } bb} &= A_{s,provided} \times \frac{b_b}{b} = 1.20 \times \frac{37}{84} = 0.53 \text{ in.}^2 \end{aligned}$$

Where b = 84 in. is the column strip width

$$A_s - A_{s, provided within bb} = 0.60 - 0.53 = 0.07 \text{ in.}^2$$

Provide 1-#4 bars with $A_s = 0.20$ in.²

Table 4 – Additional Slab Reinforcement required for moment transfer between slab and column (DDM)										
Span Location		Mu [*] (ft-kips)	γf	$\gamma_f M_u$ (ft-kips)	<i>b</i> _b (in.)	d (in.)	As req'd within bb (in ²)	A_s prov. For flexure within b_b (in ²)	Add'l Reinf.	
End Span										
Column	Exterior Negative	24.39	0.62	15.10	37	5.75	0.60	0.53	1 – #4	
Strip	Interior Negative	0.00	0.60	0.00	37	5.75	0.00	0.97	-	
* M _u is tal	* M _u is taken at the centerline of the support in Equivalent Frame Method solution.									



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3.1.4. Factored Moments in Columns

a) Interior columns:

$$M_{u} = 0.07 \times \left[(q_{Du} + 0.5 \times q_{Lu}) \times \ell_{2} \times \ell_{n}^{2} - q_{Du}' \times \ell_{2}' \times \left(\ell_{n}'\right)^{2} \right]$$

$$\underline{ACI 318-14 (8.10.7.2)}$$

$$M_{u} = 0.07 \times \left((129 + 0.5 \times 64) \times 14 \times \left(18 - \frac{16}{12}\right)^{2} - 129 \times 18 \times \left(14 - \frac{16}{12}\right)^{2} \right) = 17.75 \text{ ft-kips}$$

With the same column size and length above and below the slab,

$$M_{column} = \frac{17.75}{2} = 8.87$$
 ft-kips

b) Exterior Columns:

Total exterior negative moment from slab must be transferred directly to the column: $M_u = 24.39$ ft-kips. With the same column size and length above and below the slab,

$$M_{column} = \frac{24.39}{2} = 12.20$$
 ft-kips

The moments determined above are combined with the factored axial loads (for each story) for design of column sections as shown later in this example.



3.2. Equivalent Frame Method (EFM)

EFM is the most comprehensive and detailed procedure provided by the ACI 318 for the analysis and design of two-way slab systems where the structure is modeled by a series of equivalent frames (interior and exterior) on column lines taken longitudinally and transversely through the building.

The equivalent frame consists of three parts:

- 1) Horizontal slab-beam strip, including any beams spanning in the direction of the frame. Different values of moment of inertia along the axis of slab-beams should be taken into account where the gross moment of inertia at any cross section outside of joints or column capitals shall be taken, and the moment of inertia of the slab-beam at the face of the column, bracket or capital divide by the quantity $(1-c_2/l_2)^2$ shall be assumed for the calculation of the moment of inertia of slab-beams from the center of the column to the face of the column, bracket or capital.
- 2) Columns or other vertical supporting members, extending above and below the slab. Different values of moment of inertia along the axis of columns should be taken into account where the moment of inertia of columns from top and bottom of the slab-beam at a joint shall be assumed to be infinite, and the gross cross section of the concrete is permitted to be used to determine the moment of inertia of columns at any cross section outside of joints or column capitals.
 ACI 318-14 (8.11.4)
- 3) Elements of the structure (Torsional members) that provide moment transfer between the horizontal and vertical members. These elements shall be assumed to have a constant cross section throughout their length consisting of the greatest of the following: (1) portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined, (2) portion of slab specified in (1) plus that part of the transverse beam above and below the slab for monolithic or fully composite construction, (3) the transverse beam includes that portion of slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness. ACI 318-14 (8.11.5)

3.2.1. Equivalent Frame Method Limitations

In EFM, live load shall be arranged in accordance with 6.4.3 which requires slab systems to be analyzed and designed for the most demanding set of forces established by investigating the effects of live load placed in various critical patterns.

ACI 318-14 (8.11.1.2 & 6.4.3)

Complete analysis must include representative interior and exterior equivalent frames in both the longitudinal and transverse directions of the floor <u>ACI 318-14 (8.11.2.1)</u>

Panels shall be rectangular, with a ratio of longer to shorter panel dimensions, measured center-to-center of supports, not to exceed 2. <u>ACI 318-14 (8.10.2.3)</u>





3.2.2. Frame Members of Equivalent Frame

Determine moment distribution factors and fixed-end moments for the equivalent frame members. The moment distribution procedure will be used to analyze the equivalent frame. Stiffness factors k, carry over factors COF, and fixed-end moment factors FEM for the slab-beams and column members are determined using the design aids tables at <u>Appendix 20A of PCA Notes on ACI 318-11</u>.

These calculations are shown below.

a) Flexural stiffness of slab-beams at both ends, K_{sb}

$$\frac{c_{N1}}{\ell_1} = \frac{16}{(18 \times 12)} = 0.074, \ \frac{c_{N2}}{\ell_2} = \frac{16}{(14 \times 12)} = 0.095$$

For $c_{FI} = c_{F2}$, stiffness factors, $k_{NF} = k_{FN} = 4.127$

Thus, $K_{sb} = k_{NF} \frac{E_{cs} \times I_s}{\ell_1} = 4.127 \times \frac{E_{cs} \times I_s}{\ell_1}$ <u>PCA Notes on ACI 318-11 (Table A1)</u>

 $K_{sb} = 4.127 \times \frac{3,834 \times 10^3 \times 4,802}{216} = 351,766,909$ in.-lb

Where, $I_s = \frac{\ell_2 \times h^3}{12} = \frac{168 \times 7^3}{12} = 4,802 \text{ in.}^4$

$$E_{cs} = w_c^{1.5} \times 33 \times \sqrt{f_c'} = 150^{1.5} \times 33 \times \sqrt{4,000} = 3,834 \times 10^3 \text{ psi}$$

ACI 318-14 (19.2.2.1.a)

PCA Notes on ACI 318-11 (Table A1)

PCA Notes on ACI 318-11 (Table A1)

PCA Notes on ACI 318-11 (Table A1)

Carry-over factor COF = 0.507

Fixed-end moment $FEM = 0.08429 \times w_u \times \ell_2 \times \ell_1^2$

b) Flexural stiffness of column members at both ends, K_c

Referring to *Table A7, Appendix 20A*,

$$t_a = h/2 = 7/2 = 3.50$$
 in. $t_b = h/2 = 7/2 = 3.50$ in.

H = 9 ft = 108 in. $H_c = H - t_a - t_b = 108 - 3.5 - 3.5 = 101$ in.

Then,

$$\frac{t_a}{t_b} = 1, \ \frac{H}{H_c} = 1.069$$

Thus, $k_{AB} = k_{BA} = 4.74$ by interpolation.



$$K_{c} = \frac{4.74 \times E_{cc} \times I_{c}}{\ell_{c}}$$

$$K_{c} = 4.74 \times \frac{4,696 \times 10^{3} \times 5,461}{108} = 1,125,592,936 \text{ in.-lb}$$
Where $I_{c} = \frac{c^{4}}{12} = \frac{(16)^{4}}{12} = 5,461 \text{ in.}^{4}$

$$E_{cs} = w_{c}^{1.5} \times 33 \times \sqrt{f_{c}'} = 150^{1.5} \times 33 \times \sqrt{6,000} = 4,696 \times 10^{3} \text{ psi}$$

$$\frac{ACI 318-14 (19.2.2.1.a)}{\ell_{c}}$$

$$\ell_{c} = 9 \text{ ft} = 108 \text{ in.}$$

c) Torsional stiffness of torsional members, K_t .

$$K_{t} = \frac{9 \times E_{cs} \times C}{\left[\ell_{2} \times \left(1 - \frac{c_{2}}{\ell_{2}}\right)^{3}\right]}$$

$$K_{t} = \frac{9 \times 3,834 \times 10^{3} \times 1,325}{168 \times \left(1 - \frac{16}{14 \times 12}\right)^{3}} = 367,484,240 \text{ in.-lb}$$
Where $C = \sum \left(1 - 0.63 \times \frac{x}{y}\right) \times \left(\frac{x^{3} \times y}{3}\right)$

$$C = \left(1 - 0.63 \times \frac{7}{16}\right) \times \left(\frac{7^{3} \times 16}{3}\right) = 1,325 \text{ in.}^{4}$$

$$c_{2} = 16 \text{ in., and } \ell_{2} = 14 \text{ ft} = 168 \text{ in.}$$



Figure 6 – Torsional Member



d) Equivalent column stiffness K_{ec} .

$$K_{ec} = \frac{\sum K_c \times \sum K_t}{\sum K_c + \sum K_t}$$
$$K_{ec} = \frac{(2 \times 1,125.59) \times (2 \times 367.48)}{[(2 \times 1,125.59) + (2 \times 367.48)]} \times 10^6 = 554,074,058 \text{ in.-lb}$$

Where ΣK_t is for two torsional members one on each side of the column, and ΣK_c is for the upper and lower columns at the slab-beam joint of an intermediate floor.





e) Slab-beam joint distribution factors, DF.

At exterior joint

At interior joint

$$DF = \frac{351.77}{(351.77 + 554.07)} = 0.388$$

 $DF = \frac{351.77}{(351.77 + 351.77 + 554.07)} = 0.280$

COF for slab-beam = 0.507





3.2.3. Equivalent Frame Analysis

Determine negative and positive moments for the slab-beams using the moment distribution method. Since the unfactored live load does not exceed three-quarters of the unfactored dead load, design moments are assumed to occur at all critical sections with full factored live on all spans. <u>ACI 318-14 (6.4.3.2)</u>

$$\frac{L}{D} = \frac{40}{(87.5 + 20)} = 0.37 < \frac{3}{4}$$

a) Factored load and Fixed-End Moments (FEM's).

Factored dead load, $q_{Du} = 1.20 \times (87.5 + 20) = 129 \text{ psf}$ Factored live load, $q_{Lu} = 1.60 \times 40 = 64 \text{ psf}$ Total factored load, $q_u = 193 \text{ psf}$ *FEM*'s for slab-beams $FEM = m_{NF} \times q_u \times \ell_2 \times \ell_1^2$ *PCA Notes on ACI 318-11 (Table A1)*

- $FEM = 0.08429 \times 0.193 \times 14 \times 18^2 = 73.79$ ft-kips
- b) Moment distribution. Computations are shown in <u>Table 5</u>. Counterclockwise rotational moments acting on the member ends are taken as positive. Maximum positive span moments are determined from the following equation:

$$M_{max}^{+} = \frac{(q_u \times \ell_2) \times \ell_1^2}{8} - \frac{M_L^{-} + M_R^{-}}{2} + \frac{(M_L^{-} - M_R^{-})^2}{2 \times (q_u \times \ell_2) \times \ell_1^2} \text{ at distance } x_{max} = \frac{\ell_1}{2} + \frac{M_L^{-} - M_R^{-}}{(q_u \times \ell_2) \times \ell_1}$$

Where:

- M_{max}^+ = Maximum positive moment in the span
- M_L^- = Negative moment in the left support
- M_{R} = Negative moment in the right support
- ℓ_1 = The span length

The reactions (shear forces) at supports are given by the following equations:

$$V_{L} = \frac{(q_{u} \times \ell_{2}) \times \ell_{1}}{2} + \frac{M_{L}^{-} - M_{R}^{-}}{\ell_{1}} \qquad \qquad V_{R} = \frac{(q_{u} \times \ell_{2}) \times \ell_{1}}{2} - \frac{M_{L}^{-} - M_{R}^{-}}{\ell_{1}}$$



Where:

- V_L = Reaction (shear force) at the left support
- V_R = Reaction (shear force) at the right support

Maximum positive moment in spans 1-2 and 3-4:

$$M_{max}^{+} = \frac{(0.193 \times 14) \times 18^2}{8} - \frac{46.65 + 83.91}{2} + \frac{(46.65 - 83.91)^2}{2 \times (0.193 \times 14) \times 18^2} = 44.94 \text{ ft-kip}$$

$$x_{max} = \frac{18}{2} + \frac{(46.65 - 83.91)}{(0.193 \times 14) \times 18} = 8.23 \text{ ft}$$

$$V_L = \frac{2.70 \times 18}{2} + \frac{(46.65 - 83.91)}{18} = 22.25 \text{ kips}$$

$$V_{R} = \frac{2.70 \times 18}{2} - \frac{(46.65 - 83.91)}{18} = 26.39$$
 kips

Where:

$$M_L^- = 46.65 \text{ ft-kip}$$
 $M_R^- = 83.91 \text{ ft-kip}$

Maximum positive moment in span 2-3:

$$M_{max}^{+} = \frac{(0.193 \times 14) \times 18^{2}}{8} - \frac{76.21 + 76.21}{2} + \frac{(76.21 - 76.21)}{2 \times (0.193 \times 14) \times 18^{2}} = 33.23 \text{ ft-kip}$$

$$x_{max} = \frac{18}{2} + \frac{(76.21 - 76.21)}{(0.193 \times 14) \times 18} = 9.00 \text{ ft}$$

$$V_{L} = \frac{2.70 \times 18}{2} + \frac{(76.21 - 76.21)}{18} = 24.32 \text{ kips}$$

$$V_{R} = \frac{2.70 \times 18}{2} - \frac{(76.21 - 76.21)}{18} = 24.32 \text{ kips}$$

Where:

$$M_L^- = 76.21 \text{ ft-kip}$$
 $M_R^- = 76.21 \text{ ft-kip}$



Table 5 – Moment distribution for equivalent frame method (EFM)										
(+		<i>m</i>	<u></u>	<i>'''''</i>		<i>11111</i>				
~	- x				→ x					
	utur	~	ttm.	utu		utur				
Joint	1	2		3		4				
Member	1-2	2-1	2-3	3-2	3-4	4-3				
DF	0.388	0.280	0.280	0.280	0.280	0.388				
COF	0.507	0.507	0.507	0.507	0.507	0.507				
FEM	73.79	-73.79	73.79	-73.79	73.79	-73.79				
Dist	-28.66	0.00	0.00	0.00	0.00	28.66				
CO	0.00	-14.53	0.00	0.00	14.53	0.00				
Dist	0.00	4.06	4.06	-4.06	-4.06	0.00				
СО	2.06	0.00	-2.06	2.06	0.00	-2.06				
Dist	-0.80	0.58	0.58	-0.58	-0.58	0.80				
СО	0.29	-0.41	-0.29	0.29	0.41	-0.29				
Dist	-0.11	0.20	0.20	-0.20	-0.20	0.11				
СО	0.10	-0.06	-0.10	0.10	0.06	-0.10				
Dist	-0.04	0.04	0.04	-0.04	-0.04	0.04				
CO	0.02	-0.02	-0.02	0.02	0.02	-0.02				
Dist	-0.01	0.01	0.01	-0.01	-0.01	0.01				
CO	0.01	0.00	-0.01	0.01	0.00	-0.01				
Dist	0.00	0.00	0.00	0.00	0.00	0.00				
M ⁻ max	46.65	-83.91	76.21	-76.21	83.91	-46.65				
V	22.25	26.39	24.32	24.32	26.39	22.25				
x _{max}	8.	23	9.	.00	9.	.77				
M^+_{max}	44	.94	33	.23	44	.94				



3.2.4. Design Moments

Positive and negative factored moments for the slab system in the direction of analysis are plotted in Figure 9. The negative moments used for design are taken at the faces of supports (rectangle section or equivalent rectangle for circular or polygon sections) but not at distances greater than $0.175 \times l_1$ from the centers of supports.

ACI 318-14 (8.11.6.1)





Figure 9 - Positive and Negative Design Moments for Slab-Beam (All Spans Loaded with Full Factored Live Load)



3.2.5. Distribution of Design Moments

a) Check whether the moments calculated above can take advantage of the reduction permitted by <u>ACI 318-14</u> (8.11.6.5):

If the slab system analyzed using EFM within the limitations of <u>ACI 318-14 (8.10.2)</u>, it is permitted by the ACI code to reduce the calculated moments obtained from EFM in such proportion that the absolute sum of the positive and average negative design moments need not exceed the value obtained from the following equation:

$$M_o = \frac{q_u \times \ell_2 \times \ell_n^2}{8} = 0.193 \times 14 \times \frac{(16.67)^2}{8} = 93.82$$
 ft-kips

ACI 318-14 (Eq. 8.10.3.2)

End spans: $44.94 + \frac{(32.42 + 66.99)}{2} = 94.65$ ft-kips

Interior span: $33.23 + \frac{(60.61 + 60.61)}{2} = 93.83$ ft-kips

The total design moments from the Equivalent Frame Method yield a static moment equal to that given by the Direct Design Method and no appreciable reduction can be realized.

b) Distribute factored moments to column and middle strips:

After the negative and positive moments have been determined for the slab-beam strip, the ACI code permits the distribution of the moments at critical sections to the column strips, beams (if any), and middle strips in accordance with the DDM. <u>ACI 318-14 (8.11.6.6)</u>

Table 6 – Distribution of factored moments									
		Slab-beam Strip	ın Strip	Midd	Middle Strip				
L	ocation	Moment (ft-kips)	Percent	Moment (ft-kips)	Percent	Moment (ft-kips)			
End Span	Exterior Negative	32.42	100	32.42	0	0			
	Positive	44.94	60	26.96	40	17.98			
	Interior Negative	66.99	75	50.24	25	16.75			
Interior Span	Negative	60.61	75	45.46	25	15.15			
	Positive	33.23	60	19.94	40	13.29			

Distribution of factored moments at critical sections is summarized in Table 6.





3.2.6. Flexural Reinforcement Requirements

a) Determine flexural reinforcement required for strip moments

The flexural reinforcement calculation for the column strip of end span – exterior negative location is provided below.

 $M_u = 32.42$ ft-kips

Use average $d_{avg} = 5.75$ in.

To determine the area of steel, assumptions have to be made whether the section is tension or compression controlled, and regarding the distance between the resultant compression and tension forces along the slab section (*jd*). In this example, tension-controlled section will be assumed so the reduction factor ϕ is equal to 0.9, and *jd* will be taken equal to 0.95*d*. The assumptions will be verified once the area of steel in finalized.

Assume $jd = 0.95 \times d = 5.46$ in.

Column strip width, $b = \frac{14 \times 12}{2} = 84$ in.

Middle strip width, $b = \frac{14 \times 12}{2} = 84$ in.

$$A_s = \frac{M_u}{\phi \times f_v \times jd} = \frac{32.42 \times 12,000}{0.9 \times 60,000 \times 0.95 \times 5.46} = 1.32 \text{ in.}^2$$

Recalculate 'a' for the actual $A_s = 1.32$ in.²:

$$a = \frac{A_s \times f_y}{0.85 \times f_c' \times b} = \frac{1.32 \times 60,000}{0.85 \times 4,000 \times 84} = 0.277 \text{ in.}$$
$$c = \frac{a}{\beta_1} = \frac{0.277}{0.85} = 0.326 \text{ in.}$$
$$\varepsilon = \left(\frac{0.003}{0.85}\right) \times d_z = 0.003 = \left(\frac{0.003}{0.003}\right) \times 5.75 = 0.003 = 0.049$$

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{0.326}\right) \times 5.75 - 0.003 = 0.0499 \ge 0.005$$

Therefore, the assumption that section is tension-controlled is valid.

$$A_{s} = \frac{M_{u}}{\phi \times f_{y} \times \left(d - \frac{a}{2}\right)} = \frac{32.42 \times 12 \times 1,000}{0.9 \times 60,000 \times \left(5.75 - \frac{0.277}{2}\right)} = 1.28 \text{ in.}^{2}$$



ACI 318-14 (24.4.3.2)

Maximum spacing $s_{\text{max}} = 2 \times h = 2 \times 7 = 14$ in. <18 in.

 $A_{s.min} = 0.018 \times 84 \times 7 = 1.06 \text{ in.}^2 < 1.28 \text{ in.}^2$

Provide 7 – #4 bars with $A_s = 1.40$ in.² and $s = \frac{84}{7} = 12$ in. $\leq s_{max}$

Based on the procedure outlined above, values for all span locations are given in Table 7.

Table 7 – Required Slab Reinforcement for Flexure (EFM)									
Span	Location	M _u (ft-kips)	b (in.)	d (in.)	A _{s,req} (in. ²)	$A_{s,min}$ (in. ²)	Reinforcement Provided	As,provided (in. ²)	
			End S	pan					
	Exterior Negative	32.42	84	5.75	1.28	1.06	7 – #4	1.40	
Column Strip	Positive	26.96	84	5.75	1.06	1.06	6-#4	1.20	
	Interior Negative	50.24	84	5.75	2.02	1.06	11 – #4	2.20	
	Exterior Negative	0.00	84	5.75	0.00	1.06	6-#4	1.20	
Middle Strip	Positive	17.98	84	5.75	0.70	1.06	6 – #4	1.20	
	Interior Negative	16.75	84	5.75	0.66	1.06	6-#4	1.20	
	Interior Span								
Column Strip Positive		19.94	84	5.75	0.78	1.06	6-#4	1.20	
Middle Strip	Positive	13.29	84	5.75	0.52	1.06	6-#4	1.20	



ACI 318-14 (8.7.2.2)

b) Calculate additional slab reinforcement at columns for moment transfer between slab and column by flexure

The factored slab moment resisted by the column $(\gamma_f \times M_u)$ shall be assumed to be transferred by flexure. Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist this moment. The fraction of slab moment not calculated to be resisted by flexure shall be assumed to be resisted by eccentricity of shear. ACI 318-14 (8.4.2.3)

Portion of the unbalanced moment transferred by flexure is $\gamma_f \times M_u$ ACI 318-14 (8.4.2.3.1)

Where:

$$\gamma_{f} = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{b_{1}}{b_{2}}}}$$
ACI 318-14 (8.4.2.3.2)

- b_1 = Dimension of the critical section b_o measured in the direction of the span for which moments are determined in ACI 318, Chapter 8 (see Figure 5).
- b_2 = Dimension of the critical section b_o measured in the direction perpendicular to b_1 in ACI 318, Chapter 8 (see Figure 5).
- b_b = Effective slab width = $c_2 + 3 \times h$

The additional reinforcement calculation for the column strip of end span - exterior negative location is provided below.

$$M_{u} = 46.65 \text{ ft-kips} \qquad A_{s(prov)} = 1.40 \text{ in.}^{2}$$

$$b_{1} = c_{1} + \frac{d}{2} = 16 + \frac{5.75}{2} = 18.88 \text{ in.} \qquad b_{2} = c_{2} + d = 16 + 5.75 = 22.75 \text{ in.}$$

$$b_{b} = c_{2} + 3 \times h = 16 + 3 \times 7 = 37 \text{ in.} \qquad \gamma_{f} = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{18.88}{22.75}}} = 0.62$$

$$A_{s} = \frac{0.85 \times f_{c}' \times b_{b}}{f_{y}} \times \left(d - \sqrt{d^{2} - \frac{2 \times \gamma_{f} \times M_{u}}{\phi \times 0.85 \times f_{c}' \times b_{b}}} \right)$$

$$A_{s} = \frac{0.85 \times 4000 \times 37}{60,000} \times \left(5.75 - \sqrt{5.75^{2} - \frac{2 \times 0.62 \times 46.65}{0.9 \times 0.85 \times 4,000 \times 37}} \right) = 1.17 \text{ in.}^{2}$$

$$A_{s,provided within bb} = A_{s,provided} \times \frac{b_{b}}{b} = 1.40 \times \frac{37}{84} = 0.62 \text{ in.}^{2}$$



ACI 318-14 (8.4.2.3.2)

ACI 318-14 (8.4.2.3.3)



 $A_s - A_{s, provided within bb} = 1.17 - 0.62 = 0.55 \text{ in.}^2$

Provide 3 - #4 bars with $A_s = 0.60$ in.²

Table 8 – Additional Slab Reinforcement required for moment transfer between slab and column (EFM)										
Span Location		Mu [*] (ft-kips)	γf	$\gamma_f M_u$ (ft-kips)	<i>b</i> _b (in.)	d (in.)	A _s req'd within b _b (in ²)	A _s prov. For flexure within b _b (in ²)	Add'l Reinf.	
End Span										
Column	Exterior Negative	46.65	0.62	28.78	37	5.75	1.17	0.62	3-#4	
Strip	Interior Negative	7.71	0.60	4.63	37	5.75	0.18	0.97	-	
* M _u is tal	* M _u is taken at the centerline of the support in Equivalent Frame Method solution.									



3.2.7. Column Design Moments

The unbalanced moment from the slab-beams at the supports of the equivalent frame are distributed to the support columns above and below the slab-beam in proportion to the relative stiffness of the support columns. Referring to Figure 9, the unbalanced moment at joints 1 and 2 are:

Joint 1 = +46.65 ft-kips

Joint 2= - 83.91 + 76.21 = - 7.71 ft-kips

The stiffness and carry-over factors of the actual columns and the distribution of the unbalanced slab moments to the exterior and interior columns are shown in the <u>following Figure</u>.



Figure 10 - Column Moments (Unbalanced Moments from Slab-Beam)



In summary:

 $M_{col,Exterior}$ = 22.15 ft-kips

 $M_{col,Interior} = 3.66$ ft-kips

The moments determined above are combined with the factored axial loads (for each story) and factored moments in the transverse direction for design of column sections. The <u>following Figure</u> shows the moment diagrams in the longitudinal and transverse direction for the interior and exterior equivalent frames. Following the previous procedure, the moment values at the face of interior, exterior, and corner columns from the unbalanced moment values can be obtained. These values are shown in the following table.





M_u	Column number (See the previous Figure)									
(kips-ft)	1	2	3	4						
Mux	3.66	22.15	2.05	12.47						
Muy	2.27	1.24	12.26	6.78						

4. Design of Interior, Edge, and Corner Columns

This section includes the design of interior, edge, and corner columns using <u>spColumn</u> software. The preliminary dimensions for these columns were calculated previously in section one. The reduction of live load per <u>ASCE 7-</u> <u>10</u> will be ignored in this example. However, the detailed procedure to calculate the reduced live loads is explained in the "<u>One-Way Wide Module (Skip Joist) Concrete Floor System Design (ACI 318-14)</u>" Design Example.

4.1. Determination of Factored Loads

Assume 4 story building

Interior Column (Column #1):

Tributary area for interior column is $A_{Tributary} = (18 \times 14) = 252 \text{ ft}^2$

- $P_u = 4 \times q_u \times A_{Tributary} = 4 \times 0.193 \times 252 = 194.54$ kips
- $M_{u,x} = 3.66$ ft-kips (see the previous Table)
- $M_{u,y} = 2.27$ ft-kips (see the previous Table)

Edge (Exterior) Column (Column #2):

Tributary area for interior column is $A_{Tributary} = \left(\frac{18}{2} \times 14\right) = 126 \text{ ft}^2$

- $P_u = 4 \times q_u \times A_{Tributary} = 4 \times 0.193 \times 126 = 97.27$ kips
- $M_{u,x} = 22.15$ ft-kips (see the previous Table)
- $M_{u,y} = 1.23$ ft-kips (see the previous Table)



Edge (Exterior) Column (Column #3):

Tributary area for interior column is $A_{Tributary} = \left(18 \times \frac{14}{2}\right) = 126 \text{ ft}^2$

- $P_u = 4 \times q_u \times A_{Tributary} = 4 \times 0.193 \times 126 = 97.27$ kips
- $M_{u,x} = 2.05$ ft-kips (see the previous Table)
- $M_{u,y} = 12.26$ ft-kips (see the previous Table)

Corner Column (Column #4):

Tributary area for interior column is $A_{Tributary} = \left(\frac{18}{2} \times \frac{14}{2}\right) = 63 \text{ ft}^2$

- $P_u = 4 \times q_u \times A_{Tributary} = 4 \times 0.193 \times 63 = 48.64$ kips
- $M_{u,x} = 12.47$ ft-kips (see the previous Table)
- $M_{u,y} = 6.78$ ft-kips (see the previous Table)

The factored loads are then input into spColumn to construct the axial load - moment interaction diagram.




4.2. Column Capacity Diagram (Axial-Moment Interaction Diagram)

Interior Column (Column #1):









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Bar layout	Rectangular								
Cover to	Longitudal bars								
Clear cover	1.88 in								
Bars	4 #8								
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T-1-1-1	0.40.1.2								
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ly	5461.33 in4	1	97.3	2.0	12.3	97.27	23.62	141.25	0.0
Reinforcement								Max. Capac	ity Ratio: 0.09
Pattern	All sides equal								
Bar layout	Rectangular								
Cover to	Longitudal bars								
Clear cover	1.88 in								
Bars	4 #8								
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Туре	Rectangular								
Width	16 in								
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ly	5401,55 111								
Reinforcement								Max. Capad	city Ratio: 0.11
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Bar layout	Rectangular								
Cover to	Longitudal bars								
Clear cover	1.88 in								
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5. Two-Way Slab Shear Strength

Shear strength of the slab in the vicinity of columns/supports includes an evaluation of one-way shear (beam action) and two-way shear (punching) in accordance with <u>ACI 318 Chapter 22</u>.

5.1. One-Way (Beam Action) Shear Strength

ACI 318-14 (22.5)

One-way shear is critical at a distance *d* from the face of the column as shown in Figure 3. Figure 12 shows the factored shear forces (V_u) at the critical sections around each column. In members without shear reinforcement, the design shear capacity of the section equals to the design shear capacity of the concrete:

$$\phi V_{p} = \phi V_{c} + \phi V_{s} = \phi V_{c}, \ (\phi V_{s} = 0)$$
ACI 318-14 (Eq. 22.5.1.1)

Where:

$$\phi V_c = \phi \times 2 \times \lambda \times \sqrt{f'_c} \times b_w \times d$$
ACI 318-14 (Eq. 22.5.5.1)

 $\lambda = 1$ for normal weight concrete

$$\phi V_c = 0.75 \times 2 \times 1.0 \times \frac{\sqrt{4,000}}{1,000} \times (14 \times 12) \times 5.75 = 91.64$$
 kips

Because $\phi V_c \ge V_u$ at all the critical sections, the slab has adequate one-way shear strength.



Figure 12 - One-Way Shear at Critical Sections (at Distance d from the Face of the Supporting Column)





ACI 318-14 (22.6)

Two-way shear is critical on a rectangular section located at d/2 away from the face of the column as shown in Figure 5.

a) Exterior column:

The factored shear force (V_u) in the critical section is computed as the reaction at the centroid of the critical section minus the self-weight and any superimposed surface dead and live load acting within the critical section (d/2 away from column face).

$$V_u = 22.25 - 0.193 \times \left(\frac{21.75 \times 18.88}{144}\right) = 21.70$$
 kips

The factored unbalanced moment used for shear transfer, M_{unb} , is computed as the sum of the joint moments to the left and right. Moment of the vertical reaction with respect to the centroid of the critical section is also taken into account.

$$M_{unb} = 46.65 - 21.70 \times \left(\frac{18.88 - 5.99 - 16/2}{12}\right) = 37.81 \text{ kips-ft}$$

For the exterior column in Figure 5, the location of the centroidal axis z-z is:

$$b_1 = c_1 + \frac{d}{2} = 16 + \frac{5.75}{2} = 18.88$$
 in. $b_2 = c_2 + d = 16 + 5.75 = 21.75$ in.

$$c_{AB} = \frac{b_1^2}{2 \times b_1 + b_2} = \frac{18.88^2}{2 \times 18.88 + 21.75} = 5.99$$
 in.

The polar moment J_c of the shear perimeter is:

 $\gamma_v = 1 - \gamma_f = 1 - 0.617 = 0.383$

$$J_{c} = 2 \times \left(\frac{b_{1} \times d^{3}}{12} + \frac{d \times b_{1}^{3}}{12} + (b_{1} \times d) \left(\frac{b_{1}}{2} - c_{AB}\right)^{2}\right) + b_{2} \times d \times c_{AB}^{2}$$
$$J_{c} = 2 \times \left(\frac{18.88 \times 5.75^{3}}{12} + \frac{5.75 \times 18.88^{3}}{12} + (18.88 \times 5.75) \times \left(\frac{18.88}{2} - 5.99\right)^{2}\right) + 21.75 \times 5.75 \times 5.99^{2} = 14,109 \text{ in.}^{4}$$



Where:

$$\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{b_1}{b_2}}}$$
$$\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{18.88}{21.75}}} = 0.617$$

The length of the critical perimeter for the exterior column:

 $b_o = 2 \times b_1 + b_2 = 2 \times 18.88 + 21.75 = 59.50$ in.

The two-way shear stress (v_u) can then be calculated as:

$$\begin{aligned} v_{u} &= \frac{V_{u}}{b_{o} \times d} + \frac{\gamma_{v} \times M_{unb} \times c_{AB}}{J_{c}} \end{aligned} \qquad \underline{ACI 318-14 (R.8.4.4.2.3)} \\ v_{u} &= \frac{21.70 \times 1,000}{59.5 \times 5.75} + \frac{0.383 \times (37.81 \times 12 \times 1,000) \times 5.99}{14,109} = 63.43 + 73.77 = 137.20 \text{ psi} \end{aligned}$$

$$\begin{aligned} v_{c} &= \min \left\{ \begin{cases} 4 \times \lambda \times \sqrt{f_{c}'} \\ \left(2 + \frac{4}{\beta}\right) \times \lambda \times \sqrt{f_{c}'} \\ \left(\frac{\alpha_{s} \times d}{b_{o}} + 2\right) \times \lambda \times \sqrt{f_{c}'} \end{cases} \right\} \end{aligned}$$

$$\underbrace{ACI 318-14 (Table 22.6.5.2)}_{309.85}$$

 $\phi v_c = 0.75 \times 252.98 = 189.74$ psi

Since $\phi v_c \ge v_u$ at the critical section, the slab has adequate two-way shear strength at this joint.



b) <u>Interior column:</u>

$$V_u = (24.32 + 26.39) - 0.193 \times (\frac{21.75 \times 21.75}{144}) = 50.08$$
 kips

 $M_{uub} = 83.91 - 76.21 - 50.08 \times (0) = 7.70$ kips-ft

For the interior column in Figure 5, the location of the centroidal axis z-z is:

 $b_1 = c_1 + d = 16 + 5.75 = 21.75$ in. $b_2 = c_2 + d = 16 + 5.75 = 21.75$ in.

$$c_{AB} = \frac{b_1}{2} = \frac{21.75}{2} = 10.88$$
 in.

The polar moment J_c of the shear perimeter is:

$$J_{c} = 2 \times \left(\frac{b_{1} \times d^{3}}{12} + \frac{d \times b_{1}^{3}}{12} + (b_{1} \times d) \times \left(\frac{b_{1}}{2} - c_{AB}\right)^{2}\right) + 2 \times b_{2} \times d \times c_{AB}^{2}$$

$$J_{c} = 2 \times \left(\frac{21.75 \times 5.75^{3}}{12} + \frac{5.75 \times 21.75^{3}}{12} + (21.75 \times 5.75) \times (0)^{2}\right) + 2 \times 21.75 \times 5.75 \times 10.88^{2} = 40,131 \text{ in.}^{4}$$

$$\gamma_{v} = 1 - \gamma_{f} = 1 - 0.60 = 0.40$$
ACI 318-14 (Eq. 8.4.4.2.2)

Where:

$$\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{b_1}{b_2}}}$$

$$\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{21.75}{21.75}}} = 0.60$$

The length of the critical perimeter for the interior column:

 $b_o = 2 \times 21.75 + 2 \times 21.75 = 87$ in.





The two-way shear stress (v_u) can then be calculated as:

$$\begin{aligned} v_{u} &= \frac{V_{u}}{b_{v} \times d} + \frac{\gamma_{v} \times M_{unb} \times c_{AB}}{J_{c}} \\ v_{u} &= \frac{50.08 \times 1,000}{87 \times 5.75} + \frac{0.40 \times (7.7 \times 12 \times 1,000) \times 10.88}{40,431} = 100.10 + 10.02 = 110.12 \text{ psi} \end{aligned}$$

$$v_{c} &= \min \begin{cases} 4 \times \lambda \times \sqrt{f_{c}^{\prime}} \\ \left(2 + \frac{4}{\beta}\right) \times \lambda \times \sqrt{f_{c}^{\prime}} \\ \left(\frac{\alpha_{s} \times d}{b_{o}} + 2\right) \times \lambda \times \sqrt{f_{c}^{\prime}} \end{cases}$$

$$\frac{ACI 318 \cdot 14 (Table 22.6.5.2)}{ACI 318 \cdot 14 (Table 22.6.5.2)}$$

$$v_{c} &= \min \begin{cases} 4 \times 1 \times \sqrt{4,000} \\ \left(2 + \frac{4}{1}\right) \times 1 \times \sqrt{4,000} \\ \left(\frac{40 \times 5.75}{87} + 2\right) \times 1 \times \sqrt{4,000} \end{cases} = \min \begin{cases} 252.98 \\ 379.47 \\ 293.69 \end{cases} = 252.98 \text{ psi} \end{aligned}$$

$$\phi v_c = 0.75 \times 252.98 = 189.74$$
 psi

Since $\phi v_c \ge v_u$ at the critical section, the slab has adequate two-way shear strength at this joint.



c) <u>Corner column:</u>

In this example, interior equivalent frame strip was selected where it only has exterior and interior supports (no corner supports are included in this strip). However, the two-way shear strength of corner supports usually governs. Thus, the two-way shear strength for the corner column in this example will be checked for educational purposes. Same procedure is used to find the reaction and factored unbalanced moment used for shear transfer at the centroid of the critical section for the corner support for the exterior equivalent frame strip.

$$V_u = 12.19 - 0.193 \times \left(\frac{18.88 \times 18.88}{144}\right) = 11.71 \text{ kips}$$

$$M_{unb} = 26.08 - 11.71 \times \left(\frac{18.88 - 4.72 - 16/2}{12}\right) = 20.07$$
 kips-ft

For the corner column in Figure 5, the location of the centroidal axis z-z is:

$$b_1 = c_1 + \frac{d}{2} = 16 + \frac{5.75}{2} = 18.88$$
 in. $b_2 = c_2 + \frac{d}{2} = 16 + \frac{5.75}{2} = 18.88$ in.

$$c_{AB} = \frac{b_1^2}{2 \times b_1 + b_2} = \frac{18.88^2}{2 \times 18.88 + 18.88} = 4.72$$
 in

The polar moment J_c of the shear perimeter is:

$$J_{c} = 2 \times \left(\frac{b_{1} \times d^{3}}{12} + \frac{d \times b_{1}^{3}}{12} + (b_{1} \times d) \times \left(\frac{b_{1}}{2} - c_{AB}\right)^{2}\right) + b_{2} \times d \times c_{AB}^{2}$$

$$J_{c} = 2 \times \left(\frac{18.88 \times 5.75^{3}}{12} + \frac{5.75 \times 18.88^{3}}{12} + (18.88 \times 5.75) \times \left(\frac{18.88}{2} - 4.72\right)^{2}\right) + 18.88 \times 5.75 \times 4.72^{2} = 8,354 \text{ in.}^{4}$$

$$\gamma_{v} = 1 - \gamma_{f} = 1 - 0.60 = 0.40$$
 ACI 318-14 (Eq. 8.4.4.2.2)

Where:

 $\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{b_1}{b_2}}}$

ACI 318-14 (8.4.2.3.2)

$$\gamma_f = \frac{1}{1 + \frac{2}{3} \times \sqrt{\frac{18.88}{18.88}}} = 0.60$$



The length of the critical perimeter for the exterior column:

$$b_o = 18.88 + 18.88 = 37.75$$
 in

The two-way shear stress (v_u) can then be calculated as:

$$\begin{aligned} v_{u} &= \frac{V_{u}}{b_{v} \times d} + \frac{\gamma_{v} \times M_{uub} \times c_{AB}}{J_{c}} & \underline{ACI 318.14 (R.8.4.4.2.3)} \\ v_{u} &= \frac{11.71 \times 1,000}{37.75 \times 5.75} + \frac{0.40 \times (20.07 \times 12 \times 1,000) \times 4.72}{8,354} = 53.96 + 54.41 = 108.37 \text{ psi} \end{aligned}$$

$$\begin{aligned} v_{c} &= \min \left\{ \begin{cases} 4 \times \lambda \times \sqrt{f_{c}'} \\ \left(2 + \frac{4}{\beta}\right) \times \lambda \times \sqrt{f_{c}'} \\ \left(\frac{\alpha_{s} \times d}{b_{o}} + 2\right) \times \lambda \times \sqrt{f_{c}'} \end{cases} \right\} \\ \left(\frac{\alpha_{s} \times d}{b_{o}} + 2\right) \times \lambda \times \sqrt{f_{c}'} \end{cases} = \min \left\{ \begin{cases} 252.98 \\ 379.47 \\ 319.16 \end{cases} = 252.98 \text{ psi} \end{cases} \end{aligned}$$

 $\phi v_c = 0.75 \times 252.98 = 189.74$ psi

Since $\phi v_c \ge v_u$ at the critical section, the slab has adequate two-way shear strength at this joint.



6. Two-Way Slab Deflection Control (Serviceability Requirements)

Since the slab thickness was selected based on the minimum slab thickness tables in ACI 318-14, the deflection calculations are not required. However, the calculations of immediate and time-dependent deflections are covered in this section for illustration and comparison with <u>spSlab</u> model results.

6.1. Immediate (Instantaneous) Deflections

The calculation of deflections for two-way slabs is challenging even if linear elastic behavior can be assumed. Elastic analysis for three service load levels $(D, D + L_{sustained}, D + L_{Full})$ is used to obtain immediate deflections of the two-way slab in this example. However, other procedures may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests. ACI 318-14 (24.2.3)

The effective moment of inertia (I_e) is used to account for the cracking effect on the flexural stiffness of the slab. I_e for uncracked section ($M_{cr} > M_a$) is equal to I_g . When the section is cracked ($M_{cr} < M_a$), then the following equation should be used:

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

ACI 318-14 (Eq. 24.2.3.5a)

Where:

 M_a = Maximum moment in member due to service loads at stage deflection is calculated.

The values of the maximum moments for the three service load levels are calculated from structural analysis as shown previously in this document. These moments are shown in the <u>following Figure</u>.





Moment diagram (ft-kips) 3. DL + LL_{full}



Figure 13 – Maximum Moments for the Three Service Load Levels

 M_{cr} = cracking moment.

$$M_{cr} = \frac{f_r \times I_g}{y_t} = \frac{474.34 \times 4,802}{3.5} \times \frac{1}{12 \times 1,000} = 54.23 \text{ ft-kip}$$
ACI 318-14 (Eq. 24.2.3.5b)

 f_r = Modulus of rapture of concrete.

$$f_r = 7.5 \times \lambda \times \sqrt{f_c'} = 7.5 \times 1.0 \times \sqrt{4000} = 474.34 \text{ psi}$$
 ACI 318-14 (Eq. 19.2.3.1)

 I_g = Moment of inertia of the gross uncracked concrete section



$$I_g = \frac{l_2 \times h^3}{12} = \frac{(14 \times 12) \times 7^3}{12} = 4,802 \text{ in.}^2$$

$$y_t = \frac{h}{2} = \frac{7}{2} = 3.5$$
 in.

*I*_{cr} = moment of inertia of the cracked section transformed to concrete. <u>PCA Notes on ACI 318-11 (9.5.2.2)</u>

The calculations shown below are for the design strip (frame strip). The values of these parameters for column and middle strips are shown in <u>Table 9</u>.

As calculated previously, the exterior span frame strip near the interior support is reinforced with 17 #4 bars located at 1.25 in. along the section from the top of the slab. Figure 14 shows all the parameters needed to calculate the moment of inertia of the cracked section transformed to concrete.



Figure 14 - Cracked Transformed Section

 E_{cs} = Modulus of elasticity of slab concrete.

$$E_{cs} = w_c^{1.5} \times 33 \times \sqrt{f_c'} = 150^{1.5} \times 33 \times \sqrt{4,000} = 3,834 \times 10^6 \text{ psi}$$

$$n = \frac{E_s}{E_{cs}} = \frac{29,000,000}{3,834,000} = 7.56$$

$$PCA \text{ Notes on ACI 318-11 (Table 10-2)}$$

$$B = \frac{b}{n \times A_s} = \frac{14 \times 12}{7.56 \times (17 \times 0.2)} = 6.53 \text{ in.}^{-1}$$

$$PCA \text{ Notes on ACI 318-11 (Table 10-2)}$$

$$kd = \frac{\sqrt{2 \times d \times B + 1} - 1}{B} = \frac{\sqrt{2 \times 5.75 \times 6.53 + 1} - 1}{6.53} = 1.18 \text{ in.}$$

$$PCA \text{ Notes on ACI 318-11 (Table 10-2)}$$

$$I_{cr} = \frac{b \times (kd)^3}{3} + n \times A_s \times (d - kd)^2$$

$$PCA \text{ Notes on ACI 318-11 (Table 10-2)}$$



The effective moment of inertia procedure described in the Code is considered sufficiently accurate to estimate deflections. The effective moment of inertia, I_e , was developed to provide a transition between the upper and lower bounds of I_g and I_{cr} as a function of the ratio M_{cr}/M_a . For conventionally reinforced (nonprestressed) members, the effective moment of inertia, I_e , shall be calculated by Eq. (24.2.3.5a) unless obtained by a more comprehensive analysis.

 I_e shall be permitted to be taken as the value obtained from Eq. (24.2.3.5a) at midspan for simple and continuous spans, and at the support for cantilevers. ACI 318-14 (24.2.3.7)

For continuous one-way slabs and beams. I_e shall be permitted to be taken as the average of values obtained from Eq. (24.2.3.5a) for the critical positive and negative moment sections. <u>ACI 318-14 (24.2.3.6)</u>



For the exterior span (span with one end continuous) with service load level $(D + LL_{full})$:

Since $M_{cr} = 54.23$ ft-kips $< M_a = 64.13$ ft-kips

$$I_e^- = \left(\frac{M_{cr}}{M_a}\right)^3 \times I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \times I_{cr} \qquad \underline{ACI 318-14 (24.2.3.5a)}$$

Where I_e^- is the effective moment of inertia for the critical negative moment section (near the support).

$$I_{e}^{-} = \left(\frac{54.23}{64.13}\right)^{3} \times 4,802 + \left[1 - \left(\frac{54.23}{64.17}\right)^{3}\right] \times 629 = 3,152 \text{ in.}^{4}$$
$$I_{e}^{+} = I_{g} = 4,802 \text{ in.}^{4} \text{ , since } M_{cr} = 54.23 \text{ ft-kips} > M_{a} = 34.35 \text{ ft-kips}$$

Where I_e^+ is the effective moment of inertia for the critical positive moment section (midspan).

Since midspan stiffness (including the effect of cracking) has a dominant effect on deflections, midspan section is heavily represented in calculation of I_e and this is considered satisfactory in approximate deflection calculations. The averaged effective moment of inertia ($I_{e,avg}$) is given by:

$$I_{e,avg} = 0.85 \times I_e^+ + 0.15 \times I_e^- \text{ for end span}$$

$$PCA \text{ Notes on ACI 318-11 (9.5.2.4(1))}$$

$$I_{e,avg} = 0.85 \times (4,802) + 0.15 \times (3,148) = 4,554 \text{ in.}^4$$

Where:

- I_e^- = The effective moment of inertia for the critical negative moment section near the support.
- I_e^+ = The effective moment of inertia for the critical positive moment section (midspan).



For the interior span (span with both ends continuous) with service load level $(D + LL_{full})$:

$$I_{e}^{-} = \left(\frac{M_{cr}}{M_{a}}\right)^{3} \times I_{g} + \left[1 - \left(\frac{M_{cr}}{M_{a}}\right)^{3}\right] \times I_{cr} \text{, since } M_{cr} = 54.23 \text{ ft-kips} < M_{a} = 58.24 \text{ ft-kips} \quad \underline{ACI 318-14 (24.2.3.5a)}$$
$$I_{e}^{-} = \left(\frac{54.23}{58.24}\right)^{3} \times 4,802 + \left[1 - \left(\frac{54.23}{58.24}\right)^{3}\right] \times 629 = 3,998 \text{ in.}^{4}$$
$$I_{e}^{+} = I_{g} = 4,802 \text{ in.}^{4} \text{, since } M_{cr} = 54.23 \text{ ft-kips} > M_{a} = 25.39 \text{ ft-kips}$$

The averaged effective moment of inertia $(I_{e,avg})$ is given by:

 $I_{e,avg} = 0.70 \times I_{e}^{+} + 0.15 \times (I_{e,l}^{-} + I_{e,r}^{-}) \text{ for interior span} \qquad \underline{PCA \text{ Notes on ACI 318-11 (9.5.2.4(2))}}$ $I_{e,avg} = 0.70 \times (4,802) + 0.15 \times (3,993 + 3,993) = 4,559 \text{ in.}^{4}$

Where:

 $I_{e,L}$ = The effective moment of inertia for the critical negative moment section near the left support.

 $I_{e,R}$ = The effective moment of inertia for the critical negative moment section near the right support.

The <u>Table 9</u> provides a summary of the required parameters and calculated values needed for deflections for exterior and interior equivalent frame. It also provides a summary of the same values for column strip and middle strip to facilitate calculation of panel deflection.

	Table 9 – Averaged Effective Moment of Inertia Calculations												
					For	· Frame	Strip						
		т	т	N	Ia (ft-kij))	м		Ie (in.4)		I	e,avg (in. ⁴)
Span	zone	(in. ⁴)	(in. ⁴)	D	D + LL _{Sus}	D + L _{full}	Mcr (k-ft)	D	D + LL _{Sus}	D + L _{full}	D	D + LL _{Sus}	D + L _{full}
	Left		499	-25.99	-25.99	-35.65		4802	4802	4802			
Ext	Midspan		466	25.03	25.03	34.35		4802	4802	4802	4802	4802	4555
	Right	4900	629	-46.74	-46.74	-64.13	54.02	4802	4802	3153			
	Left	4802	629	-42.45	-42.45	-58.24	54.25	4802	4802	3999			
Int	Mid		466	18.51	18.51	25.39		4802	4802	4802	4802	4802	4561
	Right		629	-42.45	-42.45	-58.24		4802	4802	3999			





Deflections in two-way slab systems shall be calculated taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges. For immediate deflections two-way slab systems the midpanel deflection is computed as the sum of deflection at midspan of the column strip or column line in one direction (Δ_{cx} or Δ_{cy}) and deflection at midspan of the middle strip in the orthogonal direction (Δ_{mx} or Δ_{my}). Figure 15 shows the deflection computation for a rectangular panel. The average Δ for panels that have different properties in the two direction is calculated as follows:







To calculate each term of the previous equation, the following procedure should be used. Figure 16 shows the procedure of calculating the term Δ_{cx} . same procedure can be used to find the other terms.



<u>Figure 16 – Δ_{cx} Calculation Procedure</u>

For exterior span - service dead load case:

$$\Delta_{frame, fixed} = \frac{W \times l^4}{384 \times E_c \times I_{frame, averaged}}$$
PCA Notes on ACI 318-11 (9.5.3.4 Eq. 10)

Where:

 Δ_{frame_fixed} = Deflection of column strip assuming fixed-end condition.

$$w = \left(20 + 150 \times \frac{7}{12}\right) \times 14 = 1,505 \quad \frac{\text{lb}}{\text{ft}}$$
$$E_{cs} = w_c^{1.5} \times 33 \times \sqrt{f_c'} = 150^{1.5} \times 33 \times \sqrt{4,000} = 3,834 \times 10^3 \text{ psi}$$
ACI 318-14 (19.2.2.1.a)

 $I_{frame,averaged}$ = The averaged effective moment of inertia ($I_{e,avg}$) for the frame strip for service dead load case from Table 9 = 4,802 in.⁴

$$\Delta_{frame, fixed} = \frac{1,505 \times 18^4 \times 12^3}{384 \times (3,834 \times 10^3) \times 4,802} = 0.039 \text{ in.}$$

$$\Delta_{c,fixed} = LDF_c \times \Delta_{frame,fixed} \times \frac{I_{frame,averaged}}{I_{c,g}}$$

PCA Notes on ACI 318-11 (9.5.3.4 Eq. 11)



Where LDF_c is the load distribution factor for the column strip. The load distribution factor for the column strip can be found from the following equation:

$$LDF_{c} = \frac{LDF^{+} + \frac{LDF_{l}^{-} + LDF_{R}^{-}}{2}}{2}$$

And the load distribution factor for the middle strip can be found from the following equation:

$$LDF_m = 1 - LDF_c$$

For the end span, LDF for exterior negative region (LDF_L) , interior negative region (LDF_R) , and positive region (LDF_L) are 1.00, 0.75, and 0.60, respectively (From Table 6 of this document). Thus, the load distribution factor for the column strip for the end span is given by:

$$LDF_{c} = \frac{0.6 + \frac{1.0 + 0.75}{2}}{2} = 0.738$$

 $I_{c,g}$ = The gross moment of inertia (I_g) for the column strip for service dead load = 2,401 in.⁴

$$\Delta_{c,fixed} = 0.738 \times 0.039 \times \frac{4,802}{2,401} = 0.057 \text{ in.}$$

$$\theta_{c,L} = \frac{(M_{net,L})_{frame}}{K_{ec}}$$
PCA Notes on ACI 318-11 (9.5.3.4 Eq. 12)

Where:

 $\theta_{c,L}$ = Rotation of the left support

 $(M_{net,L})_{frame} = 25.99$ ft-kips = Net frame strip negative moment of the left support

 K_{ec} = Effective column stiffness = 554.07 × 10⁶ in.-lb (<u>calculated previously</u>).

$$\theta_{c,L} = \frac{25.99 \times 12 \times 1,000}{554.3 \times 10^6} = 0.0006 \text{ rad}$$

$$\Delta \theta_{c,L} = \theta_{c,L} \times \left(\frac{l}{8}\right) \times \left(\frac{I_g}{I_e}\right)_{frame}$$

PCA Notes on ACI 318-11 (9.5.3.4 Eq. 14)

Where:



 $\Delta \theta_{c,L}$ = Midspan deflection due to rotation of left support.

 $(I_g / I_e)_{frame}$ = Gross to effective moment of inertia ratio for frame strip.

 $\Delta \theta_{c,L} = 0.0006 \times \frac{18 \times 12}{8} \times \frac{4,802}{4,802} = 0.0152 \text{ in.}$

$$\theta_{c,R} = \frac{(M_{net,R})_{frame}}{K_{ec}} = \frac{(46.74 - 42.45) \times 12 \times 1000}{554.3 \times 10^6} = 0.00010 \text{ rad}$$

Where

 $\theta_{c,R}$ = rotation of the span right support.

 $(M_{net,R})_{frame}$ = Net frame strip negative moment of the right support.

$$\Delta \theta_{c,R} = \theta_{c,R} \times \left(\frac{l}{8}\right) \times \left(\frac{I_g}{I_e}\right)_{frame} = 0.0001 \times \frac{18 \times 12}{8} \times \frac{4,802}{4,802} = 0.0025 \text{ in.}$$

Where:

 $\Delta \theta_{c,R}$ = Midspan deflection due to rotation of right support.

$$\Delta_{cx} = \Delta_{cx, fixed} + \Delta \theta_{cx,R} + \Delta \theta_{cx,L}$$
PCA Notes on ACI 318-11 (9.5.3.4 Eq. 9)

$$\Delta_{cr} = 0.057 + 0.0152 + 0.0025 = 0.0747$$
 in.

Following the same procedure, Δ_{mx} can be calculated for the middle strip. This procedure is repeated for the equivalent frame in the orthogonal direction to obtain Δ_{cy} , and Δ_{my} for the end and middle spans for the other load levels ($D+LL_{sus}$ and $D+LL_{full}$).

Assuming square panel, $\Delta_{cx} = \Delta_{cy} = 0.0747$ in. and $\Delta_{mx} = \Delta_{my} = 0.0386$ in.

The average \varDelta for the corner panel is calculated as follows:

$$\Delta = \frac{\left(\Delta_{cx} + \Delta_{my}\right) + \left(\Delta_{cy} + \Delta_{mx}\right)}{2} = \left(\Delta_{cx} + \Delta_{my}\right) = \left(\Delta_{cy} + \Delta_{mx}\right) = 0.0747 + 0.0386 = 0.113 \text{ in.}$$





Table 10 – Immediate (Instantaneous) Deflections in the x-direction

Column Strip

a					D			
Span	LDF	Δ _{frame-fixed} (in.)	Δ _{c-fixed} (in.)	θ _{c1} (rad)	θ_{c2} (rad)	Δθ _{c1} (in.)	Δθ _{c2} (in.)	Δ _{cx} , (in.)
Ext	0.738	0.0386	0.0570	0.0006	0.0001	0.0152	0.0025	0.0747
Int	0.675	0.0386	0.0521	-0.0001	-0.0001	-0.0025	-0.0025	0.0471

				D			
LDF	$\Delta_{ ext{frame-fixed}}$ (in.)	Δ _{m-fixed} (in.)	θ_{m1} (rad)	θ_{m2} (rad)	$\Delta \theta_{m1}$ (in.)	$\Delta \theta_{m2}$ (in.)	Δ_{mx} (in.)
0.263	0.0386	0.0203	0.0006	0.0001	0.0152	0.0025	0.0380
0.325	0.0386	0.0251	-0.0001	-0.0001	-0.0025	-0.0025	0.0201

Middle Strip

a					D+LL _{sus}			
Span	LDF	$\Delta_{ ext{frame-fixed}}$ (in.)	Δ _{c-fixed} (in.)	θ_{c1} (rad)	θ _{c2} (rad)	$\Delta \theta_{c1}$ (in.)	Δθ _{c2} (in.)	Δ_{cx} (in.)
Ext	0.738	0.0386	0.0570	0.0006	0.0001	0.0152	0.0025	0.0747
Int	0.675	0.0386	0.0521	-0.0001	-0.0001	-0.0025	-0.0025	0.0471

				D+LL _{sus}			
LDF	$\Delta_{ ext{frame-fixed}}$ (in.)	Δ _{m-fixed} (in.)	θ_{m1} (rad)	θ_{m2} (rad)	Δθ _{m1} (in.)	$\Delta \theta_{m2}$ (in.)	Δ_{mx} (in.)
0.263	0.0386	0.0203	0.0006	0.0001	0.0152	0.0025	0.0380
0.325	0.0386	0.0251	-0.0001	-0.0001	-0.0025	-0.0025	0.0201

a					D+LL _{full}			
Span	LDF	Δ _{frame-fixed} (in.)	$\Delta_{ ext{c-fixed}}$ (in.)	θ_{c1} (rad)	θ_{c2} (rad)	Δθ _{c1} (in.)	$\Delta \theta_{c2}$ (in.)	Δ_{cx} (in.)
Ext	0.738	0.0559	0.0782	0.0008	0.0001	0.0208	0.0052	0.1042
Int	0.675	0.0558	0.0715	-0.0001	-0.0001	-0.0041	-0.0041	0.0633

				D+LL _{full}			
LDF	$\Delta_{ ext{frame-fixed}}$ (in.)	Δ _{m-fixed} (in.)	θ_{m1} (rad)	θ_{m2} (rad)	$\Delta \theta_{m1}$ (in.)	$\Delta \theta_{m2}$ (in.)	Δ_{mx} (in.)
0.263	0.0559	0.0278	0.0008	0.0001	0.0208	0.0052	0.0539
0.325	0.0558	0.0344	-0.0001	-0.0001	-0.0041	-0.0041	0.0262

a	LDE	LL
Span	LDF	Δ _{cx} (in.)
Ext	0.738	0.0296
Int	0.675	0.0161

	LL
LDF	Δ_{mx} (in.)
0.262	0.0159
0.325	0.0061



From the analysis in the transverse direction the deflection values below are obtained:

For DL loading case:

 Δ_{my}

 Δ_{cy}

For *DL*+*LL*_{sust} loading case:

 Δ_{my}

 Δ_{cy}

For *DL*+*LL*_{full} loading case:

 Δ_{my}

 Δ_{cy}

These values for the x-direction are shown in the previous Table. Then, the total midpanel deflection is calculated by combining the contributions of the column and middle strip deflections from the X and Y directions:

 $\Delta = \frac{\left(\Delta_{cx} + \Delta_{my}\right) + \left(\Delta_{cy} + \Delta_{mx}\right)}{2}$

PCA Notes on ACI 318-11 (9.5.3.4 Eq. 8)

6.2. Time-Dependent (Long-Term) Deflections (Δlt)

The additional time-dependent (long-term) deflection resulting from creep and shrinkage (Δ_{cs}) may be estimated as follows:

$$\Delta_{cs} = \lambda_{\Lambda} \times (\Delta_{sust})_{last}$$
PCA Notes on ACI 318-11 (9.5.2.5 Eq. 4)

The total time-dependent (long-term) deflection is calculated as:

$$(\Delta_{total})_{lt} = (\Delta_{sust})_{Inst} \times (1 + \lambda_{\Delta}) + [(\Delta_{total})_{Inst} - (\Delta_{sust})_{Inst}]$$

$$\underline{CSA \ A23.3-04 \ (N9.8.2.5)}$$

Where:

 $(\Delta_{sust})_{Inst}$ = Immediate (instantaneous) deflection due to sustained load, in.

$$\lambda_{\Delta} = \frac{\xi}{1 + 50 \times \rho'}$$
 ACI 318-14 (24.2.4.1.1)

 $(\Delta_{total})_{lt}$ = Time-dependent (long-term) total deflection, in.

 $(\Delta_{total})_{Inst}$ = Total immediate (instantaneous) deflection, in.

 $\xi = 2$, consider the sustained load duration to be 60 months or more. ACI 318-14 (Table 24.2.4.1.3)

 ρ ` = 0, conservatively.

$$\lambda_{\Delta} = \frac{2}{1+50\times0} = 2$$

$$\Delta_{cs} = 2 \times 0.0747 = 0.1493$$
 in.

$$(\Delta_{total})_{lt} = 0.0747 \times (1+2) + (0.104 - 0.0747) = 0.254$$
 in.

The following Table shows long-term deflections for the exterior and interior spans for the analysis in the xdirection, for column and middle strips.









Table 11 – Long-Term Deflections									
Column Strip									
Span	Span $(\Delta_{sust})_{Inst}$ (in.) λ_A Δ_{cs} (in.) $(\Delta_{total})_{Inst}$ (in.) $(\Delta_{total})_{lt}$ (in.)								
Exterior	0.0747	2	0.149	0.104	0.254				
Interior	0.0471	2	0.094	0.063	0.157				
Middle Strip									
Exterior	0.0380	2	0.076	0.054	0.130				
Interior	0.0201	2	0.040	0.026	0.066				



7. spSlab Software Program Model Solution

<u>spSlab</u> program utilizes the Equivalent Frame Method described and illustrated in details here for modeling, analysis and design of two-way concrete floor slab systems. <u>spSlab</u> uses the exact geometry and boundary conditions provided as input to perform an elastic stiffness (matrix) analysis of the equivalent frame taking into account the torsional stiffness of the slabs framing into the column. It also takes into account the complications introduced by a large number of parameters such as vertical and torsional stiffness of transverse beams, the stiffening effect of drop panels, column capitals, and effective contribution of columns above and below the floor slab using the equivalent column concept (<u>ACI 318-14 (R8.11.4)</u>).

<u>spSlab</u> program models the equivalent frame as a design strip. The design strip is, then, separated by <u>spSlab</u> into column and middle strips. The program calculates the internal forces (Shear Force & Bending Moment), moment and shear capacity vs. demand diagrams for column and middle strips, instantaneous and long-term deflection results, and required flexural reinforcement for column and middle strips.







spSlab v10.00 (TM) A Computer Program for Analysis, Design, and Investigation of Reinforced Concrete Beams, One-way and Two-way Slab Systems Copyright © 1992-2024, STRUCTUREPOINT, LLC. All rights reserved



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1. Input Echo

1.1. General Information

File Name	F:\Struc\DE-Two-Way-Flat-Plate-ACI-318- 14.slbx
Project	Two-Way Flat Plate Floor
Frame	Interior Frame
Engineer	SP
Code	ACI 318-14
Units	English
Reinforcement Database	ASTM A615
Mode	Design
Number of supports =	4 + Left Cantilever + Right Cantilever
Floor System	Two-Way

1.2. Solve Options

Live load pattern ratio = 0%
Minimum free edge distance for punching shear = 4 times slab thickness.
Circular critical section around circular supports used (if possible).
Deflections are based on cracked section properties.
In negative moment regions, Ig and Mcr DO NOT include flange/slab contribution (if available)
Long-term deflections are calculated for load duration of 60 months.
0% of live load is sustained.
Compression reinforcement calculations NOT selected.
Default incremental rebar design selected.
User-defined slab strip widths NOT selected.
User-defined distribution factors NOT selected.
One-way shear in drop panel NOT selected.
Distribution of shear to strips NOT selected.
Beam T-section design NOT selected.
Longitudinal beam contribution in negative reinforcement design over support NOT selected.
Transverse beam contribution in negative reinforcement design over support NOT selected.

1.3. Material Properties

1.3.1. Concrete: Slabs / Beams

w _c	150 p
f'c	4 k
Ec	3834.25 k
fr	0.474342 k

1.3.2. Concrete: Columns

w _c	150	pcf
f'c	6	ksi
Ec	4695.98	ksi
fr	0.580948	ksi

1.3.3. Reinforcing Steel

f_y 60 ksi

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f _{yt}	60	ksi
Es	29000	ksi
Epoxy coated bars	No	

1.4. Reinforcement Database

Size	Db	Ab	Wb	Size	Db	Ab	Wb
	in	in²	lb/ft		in	in²	lb/ft
#3	0.38	0.11	0.38	#4	0.50	0.20	0.67
#5	0.63	0.31	1.04	#6	0.75	0.44	1.50
#7	0.88	0.60	2.04	#8	1.00	0.79	2.67
#9	1.13	1.00	3.40	#10	1.27	1.27	4.30
#11	1.41	1.56	5.31	#14	1.69	2.25	7.65
#18	2.26	4.00	13.60				

1.5. Span Data

1.5.1. Slabs

Notes: Deflection check required for panels where code-specified Hmin for two-way construction doesn't apply due to: *i - cantilever end span (LC, RC) support condition

Span	Loc	L1	t	wL	wR	L2L	L2R	H _{min}
		ft	in	ft	ft	ft	ft	in
1	Int	0.667	7.00	7.000	7.000	14.000	14.000	LC *i
2	Int	18.000	7.00	7.000	7.000	14.000	14.000	6.67
3	Int	18.000	7.00	7.000	7.000	14.000	14.000	6.06
4	Int	18.000	7.00	7.000	7.000	14.000	14.000	6.67
5	Int	0.667	7.00	7.000	7.000	14.000	14.000	RC *i

1.6. Support Data

1.6.1. Columns

Support	c1a	c2a	Ha	c1b	c2b	Hb	Red %
	in	in	ft	in	in	ft	
1	16.00	16.00	9.000	16.00	16.00	9.000	100
2	16.00	16.00	9.000	16.00	16.00	9.000	100
3	16.00	16.00	9.000	16.00	16.00	9.000	100
4	16.00	16.00	9.000	16.00	16.00	9.000	100

1.6.2. Boundary Conditions

Support	Spring		Far Er	nd
	K _z kips/in	K _{ry} kip-in/rad	Above	Below
1	0.00	0.00	Fixed	Fixed
2	0.00	0.00	Fixed	Fixed
3	0.00	0.00	Fixed	Fixed
4	0.00	0.00	Fixed	Fixed

1.7. Load Data

1.7.1. Load Cases and Combinations

Case	SELF	Dead	Live
Туре	DEAD	DEAD	LIVE
 U1	1.200	1.200	1.600





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1.7.2.	Area	Loads
--------	------	-------

Case/Patt	Span	Wa
		psf
SELF	1	87.50
	2	87.50
	3	87.50
	4	87.50
	5	87.50
Dead	1	20.00
	2	20.00
	3	20.00
	4	20.00
	5	20.00
Live	1	40.00
	2	40.00
	3	40.00
	4	40.00
	5	40.00

1.8. Reinforcement Criteria

1.8.1. Slabs and Ribs

	Units	Units Top Bars			Bars
		Min.	Max.	Min.	Max.
Bar Size		#4	#4	#4	#4
Bar spacing	in	1.00	18.00	1.00	18.00
Reinf ratio	%	0.18	2.00	0.18	2.00
Clear Cover	in	1.00		1.00	

There is NOT more than 12 in of concrete below top bars.

1.8.2. Beams

	Units	Top Ba	ars	Bottom I	Bars	Stirrups	
		Min.	Max.	Min.	Max.	Min.	Max
Bar Size		#5	#8	#5	#8	#3	#5
Bar spacing	in	1.00	18.00	1.00	18.00	6.00	18.00
Reinf ratio	%	0.14	5.00	0.14	5.00		
Clear Cover	in	1.50		1.50			
Layer dist.	in	1.00		1.00			
No. of legs						2	6
Side cover	in					1.50	
1st Stirrup	in					3.00	

There is NOT more than 12 in of concrete below top bars.

2. Design Results*

*Unless otherwise noted, all results are in the direction of analysis only. Another analysis in the perpendicular direction has to be carried out for two-way slab systems.

2.1. Strip Widths and Distribution Factors

Notes: *Used for bottom reinforcement. **Used for top reinforcement.

			Width		Moment Factor				
Span	Strip	Left **	Right ** Bottom		Left **	Right **	Bottom *		
		ft	ft	ft					
1	Column	7.00	7.00	7.00	1.000	1.000	0.600		
	Middle	7.00	7.00	7.00	0.000	0.000	0.400		



			Width		м	Moment Factor				
Span	Strip	Left **	Right **	Bottom *	Left **	Right ** Botton				
		ft	ft	ft						
2	Column	7.00	7.00	7.00	1.000	0.750	0.600			
	Middle	7.00	7.00	7.00	0.000	0.250	0.400			
3	Column	7.00	7.00	7.00	0.750	0.750	0.600			
	Middle	7.00	7.00	7.00	0.250	0.250	0.400			
4	Column	7.00	7.00	7.00	0.750	1.000	0.600			
	Middle	7.00	7.00	7.00	0.250	0.000	0.400			
5	Column	7.00	7.00	7.00	1.000	1.000	0.600			
	Middle	7.00	7.00	7.00	0.000	0.000	0.400			

2.2. Top Reinforcement

Notes: *3 - Design governed by minimum reinforcement.

Span	Strip	Zone	Width	M _{max}	X _{max}	$\mathbf{A}_{\mathrm{s,min}}$	A _{s,max}	$A_{s,req}$	SpProv	Bars	
			ft	kip-ft	ft	in²	in²	in²	in		
1	Column	Left	7.00	0.06	0.192	1.058	8.724	0.002	14.000	6-#4	*3
		Midspan	7.00	0.19	0.357	1.058	8.724	0.007	14.000	6-#4	*3
		Right	7.00	0.43	0.550	1.058	8.724	0.016	12.000	7-#4	*3
	Middle	Left	7.00	0.00	0.000	1.058	8.724	0.000	14.000	6-#4	*3
		Midspan	7.00	0.00	0.275	1.058	8.724	0.000	14.000	6-#4	*3
		Right	7.00	0.00	0.550	1.058	8.724	0.000	14.000	6-#4	*3
2	Column	Left	7.00	32.66	0.667	1.058	8.724	1.293	12.000	7-#4	
		Midspan	7.00	0.00	9.000	0.000	8.724	0.000	0.000		
		Right	7.00	50.21	17.333	1.058	8.724	2.015	7.636	11-#4	
	Middle	Left	7.00	0.20	1.662	1.058	8.724	0.008	14.000	6-#4	*3
		Midspan	7.00	0.00	9.000	0.000	8.724	0.000	0.000		
		Right	7.00	16.74	17.333	1.058	8.724	0.655	14.000	6-#4	*3
3	Column	Left	7.00	45.47	0.667	1.058	8.724	1.818	7.636	11-#4	
		Midspan	7.00	0.00	9.000	0.000	8.724	0.000	0.000		
		Right	7.00	45.47	17.333	1.058	8.724	1.818	7.636	11-#4	
	Middle	Left	7.00	15.16	0.667	1.058	8.724	0.592	14.000	6-#4	*3
		Midspan	7.00	0.00	9.000	0.000	8.724	0.000	0.000		
		Right	7.00	15.16	17.333	1.058	8.724	0.592	14.000	6-#4	*3
4	Column	Left	7.00	50.22	0.667	1.058	8.724	2.015	7.636	11-#4	
		Midspan	7.00	0.00	9.000	0.000	8.724	0.000	0.000		
		Right	7.00	32.66	17.333	1.058	8.724	1.293	12.000	7-#4	
	Middle	Left	7.00	16.74	0.667	1.058	8.724	0.655	14.000	6-#4	*3
		Midspan	7.00	0.00	9.000	0.000	8.724	0.000	0.000		
		Right	7.00	0.20	16.338	1.058	8.724	0.008	14.000	6-#4	*3
5	Column	Left	7.00	0.43	0.117	1.058	8.724	0.016	12.000	7-#4	*3
		Midspan	7.00	0.19	0.309	1.058	8.724	0.007	14.000	6-#4	*3
		Right	7.00	0.06	0.474	1.058	8.724	0.002	14.000	6-#4	*3

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s	ipan Strip	Zone	Width ft	M_{max} kip-ft	X _{max} ft	A _{s,min} in²	A _{s,max} in²	A _{s,req} in²	Sp _{Prov} in	Bars	
	Middle	Left	7.00	0.00	0.117	1.058	8.724	0.000	14.000	6-#4 *3	
		Midspan	7.00	0.00	0.392	1.058	8.724	0.000	14.000	6-#4 *3	
		Right	7.00	0.00	0.667	1.058	8.724	0.000	14.000	6-#4 *3	

2.3. Top Bar Details

			Lef	t		Contin	nuous		Rig	ht	
Span	Strip	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
			ft		ft		ft		ft		ft
1	Column					6-#4	0.67	1-#4	0.67		
	Middle					6-#4	0.67				
2	Column	6-#4	6.17	1-#4	4.00			6-#4	6.17	5-#4	4.00
	Middle	6-#4	4.33					6-#4	5.19		
3	Column	6-#4	6.17	5-#4	4.00			6-#4	6.17	5-#4	4.00
	Middle	6-#4	5.19	10000				6-#4	5.19		
4	Column	6-#4	6.17	5-#4	4.00			6-#4	6.17	1-#4	4.00
	Middle	6-#4	5.19					6-#4	4.33		
5	Column	1-#4	0.67			6-#4	0.67				
	Middle					6-#4	0.67				

2.4. Top Bar Development Lengths

			Lef	ť		Conti	nuous		Rig	ht	
Span	Strip	Bars	DevLen	Bars	DevLen	Bars	DevLen	Bars	DevLen	Bars	DevLen
			in		in		in		in		in
1	Column					6-#4	12.00	1-#4	12.00		
	Middle					6-#4	12.00				
2	Column	6-#4	12.00	1-#4	12.00			6-#4	12.00	5-#4	12.00
	Middle	6-#4	12.00		82274 (Del 1944) (C			6-#4	12.00		
3	Column	6-#4	12.00	5-#4	12.00			6-#4	12.00	5-#4	12.00
	Middle	6-#4	12.00		100000000000000000000000000000000000000			6-#4	12.00		
4	Column	6-#4	12.00	5-#4	12.00			6-#4	12.00	1-#4	12.00
	Middle	6-#4	12.00					6-#4	12.00		
5	Column	1-#4	12.00			6-#4	12.00				
	Middle					6-#4	12.00				

2.5. Bottom Reinforcement

Notes: *3 - Design governed by minimum reinforcement.

Span Strip	Width	M _{max}	X _{max}	A _{s,min}	A _{s,max}	A _{s,req}	SpProv	Bars	
	ft	kip-ft	ft	in²	in²	in²	in		
1 Column	7.00	0.00	0.275	0.000	8.724	0.000	0.000		
Middle	7.00	0.00	0.275	0.000	8.724	0.000	0.000		
2 Column	7.00	26.87	8.129	1.058	8.724	1.059	14.000	6-#4	



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Span Strip	Width	M _{max}	X _{max}	A _{s,min}	A _{s,max}	A _{s,req}	SpProv	Bars	
	ft	kip-ft	ft	in²	in²	in²	in		
Middle	7.00	17.91	8.129	1.058	8.724	0.701	14.000	6-#4	*3
3 Column	7.00	19.90	9.124	1.058	8.724	0.780	14.000	6-#4	*3
Middle	7.00	13.27	9.124	1.058	8.724	0.518	14.000	6-#4	*3
4 Column	7.00	26.87	9.871	1.058	8.724	1.059	14.000	6-#4	
Middle	7.00	17.91	9.871	1.058	8.724	0.701	14.000	6-#4	*3
5 Column	7.00	0.00	0.392	0.000	8.724	0.000	0.000		
Middle	7.00	0.00	0.392	0.000	8.724	0.000	0.000		

2.6. Bottom Bar Details

		L	ong Ba	ars	Short Bars				
Span	Strip	Bars	Start	Length	Bars	Start	Length		
			ft	ft		ft	ft		
1	Column								
	Middle								
2	Column	6-#4	0.00	18.00					
	Middle	6-#4	0.00	18.00					
3	Column	6-#4	0.00	18.00					
	Middle	6-#4	0.00	18.00					
4	Column	6-#4	0.00	18.00					
	Middle	6-#4	0.00	18.00					
5	Column								
	Middle								

2.7. Bottom Bar Development Lengths

		Lon	g Bars	Short Bars		
Span	Strip	Bars	DevLen	Bars	DevLen	
1	Column					
	Middle					
2	Column	6-#4	12.00			
	Middle	6-#4	12.00			
3	Column	6-#4	12.00			
	Middle	6-#4	12.00			
4	Column	6-#4	12.00			
	Middle	6-#4	12.00			
5	Column					
	Middle					



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2.8. Flexural Capacity

		Тор					Bottom					
Span Strip	р	x	$A_{s,top}$	ΦM _n -	Mu-	Comb Pat	Status	A _{s,bot}	ΦM _n +	M _u +	Comb Pat	Status
		ft	in²	kip-ft	kip-ft			in²	kip-ft	kip-ft		
1 Colu	ımn	0.000	1.40	-35.30	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.192	1.40	-35.30	-0.06	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.333	1.40	-35.30	-0.17	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.357	1.40	-35.30	-0.19	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.550	1.40	-35.30	-0.43	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.667	1.40	-35.30	-0.60	U1 All		0.00	0.00	0.00	U1 All	
Midd	dle	0.000	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.192	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.333	1.20	-30.37	0.00	U1 All	ок	0.00	0.00	0.00	U1 All	OK
		0.357	1.20	-30.37	0.00	U1 All	ок	0.00	0.00	0.00	U1 All	OK
		0.550	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK
		0.667	1.20	-30.37	0.00	U1 All		0.00	0.00	0.00	U1 All	
2 Colu	ımn	0.000	1.40	-35.30	-47.36	U1 All		1.20	30.37	0.00	U1 All	
		0.444	1.40	-35.30	-37.39	U1 All		1.20	30.37	0.00	U1 All	
		0.667	1.40	-35.30	-32.66	U1 All	OK	1.20	30.37	0.00	U1 All	OK
		3.000	1.40	-35.30	0.00	U1 All	OK	1.20	30.37	4.62	U1 All	OK
		4.000	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	12.31	U1 All	OK
		5.167	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	19.23	U1 All	OK
		6.167	0.00	0.00	0.00	U1 All	OK	1.20	30.37	23.40	U1 All	OK
		6.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	24.42	U1 All	OK
		8.129	0.00	0.00	0.00	U1 All	OK	1.20	30.37	26.87	U1 All	OK
		9.000	0.00	0.00	0.00	U1 All	OK	1.20	30.37	26.40	U1 All	OK
		11.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	18.24	U1 All	OK
		11.833	0.00	0.00	0.00	U1 All	OK	1.20	30.37	16.40	U1 All	OK
		12.833	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	9.76	U1 All	OK
		14.000	1.20	-30.37	-0.05	U1 All	OK	1.20	30.37	0.00	U1 All	OK
		15.000	2.20	-54.64	-13.33	U1 All	OK	1.20	30.37	0.00	U1 All	OK
		17.333	2.20	-54.64	-50.21	U1 All	OK	1.20	30.37	0.00	U1 All	OK
		17.778	2.20	-54.64	-58.08	U1 All		1.20	30.37	0.00	U1 All	
		18.000	2.20	-54.64	-62.11	U1 All		1.20	30.37	0.00	U1 All	
Midd	dle	0.000	1.20	-30.37	0.47	U1 All		1.20	30.37	0.00	U1 All	
		0.667	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	0.00	U1 All	OK
		1.662	1.20	-30.37	-0.20	U1 All	OK	1.20	30.37	0.00	U1 All	OK
		3.333	1.20	-30.37	0.00		OK	1.20	30.37	4.91		OK
		4.333	0.00	0.00	0.00		OK	1.20	30.37	9.67		OK
		0.500	0.00	0.00	0.00		OK	1.20	30.37	10.20		OK
		0.129	0.00	0.00	0.00		OK	1.20	30.37	17.91		OK
		11 500	0.00	0.00	0.00		OK	1.20	20.27	12.16		OK
		12 800	0.00	0.00	0.00		OK	1.20	30.37	6.63		OK
		13 809	1.20	-30.37	0.00		OK	1.20	30.37	1 15		OK
		17 333	1.20	-30.37	-16 74		OK	1.20	30.37	0.00		OK
		18.000	1.20	-30.37	-21.82	U1 All		1.20	30.37	0.00	U1 All	
2 Colu		0.000	2.20	E4 64	57 10	114 .0.1		1.00	20.27	0.00	111 41	
3 Colu	ar1111	0.000	2.20	-54.64	-57.18			1.20	30.37	0.00		OK
		3.000	2.20	-54.64	-45.47		OK	1.20	30.37	0.00		OK
		3.000	1.20	-30.37	-11.00		OK	1.20	30.37	0.00		OK
		5 167	1.20	-30.37	0.40		OK	1.20	30.37	8.00		OK
		6 167	0.00	0.00	0.00		OK	1.20	30.37	13.40		OK
		6 500	0.00	0.00	0.00		OK	1.20	30.37	14 84		OK
		9 000	0.00	0.00	0.00		OK	1.20	30.37	19.90		OK




				Тор					Bott	om	
Span Strip	x	$A_{s,top}$	ΦM _n -	M u-	Comb Pat	Status	A _{s,bot}	ΦM _n +	M _u +	Comb Pat	Status
	ft	in²	kip-ft	kip-ft			in²	kip-ft	kip-ft		
	9.124	0.00	0.00	0.00	U1 All	OK	1.20	30.37	19.90	U1 All	OK
	11.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	14.84	U1 All	OK
	11.833	0.00	0.00	0.00	U1 All	OK	1.20	30.37	13.40	U1 All	OK
	12.833	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	8.00	U1 All	OK
	14.000	1.20	-30.37	-0.45	U1 All	OK	1.20	30.37	0.00	U1 All	OK
	15.000	2.20	-54.64	-11.60	U1 All	OK	1.20	30.37	0.00	U1 All	OK
	17.333	2.20	-54.64	-45.47	U1 All	OK	1.20	30.37	0.00	U1 All	OK
	18.000	2.20	-54.64	-57.18	U1 All		1.20	30.37	0.00	U1 All	
Middle	0.000	1.20	-30.37	-19.06	U1 All		1.20	30.37	0.00	U1 All	
	0.667	1.20	-30.37	-15.16	U1 All	OK	1.20	30.37	0.00	U1 All	OK
	4.191	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	0.77	U1 All	OK
	5.191	0.00	0.00	0.00	U1 All	OK	1.20	30.37	5.43	U1 All	OK
	6.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	9.89	U1 All	OK
	9.000	0.00	0.00	0.00	U1 All	OK	1.20	30.37	13.27	U1 All	OK
	9.124	0.00	0.00	0.00	U1 All	OK	1.20	30.37	13.27	U1 All	OK
	11.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	9.89	U1 All	OK
	12.809	0.00	0.00	0.00	U1 All	OK	1.20	30.37	5.43	U1 All	OK
	13.809	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	0.77	U1 All	OK
	17.333	1.20	-30.37	-15.16	U1 All	ок	1.20	30.37	0.00	U1 All	OK
	18.000	1.20	-30.37	-19.06	U1 All		1.20	30.37	0.00	U1 All	
1. Oalisses	0.000	0.00	54.04	00.44	114 41		4.00	00.07	0.00	114 411	
4 Column	0.000	2.20	-54.64	-62.11			1.20	30.37	0.00		
	0.222	2.20	-54.64	-58.08	U1 All		1.20	30.37	0.00		
	0.667	2.20	-54.64	-50.22	U1 All	OK	1.20	30.37	0.00	U1 All	OK
	3.000	2.20	-54.64	-13.34	U1 All	OK	1.20	30.37	0.00		OK
	4.000	1.20	-30.37	-0.06	UT AII	OK	1.20	30.37	0.00		OK
	5.167	1.20	-30.37	0.00	UTAI	OK	1.20	30.37	9.76	UTAI	OK
	6.167	0.00	0.00	0.00	U1 All	OK	1.20	30.37	16.40	U1 All	OK
	6.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	18.24	U1 All	OK
	9.000	0.00	0.00	0.00	U1 All	OK	1.20	30.37	26.40	U1 All	OK
	9.8/1	0.00	0.00	0.00	U1 All	OK	1.20	30.37	26.87	U1 All	OK
	11.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	24.42		OK
	11.833	0.00	0.00	0.00	U1 All	OK	1.20	30.37	23.40		OK
	12.833	1.20	-30.37	0.00	U1 All	OK	1.20	30.37	19.23		OK
	14.000	1.20	-30.37	0.00		OK	1.20	30.37	12.31		OK
	15.000	1.40	-35.30	0.00	U1 All	OK	1.20	30.37	4.62	U1 All	OK
	17.333	1.40	-35.30	-32.66	U1 All	OK	1.20	30.37	0.00	U1 All	OK
	17.556	1.40	-35.30	-37.39	U1 All		1.20	30.37	0.00	UT AII	
A C dalla	18.000	1.40	-35.30	-47.36	U1 All		1.20	30.37	0.00	U1 All	
Middle	0.000	1.20	-30.37	-21.82	U1 All		1.20	30.37	0.00	U1 All	
	0.007	1.20	-30.37	-16.74		OK	1.20	30.37	0.00		OK
	4.191	1.20	-30.37	0.00		OK	1.20	30.37	1.14		OK
	5.191	0.00	0.00	0.00		OK	1.20	30.37	10.02		OK
	6.500	0.00	0.00	0.00	U1 All	OK	1.20	30.37	12.16		OK
	9.000	0.00	0.00	0.00		OK	1.20	30.37	17.60		OK
	9.8/1	0.00	0.00	0.00		OK	1.20	30.37	17.91		OK
	12.007	0.00	0.00	0.00		OK	1.20	30.37	10.28		OK
	13.007	1.00	0.00	0.00		OK	1.20	30.37	9.67		OK
	14.00/	1.20	-30.37	0.00		OK	1.20	30.37	4.91		OK
	17 222	1.20	-30.37	-0.20		OK	1.20	30.37	0.00		OK
	17.333	1.20	-30.37	0.00		UK	1.20	30.37	0.00		UK
	16.000	1.20	-30.37	0.47	UT All)?	1.20	30.37	0.00	UTAI	
5 Column	0.000	1.40	-35.30	-0.60	U1 All		0.00	0.00	0.00	U1 All	



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				Тор					Bott	om		
Span Strip	x	$A_{s,top}$	ФM _n -	M u-	Comb Pat	Status	A _{s,bot}	ФМ _n +	M _u +	Comb Pat	Status	
	ft	in²	kip-ft	kip-ft			in²	kip-ft	kip-ft			
	0.117	1.40	-35.30	-0.43	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.309	1.40	-35.30	-0.19	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.333	1.40	-35.30	-0.17	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.474	1.40	-35.30	-0.06	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.667	1.40	-35.30	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
Middle	0.000	1.20	-30.37	0.00	U1 All		0.00	0.00	0.00	U1 All		
	0.117	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.309	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.333	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.474	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK	
	0.667	1.20	-30.37	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK	

2.9. Slab Shear Capacity

Span	b	d	V _{ratio}	ΦVc	Vu	Xu
	in	in		kips	kips	ft
1	168.00	5.75	1.000	91.64	0.00	0.00
2	168.00	5.75	1.000	91.64	23.28	16.85
3	168.00	5.75	1.000	91.64	21.22	1.15
4	168.00	5.75	1.000	91.64	23.28	1.15
5	168.00	5.75	1.000	91.64	0.00	0.00

2.10. Flexural Transfer of Negative Unbalanced Moment at Supports

Support	Width	Width-c	d	M _{unb} Com	b Patt	¥٢	A _{s,req}	A _{s,prov}	Add Bars
	in	in	in	kip-ft			in²	in²	
1	37.00	37.00	5.75	46.30 U1	All	0.617	1.159	0.617	3-#4
2	37.00	37.00	5.75	7.69 U1	All	0.600	0.180	0.969	
3	37.00	37.00	5.75	7.69 U1	All	0.600	0.180	0.969	
4	37.00	37.00	5.75	46.30 U1	All	0.617	1.159	0.617	3-#4

2.11. Punching Shear Around Columns

2.11.1. Critical Section Properties

Support	Туре	b ₁	b ₂	b ₀	d _{avg}	CG	C _(left)	C(right)	Ac	Jc
		in	in	in	in	in	in	in	in²	in4
1	Rect	18.88	21.75	59.50	5.75	4.89	12.89	5.99	342.12	14109.47
2	Rect	21.75	21.75	87.00	5.75	0.00	10.88	10.88	500.25	40130.73
3	Rect	21.75	21.75	87.00	5.75	0.00	10.88	10.88	500.25	40130.73
4	Rect	18.88	21.75	59.50	5.75	-4.89	5.99	12.89	342.12	14109.47

2.11.2. Punching Shear Results

Support	Vu	Vu	M _{unb}	Comb	Patt	Y٧	Vu	ΦVc
	kips	psi	kip-ft				psi	psi
1	23.51	68.7	36.72	U1	All	0.383	140.4	189.7
2	50.06	100.1	-7.69	U1	All	0.400	110.1	189.7
3	50.06	100.1	7.69	U1	All	0.400	110.1	189.7
4	23.51	68.7	-36.72	U1	All	0.383	140.4	189.7

2.12. Material TakeOff

2.12.1. Reinforcement in the Direction of Analysis

Top Bars 336.6 lb <=> 6.08 lb/ft <=> 0.435 lb/ft²



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Bottom Bars	432.9	lb	<=>	7.82	lb/ft	<=>	0.559	lb/ft ²
Stirrups	0.0	lb	<=>	0.00	lb/ft	<=>	0.000	lb/ft ²
Total Steel	769.5	lb	<=>	13.91	lb/ft	<=>	0.993	lb/ft ²
Concrete	451.9	ft ³	<=>	8.17	ft³/ft	<=>	0.583	ft³/ft²

3. Deflection Results: Summary

3.1. Section Properties

3.1.1. Frame Section Properties

Notes:

M+ve values are for positive moments (tension at bottom face). M-ve values are for negative moments (tension at top face).

	M.	+ve		M	-ve	
Span Zone	lg	I _{cr}	M _{cr}	l _g	I _{cr}	M _{cr}
	in⁴	in⁴	kip-ft	in4	in⁴	kip-ft
1 Left	4802	0	54.23	4802	466	-54.23
Midspan	4802	0	54.23	4802	499	-54.23
Right	4802	0	54.23	4802	499	-54.23
2 Left	4802	466	54.23	4802	499	-54.23
Midspan	4802	466	54.23	4802	0	-54.23
Right	4802	466	54.23	4802	629	-54.23
3 Left	4802	466	54.23	4802	629	-54.23
Midspan	4802	466	54.23	4802	0	-54.23
Right	4802	466	54.23	4802	629	-54.23
4 Left	4802	466	54.23	4802	629	-54.23
Midspan	4802	466	54.23	4802	0	-54.23
Right	4802	466	54.23	4802	499	-54.23
5 Left	4802	0	54.23	4802	499	-54.23
Midspan	4802	0	54.23	4802	499	-54.23
Right	4802	0	54.23	4802	466	-54.23

3.1.2. Frame Effective Section Properties

				Load Lev	el			
		Dead		Sustaine	d	Dead+Liv	+Live	
Span Zone	Weight	M _{max}	l _e	M _{max}	l _e	M _{max}	l _e	
		kip-ft	in⁴	kip-ft	in⁴	kip-ft	in⁴	
1 Right	1.000	-0.33	4802	-0.33	4802	-0.46	4802	
Span Avg		0000	4802	10000	4802		4802	
2 Middle	0.850	24.94	4802	24.94	4802	34.23	4802	
Right	0.150	-46.75	4802	-46.75	4802	-64.15	3151	
Span Avg			4802		4802		4554	
3 Left	0.150	-42.47	4802	-42.47	4802	-58.27	3994	
Middle	0.700	18.47	4802	18.47	4802	25.35	4802	
Right	0.150	-42.47	4802	-42.47	4802	-58.27	3994	
Span Avg			4802		4802		4560	
4 Left	0.150	-46.75	4802	-46.75	4802	-64.15	3151	
Middle	0.850	24.94	4802	24.94	4802	34.23	4802	
Span Avg		1000	4802		4802	1 <u></u>	4554	
5 Left	1.000	-0.33	4802	-0.33	4802	-0.46	4802	
Span Avg			4802		4802		4802	



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3.1.3. Strip Section Properties at Midspan

Notes: Load distribution factor, LDL, averages moment distribution factors listed in Design Results. Ratio refers to proportion of strip to frame deflections under fix-end condtions.

	Col	umn Strip		Middle Strip				
Span	l _g in⁴	LDF	Ratio	l _g in⁴	LDF	Ratio		
1	2401	0.800	1.600	2401	0.200	0.400		
2	2401	0.738	1.475	2401	0.262	0.525		
3	2401	0.675	1.350	2401	0.325	0.650		
4	2401	0.738	1.475	2401	0.262	0.525		
5	2401	0.800	1.600	2401	0.200	0.400		

3.2. Instantaneous Deflections

3.2.1. Extreme Instantaneous Frame Deflections and Corresponding Locations

						Live		Tot	al
Span	Direction	Value	Units	Dead	Sustained	Unsustained	Total	Sustained	Dead+Live
1	Down	Def	in						
		Loc	ft						
	Up	Def	in	-0.004		-0.002	-0.002	-0.004	-0.006
		Loc	ft	0.000		0.000	0.000	0.000	0.000
2	Down	Def	in	0.055		0.023	0.023	0.055	0.078
		Loc	ft	8.378		8.378	8.378	8.378	8.378
	Up	Def	in						
		Loc	ft						
3	Down	Def	in	0.032		0.014	0.014	0.032	0.046
		Loc	ft	8.876		8.876	8.876	8.876	8.876
	Up	Def	in	0.000		0.000	0.000	0.000	0.000
		Loc	ft	0.444		0.444	0.444	0.444	0.444
4	Down	Def	in	0.055		0.023	0.023	0.055	0.078
		Loc	ft	9.622		9.622	9.622	9.622	9.622
	Up	Def	in						
		Loc	ft						
5	Down	Def	in						
		Loc	ft						
	Up	Def	in	-0.004		-0.002	-0.002	-0.004	-0.006
		Loc	ft	0.667		0.667	0.667	0.667	0.667

3.2.2. Extreme Instantaneous Column Strip Deflections and Corresponding Locations

						Live		Total		
Span	Direction	Value	Units	Dead	Sustained	Unsustained	Total	Sustained	Dead+Live	
1	Down	Def	in							
		Loc	ft							
	Up	Def	in	-0.004		-0.002	-0.002	-0.004	-0.006	
		Loc	ft	0.000		0.000	0.000	0.000	0.000	
2	Down	Def	in	0.072		0.031	0.031	0.072	0.103	
		Loc	ft	8.627		8.627	8.627	8.627	8.627	
	Up	Def	in							
		Loc	ft							
3	Down	Def	in	0.045		0.019	0.019	0.045	0.064	
		Loc	ft	8.876		8.876	8.876	8.876	8.876	
	Up	Def	in	0.000		0.000	0.000	0.000	0.000	
		Loc	ft	0.222		0.222	0.222	0.222	0.222	
4	Down	Def	in	0.072		0.031	0.031	0.072	0.103	
		Loc	ft	9.373		9.373	9.373	9.373	9.373	



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												Live		То	tal
Span	Direction	Value	Units	Dead	Sustained Un	sustained	Total	Sustained	Dead+Live						
	Up	Def	in												
		Loc	ft												
5	Down	Def	in		2227										
		Loc	ft		10000										
	Up	Def	in	-0.004	- <u></u>	-0.002	-0.002	-0.004	-0.006						
		Loc	ft	0.667		0.667	0.667	0.667	0.667						

3.2.3. Extreme Instantaneous Middle Strip Deflections and Corresponding Locations

						Live		Total		
Span	Direction	Value	Units	Dead	Sustained	Unsustained	Total	Sustained	Dead+Live	
1	Down	Def	in							
		Loc	ft							
	Up	Def	in	-0.004		-0.002	-0.002	-0.004	-0.006	
		Loc	ft	0.000		0.000	0.000	0.000	0.000	
2	Down	Def	in	0.038		0.016	0.016	0.038	0.054	
		Loc	ft	7.881		8.129	8.129	7.881	8.129	
	Up	Def	in							
		Loc	ft							
3	Down	Def	in	0.019		0.008	0.008	0.019	0.027	
		Loc	ft	8.876		8.876	8.876	8.876	8.876	
	Up	Def	in	0.000		0.000	0.000	0.000	0.000	
		Loc	ft	0.667		0.667	0.667	0.667	0.667	
4	Down	Def	in	0.038		0.016	0.016	0.038	0.054	
		Loc	ft	9.871		9.871	9.871	9.871	9.871	
	Up	Def	in							
		Loc	ft							
5	Down	Def	in							
		Loc	ft							
	Up	Def	in	-0.004		-0.002	-0.002	-0.004	-0.006	
		Loc	ft	0.667		0.667	0.667	0.667	0.667	

3.3. Long-term Deflections

3.3.1. Long-term Column Strip Deflection Factors

Notes:

Deflection multiplier, Lambda, depends on moment sign at sustained load level and Rho' in given zone. Rho' is assumed zero because Compression Reinforcement option is NOT selected in Solve Options.

Time dependant factor for sustained loads = 2.000

		M.	lve			M. _{ve}				
Span Zone	A _{s,top}	b	d	Rho'	Lambda	A _{s,bot}	b	d	Rho'	Lambda
	in²	in	in	%		in²	in	in	%	
1 Right				0.000	2.000				0.000	2.000
2 Midspan				0.000	2.000				0.000	2.000
3 Midspan				0.000	2.000				0.000	2.000
4 Midspan				0.000	2.000				0.000	2.000
5 Left				0.000	2.000				0.000	2.000



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3.3.2. Long-term Middle Strip Deflection Factors

Notes:

Poflection multiplier, Lambda, depends on moment sign at sustained load level and Rho' in given zone. Rho' is assumed zero because Compression Reinforcement option is NOT selected in Solve Options.

Time dependant factor for sustained loads = 2.000

		М.	ve				М.	ve		
Span Zone	A _{s,top}	b	d	Rho'	Lambda	A _{s,bot}	b	d	Rho'	Lambda
	in²	in	in	%		in²	in	in	%	
1 Right				0.000	2.000				0.000	2.000
2 Midspan				0.000	2.000				0.000	2.000
3 Midspan		1000		0.000	2.000				0.000	2.000
4 Midspan	12222	100000		0.000	2.000		1.000		0.000	2.000
5 Left				0.000	2.000				0.000	2.000

3.3.3. Extreme Long-term Column Strip Deflections and Corresponding Locations

Notes:

Incremental deflections due to creep and shrinkage (cs) based on sustained load level values. Incremental deflections after partitions are installed can be estimated by deflections due to: - creep and shrinkage plus unsustained live load (cs+lu), if live load applied before partitions, - creep and shrinkage plus live load (cs+l), if live load applied after partitions. Total deflections consist of dead, live, and creep and shrinkage deflections.

Span	Direction	Value	Units	cs	cs+lu	cs+l	Total
1	Down	Def	in				
		Loc	ft				
	Up	Def	in	-0.009	-0.011	-0.011	-0.015
		Loc	ft	0.000	0.000	0.000	0.000
2	Down	Def	in	0.145	0.176	0.176	0.248
		Loc	ft	8.627	8.627	8.627	8.627
	Up	Def	in				
		Loc	ft				
3	Down	Def	in	0.089	0.109	0.109	0.153
		Loc	ft	8.876	8.876	8.876	8.876
	Up	Def	in	0.000	0.000	0.000	0.000
		Loc	ft	0.222	0.222	0.222	0.222
4	Down	Def	in	0.145	0.176	0.176	0.248
		Loc	ft	9.373	9.373	9.373	9.373
	Up	Def	in				
		Loc	ft				
5	Down	Def	in				
		Loc	ft				
	Up	Def	in	-0.009	-0.011	-0.011	-0.015
		Loc	ft	0.667	0.667	0.667	0.667

3.3.4. Extreme Long-term Middle Strip Deflections and Corresponding Locations

Notes: Incremental deflections due to creep and shrinkage (cs) based on sustained load level values. Incremental deflections after partitions are installed can be estimated by deflections due to: - creep and shrinkage plus unsustained live load (cs+lu), if live load applied before partitions, - creep and shrinkage plus live load (cs+l), if live load applied after partitions. Total deflections consist of dead, live, and creep and shrinkage deflections.

Span	Direction	Value	Units	cs	cs+lu	cs+l	Total
1	Down	Def	in				
		Loc	ft				
	Up	Def	in	-0.009	-0.011	-0.011	-0.015
		Loc	ft	0.000	0.000	0.000	0.000



Span	Direction	Value	Units	CS	cs+lu	cs+l	Total
2	Down	Def	in	0.076	0.091	0.091	0.129
		Loc	ft	8.129	8.129	8.129	8.129
	Up	Def	in		(<u>4-4</u> 3		
		Loc	ft				
3	Down	Def	in	0.038	0.046	0.046	0.065
		Loc	ft	8.876	8.876	8.876	8.876
	Up	Def	in	-0.001	-0.001	-0.001	-0.001
		Loc	ft	0.667	0.667	0.667	0.667
4	Down	Def	in	0.076	0.091	0.091	0.129
		Loc	ft	9.871	9.871	9.871	9.871
	Up	Def	in				
		Loc	ft				
5	Down	Def	in				
		Loc	ft				
	Up	Def	in	-0.009	-0.011	-0.011	-0.015
		Loc	ft	0.667	0.667	0.667	0.667



4. Screenshots



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4.1. Extrude 3D view Car 3 32 00 5 Two-Way Flat Plate Floor Project: Diagram: Model View (Extrude) Slabs





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4.2. Loads - Case A - SELF







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4.3. Loads - Case B - Dead







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4.4. Loads - Case C - Live







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4.5. Internal Forces







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4.6. Moment Capacity







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4.7. Shear Capacity







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4.8. Deflection -0.078 |||1 | 1 1 1 I 1 1 1 1 1 1 I 1 1 Instantaneous Deflection - in 1 _ 1 1 | -N 1 _ 1 1 _ 1 1 1 -1 י ן 1 -I 1 -1 1 _ ; 1 I 1 | 1 i | 1 | | | 1 -0.078

 Legend:
 —
 Dead Load
 —
 Live Load

 —
 Sustained Load
 —
 Total Deflection

 Diagram:
 Deflection





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4.9. Reinforcement - <u>6-#4(8.0)</u>c 6-#4(62.3) 6-#4(62.3) 6-#4(52.0) 6-#4(62.3) #4(62.3) 6-#4(52.0) 1 1 1 1 1 1 1 1 + 1 | 1 | 6-#4(216.0)c 6-#4(216.0)c 6-#4(216.0)c | | 1 1 | | I. I. 1 1 Middle Strip Flexural Reinforcement 1 1 -) + 6-#4(8.8)c -1-#4(48.0) 6-#4(74.0) 6-#4(74.0) 5-#4(48.0) 1 5-#4(48.0) 6-#4(74.0) 6-#4(74.0) -5-#4(48.0) 5-#4(48.0) 6-#4(74.0) 6-#4(74.0) -1-#4(48.0) 1 1 1 Ť 1 1 + 1 | | | | 1 6-#4(216.0)c 6-#4(216.0)c 6-#4(216.0)c 1 i I L l i I 1 Column Strip Flexural Reinforcement Legend: - Continuous Discontinuous Project: Two-Way Flat Plate Floor Diagram: Reinforcement





Table 12 – Comp	parison of Moments obtained from Hand	d (EFM) and spSlab Solu	ition (ft-kips)							
		Hand (EFM)	spSlab							
	Exterior Span									
	Exterior Negative*	32.66								
Column Strip	Positive	26.96	26.87							
	Interior Negative [*]	50.24	50.21							
	Exterior Negative*	0.00	0.00							
Middle Strip	Positive	17.98	17.91							
	Interior Negative [*]	16.75	16.74							
	Interior Span									
Column Strin	Interior Negative*	45.46	45.47							
Column Strip	Positive	19.94	19.90							
Middle Strin	Interior Negative*	15.15	15.16							
which is strip	Positive	13.29	13.27							
* Negative moments are	taken at the faces of supports									

8. Summary and Comparison of Two-Way Slab Design Results

	Table 13 – Comparison of Reinforcement Results with Hand and spSlab Solution										
Span I	Location	Reinforcem for F	ement Provided Flexure Additional Reinforcement Provided for Unbalance Moment Transfer*			Total Reinforcement Provided					
		Hand	spSlab	Hand	spSlab	Hand	spSlab				
Exterior Span											
Exterior Negative		7 – #4	7 – #4	3-#4	3-#4	10-#4	10-#4				
Column Strip	Positive	6-#4	6-#4	n/a	n/a	6-#4	6-#4				
	Interior Negative	11-#4	11-#4			11-#4	11 – #4				
	Exterior Negative	6 – #4	6-#4	n/a	n/a	6-#4	6-#4				
Middle Strin	Positive	6-#4	6-#4	n/a	n/a	6-#4	6-#4				
omp	Interior Negative	6-#4	6-#4	n/a	n/a	6-#4	6-#4				
			Interior	Span							
Column Strip Positive		6-#4	6-#4	n/a	n/a	6-#4	6-#4				
Middle Strip	Positive	6-#4	6-#4	n/a	n/a	6-#4	6-#4				
* In the EFM, the unbalanced moment at the support centerline is used to determine the value of the additional reinforcement as compared with DDM using the moments at the face of support.											

Table 14 – Comparison of One-Way (Beam Action) Shear Check Results Using Hand and spSlab Solution										
Snan	Vu	(kips)	xu	[•] (in.)	ϕV_c (kips)					
Span	Hand	spSlab	Hand	spSlab	Hand	spSlab				
Exterior	23.29	23.28	16.85	16.85	91.64	91.64				
Interior 21.22 21.22 1.15 1.15 91.64 91.64										
x_u calculated from the centerline of the left column for each span										

	Table 15 – Comparison of Two-Way (Punching) Shear Check Results Using Hand and spSlab Solution											
Sunnant	b 1	(in.)	b ₂ (b ₂ (in.)		b ₀ (in.)		A_c (in. ²)		kips)	v_u (psi)	
Support	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab
Exterior	18.88	18.88	21.75	21.75	59.50	59.50	342.13	342.13	21.70	23.51	63.43	68.70
Interior	21.75	21.75	21.75	21.75	87.00	87.00	500.25	500.25	50.08	50.06	100.10	100.10
Sunnant	САВ	(in.)	J_c (in. ⁴)		γv	Munb (kips-ft)	v _u (psi)	ϕv_c	(psi)
Support	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab
Exterior	5.99	5.99	14,109	14,109	0.383	0.383	37.81	36.72	137.20	140.40	189.74	189.70
Interior	10.88	10.88	40,131	40,131	0.400	0.400	-7.70	-7.69	110.12	110.10	189.74	189.70



Table 16 – Comparison of Immediate Deflection Results Using Hand and spSlab Solution (in.)								
Column Strip								
Span	D		D+LL _{sus}		D+LL _{full}		LL	
	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab
Exterior	0.075	0.072	0.075	0.072	0.104	0.103	0.030	0.031
Interior	0.047	0.045	0.047	0.045	0.063	0.064	0.016	0.019
Middle Strip								
Span	D		D+LL _{sus}		D+LL _{full}		LL	
	Hand	spSlab	Hand	spSlab	Hand	spSlab	Hand	spSlab
Exterior	0.038	0.038	0.038	0.038	0.054	0.054	0.016	0.016
Interior	0.020	0.019	0.020	0.019	0.026	0.027	0.006	0.008

Table 17 – Comparison of Time-Dependent Deflection Results Using Hand and spSlab Solution								
Column Strip								
Span		λ_{Δ}	Δ_{cs}	(in.)	Δ_{total} (in.)			
	Hand	spSlab	Hand	spSlab	Hand	spSlab		
Exterior	2.0	2.0	0.149	0.145	0.254	0.248		
Interior	2.0	2.0	0.094	0.089	0.157	0.153		
Middle Strip								
Snon	λ_{Δ}		Δ_{cs} (in.)		Δ_{total} (in.)			
Span	Hand	spSlab	Hand	spSlab	Hand	spSlab		
Exterior	2.0	2.0	0.076	0.076	0.130	0.129		
Interior	2.0	2.0	0.040	0.038	0.066	0.065		

In all of the hand calculations illustrated above, the results are in close or exact agreement with the automated analysis and design results obtained from the <u>spSlab</u> model.



9. Comparison of Two-Way Slab Analysis and Design Methods

A slab system can be analyzed and designed by any procedure satisfying equilibrium and geometric compatibility. Three established methods are widely used. The requirements for two of them are described in detail in <u>ACI 318-</u> <u>14 Chapter 8 (8.2.1).</u>

Direct Design Method (DDM) is an approximate method and is applicable to flat plate concrete floor systems that meet the stringent requirements of <u>ACI 318-14 (8.10.2)</u>. In many projects, however, these requirements limit the usability of the DDM significantly.

The Equivalent Frame Method (EFM) does not have the limitations of DDM. It requires more accurate analysis methods that, depending on the size and geometry, can prove to be long, tedious, and time-consuming.

StucturePoint's <u>spSlab</u> software program solution utilizes the EFM to automate the process providing considerable time-savings in the analysis and design of two-way slab systems as compared to hand solutions using DDM or EFM.

Finite Element Method (FEM) is another method for analyzing reinforced concrete slabs, particularly useful for irregular slab systems with variable thicknesses, openings, and other features not permissible in DDM or EFM. Many reputable commercial FEM analysis software packages are available on the market today such as <u>spMats</u>. Using FEM requires critical understanding of the relationship between the actual behavior of the structure and the numerical simulation since this method is an approximate numerical method. The method is based on several assumptions and the operator has a great deal of decisions to make while setting up the model and applying loads and boundary conditions. The results obtained from FEM models should be verified to confirm their suitability for design and detailing of concrete structures.

The following table shows a general comparison between the DDM, EFM and FEM. This table covers general limitations, drawbacks, advantages, and cost-time efficiency of each method where it helps the engineer in deciding which method to use based on the project complexity, schedule, and budget.

Structure Point

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ACI 318- 14 Provision 8.10.2.1 8.10.2.2	Limitations/Applicability Minimum of three continuous spans in each direction	DDM (Hand)	EFM (Hand// <u>spSlab</u>)	FEM	
8.10.2.1	Minimum of three continuous spans in each direction			(<u>spMats</u>)	
8.10.2.2					
	Successive span lengths measured center-to- center of supports in each direction shall not differ by more than one-third the longer span				
8.10.2.3	Panels shall be rectangular, with ratio of longer to shorter panel dimensions, measured center-to-center supports, not exceed 2.		Ŋ		
8.10.2.4	Column offset shall not exceed 10% of the span in direction of offset from either axis between centerlines of successive columns				
8.10.2.5	All loads shall be due to gravity only	M			
8.10.2.5	All loads shall be uniformly distributed over an entire panel (q_u)	Ø			
8.10.2.6	Unfactored live load shall not exceed two times the unfactored dead load	Σ			
8.10.2.7	For a panel with beams between supports on all sides, slab-to-beam stiffness ratio shall be satisfied for beams in the two perpendicular directions.	Ŋ			
8.7.4.2	Structural integrity steel detailing	\checkmark	Ø	Ø	
8.5.4	Openings in slab systems	\checkmark	Ø	Ŋ	
8.2.2	Concentrated loads	Not permitted	V	Ø	
8.11.1.2	Live load arrangement (Load Patterning)	Not required	Required	Engineering judgment required based on modeling technique	
R8.10.4.5*	Reinforcement for unbalanced slab moment transfer to column (M_{sc})	Moments @ support face	Moments @ support centerline	Engineering judgment required based on modeling technique	
Irregularities (i.e. variable thickness, non-prismatic, partial bands, mixed systems, support arrangement, etc.)		Not permitted	Engineering judgment required	Engineering judgment required	
Complexity		Low	Average	Complex to very complex	
Design time/cos	osts	Fast	Limited	Unpredictable/Costly	
Design Economy		Conservative (see detailed comparison with <u>spSlab</u> output)	Somewhat conservative	Unknown - highly dependent on modeling assumptions: 1. Linear vs. non-linear 2. Isotropic vs non-isotropic 3. Plate element choice 4. Mesh size and aspect ratio	
General (Drawbacks)		Very limited applications	Limited geometry	5. Design & detailing features Limited guidance non-standard application (user dependent). Required significant engineering judgment	
General (Advantages)		Very limited analysis is required	Detailed analysis is required or via software (e.g. <u>spSlab</u>)	Unlimited applicability to handle complex situations permissible by the features of the software used (e.g. <u>spMats</u>)	

joint. In DDM only moments at the face of the support are calculated and is used, moments at the column center line are used to obtain $M_{\rm sc}(M_{\rm unb})$.