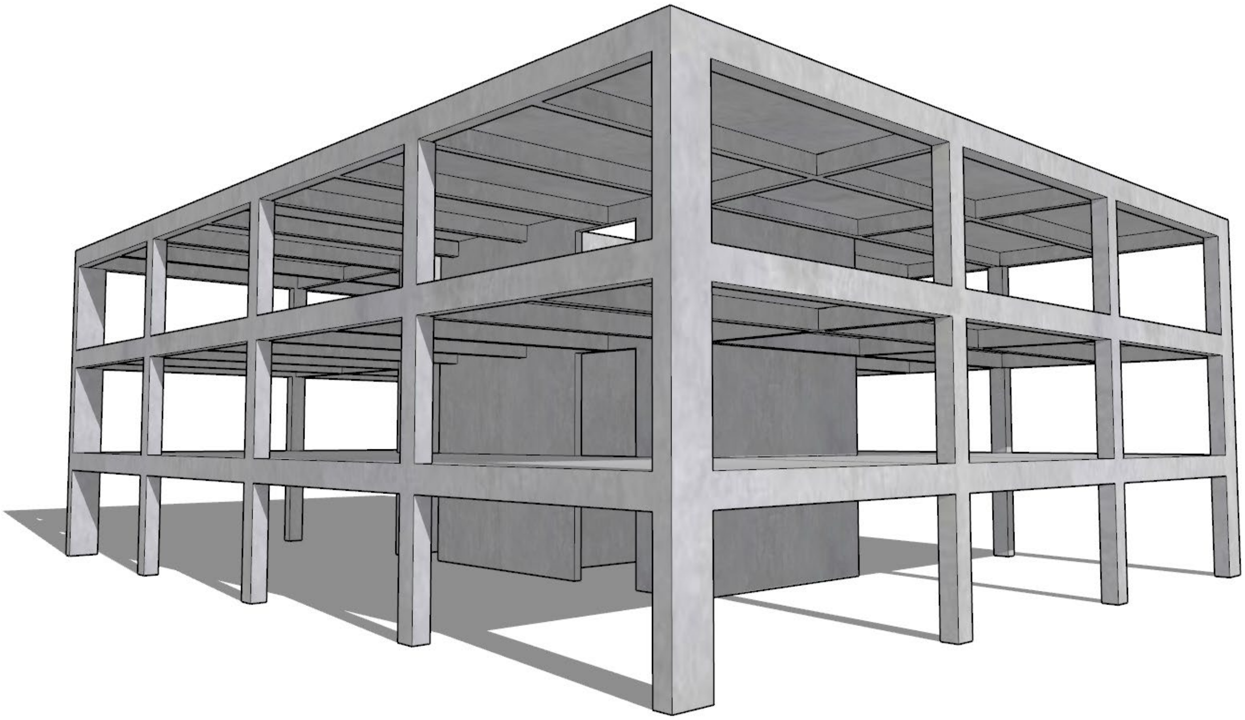
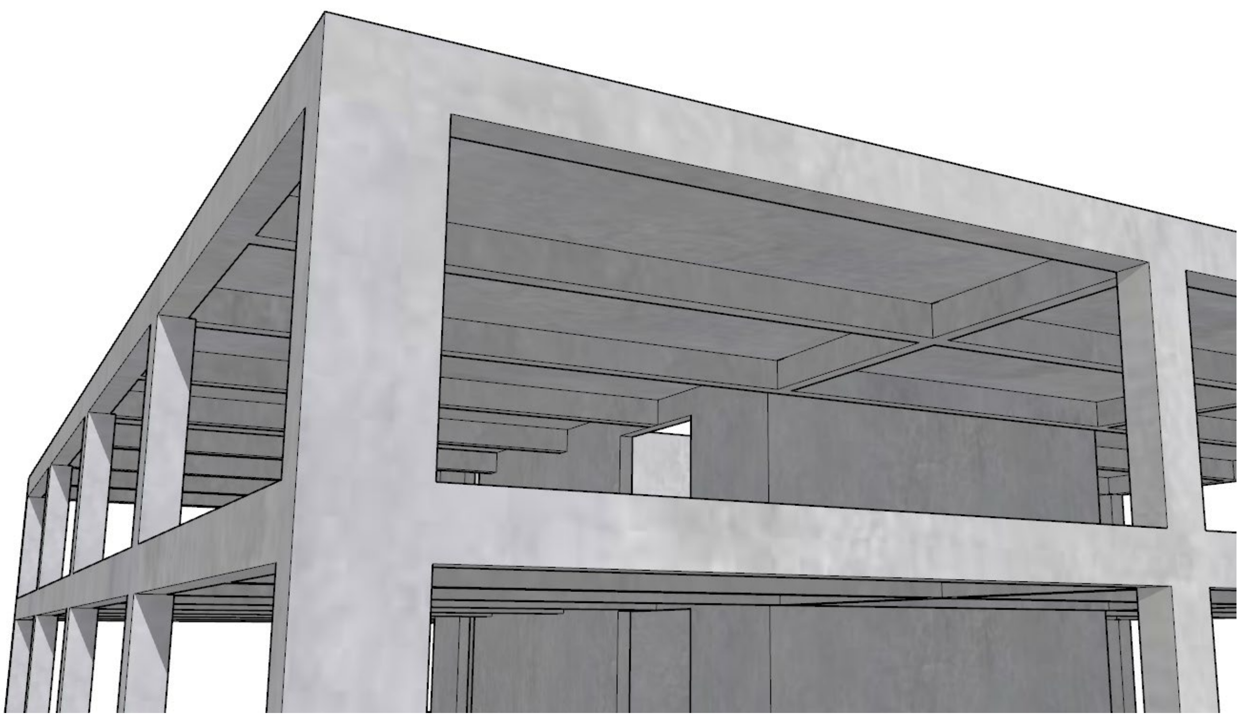
One-Way Slab Analysis and Design (ACI 318-14)





One-Way Slab Analysis and Design (ACI 318-14)

A one-way slab is a type of reinforced concrete floor system designed to carry loads predominantly in one direction. Typically, the ratio of the longer span length to the shorter span length in this system is greater than or equal to 2. In a one-way slab system, the reinforcement is primarily provided in the shorter span direction to resist the applied loads. However, minimum reinforcement is provided in the longer span direction to account for temperature and shrinkage effects.

This example will demonstrate the analysis and design of the one-way reinforced concrete slab shown below using ACI 318-14 provisions. Steps of the structural analysis, flexural design, shear design, and deflection checks will be presented. The results of hand calculations are then compared with the reference results and numerical analysis results obtained from the spBeam engineering software program by StructurePoint.

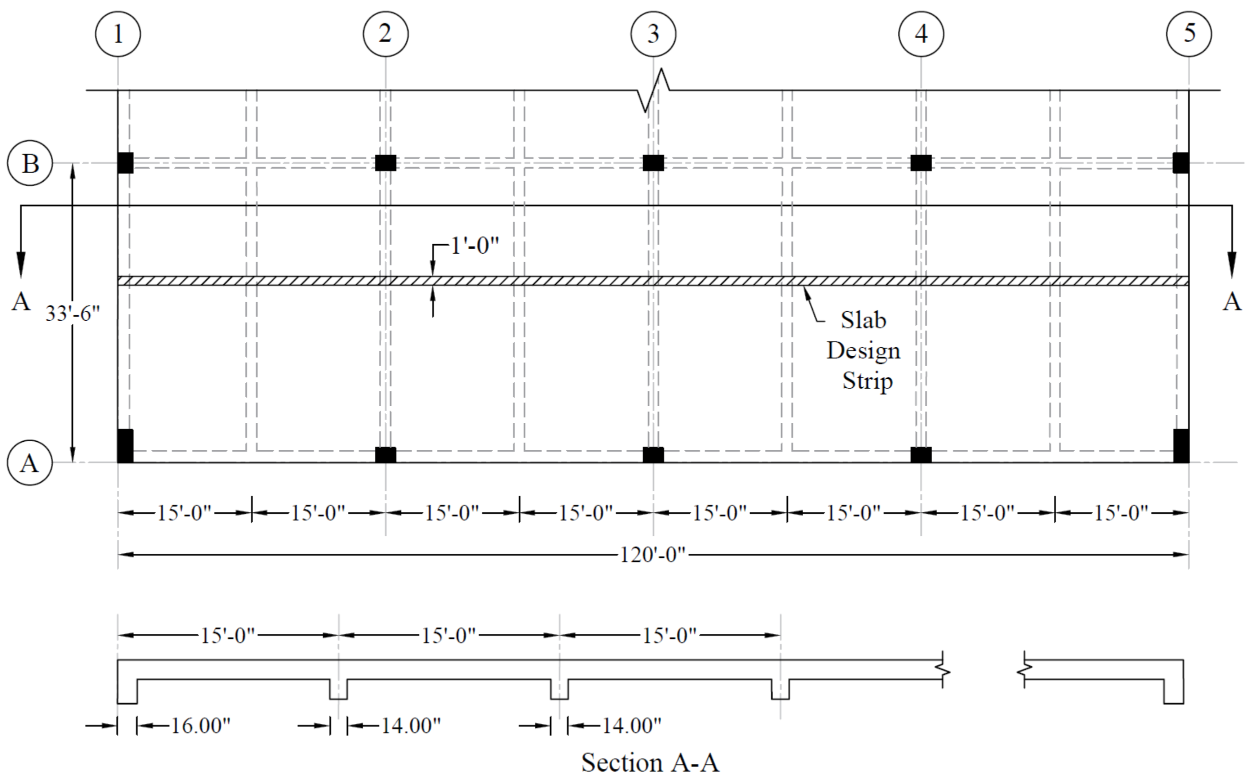


Figure 1 – Continuous One-Way Concrete Slab Floor System

Contents

Code

Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)

References

- Reinforced Concrete Mechanics and Design, 7th Edition, 2016, James Wight, Pearson, Example 5-7
- spBeam Engineering Software Program Manual v5.50, STRUCTUREPOINT, 2018
- "One-Way Wide Module (Skip) Joist Concrete Floor System Design (ACI 318-14)" Design Example, STRUCTUREPOINT, 2023
- "Reinforced Concrete Continuous Beam Analysis and Design (ACI 318-14)" Design Example, STRUCTUREPOINT, 2018
- "Deflection Observations in Girder-Supported Beams and One-way Slabs" Technical Article, STRUCTUREPOINT, 2017
- Contact Support@StructurePoint.org to obtain supplementary materials (spBeam model: One-Way-Slab-ACI-318-14.slb)

Design Data

f_c' = 4,000 psi normal weight concrete (w_c = 150 lb/ft^3)

f_y = 60,000 psi

Superimposed dead load, *SDL* = 20 psf (floor covering, the ceiling, and mechanical equipment)

Live load, L_o = 80 psf (including partitions)

Use #4 bars for reinforcement (A_s = 0.20 in.^{2}, d_b = 0.50 in.)

Clear cover = 0.75 in. ***ACI 318-14 (Table 20.6.1.3.1)***

Solution

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}

$A_{s,min}$ = minimum area of flexural reinforcement, in.^{2}

b = width of compression face of member, in.

b_w = web width or diameter of circular section, in.

c = distance from extreme compression fiber to neutral axis, in.

c_c = clear cover of reinforcement, in.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

f_c' = specified compressive strength of concrete, psi

f_s = tensile stress in reinforcement at service loads, excluding prestressing reinforcement, psi

f_y = specified yield strength for nonprestressed reinforcement, psi

h = overall thickness, height, or depth of member, in.

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.

M_u = factored moment at section, in.-lb

M_n = nominal flexural strength at section, in.-lb

s = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, tendons, or anchors, in.

V_c = nominal shear strength provided by concrete, lb

V_u = factored shear force at section, lb

w_u = factored load per unit length of beam or one-way slab, lb/in.

β_1 = factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis

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

ρ = ratio of A_s to bd

ϕ = strength reduction factor

2. Preliminary Member Sizing

In lieu of detailed calculation for deflections, ACI 318 Code gives minimum slab thickness for solid nonprestressed one-way slabs in ***Table 7.3.1.1***.

For this one-way slab system, the minimum slab thicknesses per ACI 318-14 are:

End bay: $h_s = \frac{l}{24} = \frac{15\times12-\frac{16}{2}}{24} = \frac{172}{24} = 7.17 \text{ in.}$ ***ACI 318-14 (Table 7.3.1.1)***

Interior bays: $h_s = \frac{l}{28} = \frac{15\times12}{28} = \frac{180}{28} = 6.43 \text{ in.}$ ***ACI 318-14 (Table 7.3.1.1)***

The reference selected a 7 in. slab thickness (slightly below 7.17 in.) which requires the assessment of the deflection in the exterior span.

3. Determination of Span Loads

$$\text{Slab Self-Weight} = t_{slab} \times w_c = \frac{7}{12}\times150 = 87.5 \text{ lb/ft}^2$$

The following gravity load combinations are considered:

$U = 1.40\times D$ ***ACI 318-14 (Eq. 5.3.1a)***

$$w_u = 1.40\times(87.5+20) = 150.50 \text{ psf}$$

$U = 1.20\times D + 1.60\times L$ ***ACI 318-14 (Eq. 5.3.1b)***

$$w_u = 1.20\times(87.5+20)+1.60\times80 = 257.00 \text{ psf}$$

Span loads are governed by the second load combination.

4. Determination of Design Moment and Shear

The factored moment and shear can be determined using the ACI 318 simplified method if the following requirements are satisfied: ***ACI 318-14 (6.5.1)***

- ✓ Members are prismatic.
- ✓ Loads are uniformly distributed.
- ✓ L ≤ 3D (80 psf ≤ 3 × 107.50 = 322.50 psf)
- ✓ There are at least two spans.
- ✓ The longer of two adjacent spans does not exceed the shorter by more than 20 percent.

Thus, the approximate coefficients can be used. The factored moment and shear values are determined and summarized in the following tables. ***ACI 318-14 (Table 6.5.2 and Table 6.5.3)***

Note that a unit strip of 1 ft is considered for the analysis and design of the one-way slab system covered in the Design Example.

Table 1 – One-Way Slab Design Moment Values		
Location		**Design Moment Value**
End Spans	Exterior Support Negative*	$\frac{w_u \times l_n^2}{24} = \frac{0.257 \times 13.08^2}{24} = 1.83\ \frac{\text{kips-ft}}{\text{ft}}$
	Mid-span*	$\frac{w_u \times l_n^2}{14} = \frac{0.257 \times 13.08^2}{14} = 3.14\ \frac{\text{kips-ft}}{\text{ft}}$
	Interior Support Negative†	$\frac{w_u \times l_n^2}{10} = \frac{0.257 \times 13.46^2}{10} = 4.65\ \frac{\text{kips-ft}}{\text{ft}}$
Interior Spans	Mid-span Positive**	$\frac{w_u \times l_n^2}{16} = \frac{0.257 \times 13.83^2}{16} = 3.07\ \frac{\text{kips-ft}}{\text{ft}}$
	Support Negative**	$\frac{w_u \times l_n^2}{11} = \frac{0.257 \times 13.83^2}{11} = 4.47\ \frac{\text{kips-ft}}{\text{ft}}$

* $l_n = 15.00 - \frac{16}{12} - \frac{14}{2 \times 12} = 13.08$ ft

** $l_n = 15.00 - \frac{14}{2 \times 12} - \frac{14}{2 \times 12} = 13.83$ ft

† $l_n = \frac{13.08 + 13.83}{2} = 13.46$ ft ***ACI 318-14 (Table 6.5.2)***

Table 2 – One-Way Slab Design Shear Values	
Location	**Design Shear Value**
End Span at Face of First Interior Support	$1.15 \times \frac{w_u \times l_n}{2} = 1.15 \times \frac{0.257 \times 13.08}{2} = 1.93\ \frac{\text{kips}}{\text{ft}}$
At Face of all other Supports	$\frac{w_u \times l_n}{2} = \frac{0.257 \times 13.08}{2} = 1.68\ \frac{\text{kips}}{\text{ft}}$ (For Exterior Spans) $\frac{w_u \times l_n}{2} = \frac{0.257 \times 13.83}{2} = 1.78\ \frac{\text{kips}}{\text{ft}}$ (For Interior Spans)

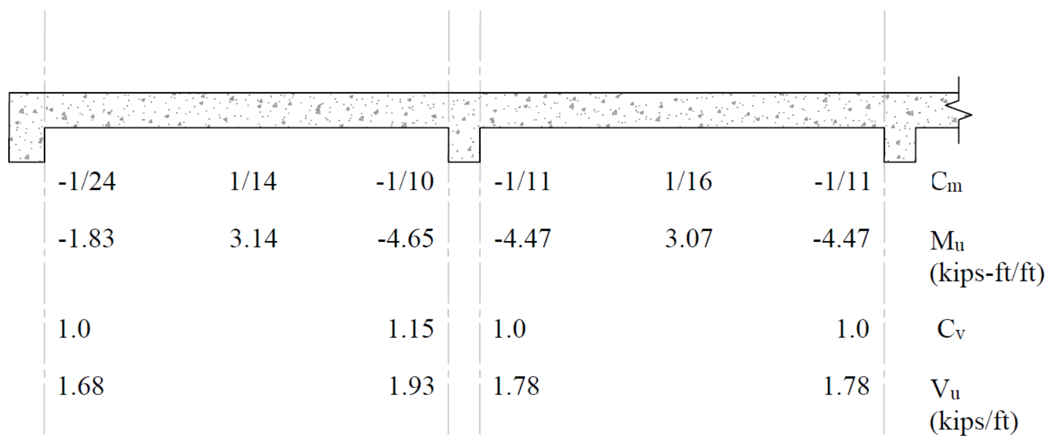


Figure 2 – Moment and Shear Coefficients and Design Values

5. Flexural Design

Calculate the required reinforcement to resist the first interior support negative moment:

M_u = 4.65 kips-ft/ft

Use #4 bars with 0.75 in. concrete clear cover per ***ACI 318-14 (Table 20.6.1.3.1)***. The distance from extreme compression fiber to the centroid of longitudinal tension reinforcement, *d*, is calculated below:

$$d = h - \text{clear cover} - \frac{d_{Longitudinal\ bar}}{2}$$

$$d = 7.00 - 0.75 - \frac{0.50}{2} = 6.00 \text{ in.}$$

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 0.978 × *d*. The assumptions will be verified once the area of steel is finalized.

Assume $jd = 0.978 \times d = 0.978 \times 6.00 = 5.87$ in.

Unit strip width, b = 12 in.

$$A_s = \frac{M_u}{\phi \times f_y \times jd} = \frac{4.65 \times 12,000}{0.90 \times 60,000 \times 5.87} = 0.176 \ \frac{\text{in}^2}{\text{ft}}$$

Recalculate '*a*' for the actual A_s = 0.176 in.^{2} per ft:

$$a = \frac{A_s \times f_y}{0.85 \times f_c' \times b} = \frac{0.176 \times 60,000}{0.85 \times 4,000 \times 12} = 0.259 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{0.259}{0.85} = 0.305 \text{ in.}$$

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{0.305}\right) \times 6.00 - 0.003 = 0.056 > 0.005$$

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

$$A_s = \frac{M_u}{\phi \times f_y \times \left(d - \frac{a}{2}\right)} = \frac{4.65 \times 12,000}{0.90 \times 60,000 \times \left(6.00 - \frac{0.259}{2}\right)} = 0.176 \ \frac{\text{in}^2}{\text{ft}}$$

The minimum reinforcement shall not be less than

$$A_{s,min} = \text{greater of} \begin{cases} \dfrac{0.0018 \times 60,000}{f_y} A_g \\ 0.0014 A_g \end{cases} = \begin{cases} \dfrac{0.0018 \times 60,000}{60,000} \times 12 \times 7 \\ 0.0014 \times 12 \times 7 \end{cases} = \begin{cases} 0.151 \\ 0.118 \end{cases} = 0.151 \ \dfrac{\text{in.}^2}{\text{ft}}$$

ACI 318-14 (Table 7.6.1.1)

$$A_{s,req} = \max \begin{Bmatrix} A_s \\ A_{s,min} \end{Bmatrix} = \max \begin{Bmatrix} 0.176 \\ 0.151 \end{Bmatrix} = 0.176 \ \dfrac{\text{in.}^2}{\text{ft}}$$

For this required steel area, the steel reinforcement ratio is

$$\rho = \frac{A_s/\text{ft}}{b \times d} = \frac{0.176}{12 \times 6} = 0.00245$$

This reinforcement ratio is notably low, typical for many slabs. Hence, it's evident that the selected slab thickness is adequate for the design bending moments.

Provide No. 4 bars at 12 in. $A_{s,prov} = 0.20 \ \dfrac{\text{in.}^2}{\text{ft}} > A_{s,req} = 0.176 \ \dfrac{\text{in.}^2}{\text{ft}}$

The maximum allowed spacing (s_{max}):

$$s_{max} = \text{lesser of} \begin{cases} 3h \\ 18 \text{ in.} \end{cases} = \begin{cases} 3 \times 7 \\ 18 \text{ in.} \end{cases} = \begin{cases} 21 \text{ in.} \\ 18 \text{ in.} \end{cases} = 18 \text{ in.}$$

ACI 318-14 (7.7.2.3)

Thus, s = 12 in. < s_{max} = 18 in.

Calculate the required reinforcement to resist the exterior span positive moment:

M_u = 3.14 kips-ft/ft

The distance from extreme compression fiber to the centroid of longitudinal tension reinforcement:

d = 6.00 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 0.986 × d. The assumptions will be verified once the area of steel is finalized.

Assume $jd = 0.986 \times d = 0.986 \times 6.00 = 5.91$ in.

Unit strip width, b = 12 in.

$$A_s = \frac{M_u}{\phi \times f_y \times jd} = \frac{3.14 \times 12{,}000}{0.90 \times 60{,}000 \times 5.91} = 0.118 \ \frac{\text{in.}^2}{\text{ft}}$$

Recalculate 'a' for the actual A_s = 0.118 in.^{2} per ft:

$$a = \frac{A_s \times f_y}{0.85 \times f'_c \times b} = \frac{0.118 \times 60{,}000}{0.85 \times 4{,}000 \times 12} = 0.174 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{0.174}{0.85} = 0.204 \text{ in.}$$

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{0.204}\right) \times 6.00 - 0.003 = 0.085 > 0.005$$

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

$$A_s = \frac{M_u}{\phi \times f_y \times \left(d - \frac{a}{2}\right)} = \frac{3.14 \times 12{,}000}{0.90 \times 60{,}000 \times \left(6.00 - \frac{0.174}{2}\right)} = 0.118 \ \frac{\text{in.}^2}{\text{ft}}$$

The minimum reinforcement shall not be less than

$$A_{s,min} = \text{greater of} \begin{cases} \dfrac{0.0018 \times 60{,}000}{f_y} A_g \\ 0.0014 A_g \end{cases} = \begin{cases} \dfrac{0.0018 \times 60{,}000}{60{,}000} \times 12 \times 7 \\ 0.0014 \times 12 \times 7 \end{cases} = \begin{cases} 0.151 \\ 0.118 \end{cases} = 0.151 \ \frac{\text{in.}^2}{\text{ft}}$$

ACI 318-14 (Table 7.6.1.1)

$$A_{s,req} = \max\begin{Bmatrix} A_s \\ A_{s,min} \end{Bmatrix} = \max\begin{Bmatrix} 0.118 \\ 0.151 \end{Bmatrix} = 0.151\ \frac{\text{in.}^2}{\text{ft}}$$

Provide No. 4 bars at 12 in. $A_{s,prov} = 0.20\ \frac{\text{in.}^2}{\text{ft}} > A_{s,req} = 0.151\ \frac{\text{in.}^2}{\text{ft}}$

The maximum allowed spacing (s_{max}):

$$s_{max} = \text{lesser of}\begin{cases} 3h \\ 18\text{ in.} \end{cases} = \begin{cases} 3\times 7 \\ 18\text{ in.} \end{cases} = \begin{cases} 21\text{ in.} \\ 18\text{ in.} \end{cases} = 18\text{ in.}$$ ***ACI 318-14 (7.7.2.3)***

Thus, s = 12 in. < s_{max} = 18 in.

<u>Check the shrinkage and temperature reinforcement ($A_{s,S\&T}$) requirement:</u>

$$A_{s,S\&T} = 0.0018\times b\times h = 0.0018\times 12\times 7 = 0.151\ \frac{\text{in.}^2}{\text{ft}} \geq 0.0014\ \frac{\text{in.}^2}{\text{ft}}$$ ***ACI 318-14 (Table 7.6.1.1)***

$$s_{max,S\&T} = \text{lesser of}\begin{cases} 5h \\ 18\text{ in.} \end{cases} = \begin{cases} 5\times 7 \\ 18\text{ in.} \end{cases} = \begin{cases} 35\text{ in.} \\ 18\text{ in.} \end{cases} = 18\text{ in.}$$ ***ACI 318-14 (7.7.2.4)***

Thus, use s = 15 in. < $s_{max,S\&T}$ = 18 in.

Therefore, provide No. 4 bars at 15 in. o.c., as shrinkage and temperature reinforcement.

$$A_s / ft = A_b\left(\frac{12\text{ in.}}{\text{bar spacing in inches, } s}\right) = 0.20\times\left(\frac{12\text{ in.}}{15\text{ in.}}\right) = 0.160\ \frac{\text{in.}^2}{\text{ft}} > A_{s,S\&T} = 0.151\ \frac{\text{in.}^2}{\text{ft}}$$

Using the equation above, this results in a steel area equal to 0.160 in.^{2}/ft. These bars can be placed either in the top or bottom of the slab. If they are placed at the top, they should be placed below the top flexural reinforcement to permit the larger effective depth for that flexural reinforcement, and similarly, they should be placed on top of the bottom layer of flexural reinforcement, as shown in Figure 3.

Check reinforcement spacing for crack control:

The maximum spacing of the flexural reinforcement closest to the tension face of the slab shall be:

$$s = 15 \times \left(\frac{40,000}{f_s}\right) - 2.5 \times c_c \le 12 \times \left(\frac{40,000}{f_s}\right)$$ ***ACI 318-14 (Table 24.3.2)***

$$f_s = \frac{2}{3} \times f_y = \frac{2}{3} \times 60,000 = 40,000 \text{ psi}$$ ***ACI 318-14 (24.3.2.1)***

c_c = 0.75 in.

Thus,

At supports and mid-span

$$s = 15 \times \left(\frac{40,000}{40,000}\right) - 2.50 \times 0.75 = 13.13 \text{ in.} \le 12 \times \left(\frac{40,000}{40,000}\right) = 12 \text{ in.}$$

$$s = 12 \text{ in.} < s_{max} = 18 \text{in.}$$

Thus, maximum bar spacing is 12 in.

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

Table 3 – One-Way Slab Flexural Design Summary					
	External Support	**Exterior Midspan**	**First Interior Support**	**Interior Midspan**	**Second Interior Support**
M_u (kips-ft/ft)	1.83	3.14	4.65	3.07	4.47
$A_{s,\ req'd}$ (in.^{2})	0.068	0.118	0.176	0.115	0.169
$A_{s,\ min}$ (in.^{2})	0.151	0.151	0.151	0.151	0.151
Select bars	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.
$A_{s,\ provided}$ (in.^{2})	0.200	0.200	0.200	0.200	0.200

The reference provides standard bar cutoff points for one-way slabs that satisfy the limitations on span lengths and loadings in ***ACI 318-14 (Table 6.5.2)***. These values are shown in the following Figure.

Wight 7th (8-8 and Fig. A-5c)

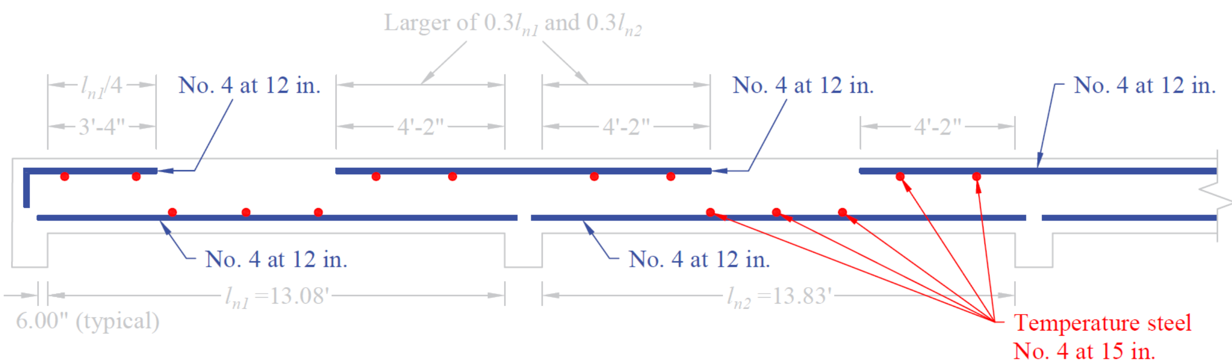


Figure 3 – Slab Reinforcement Detailing

6. Shear Design

From Table 2 above, the shear value in end span at face of first interior support governs.

$$V_u = 1.15 \times \frac{w_u \times l_n}{2} = 1.15 \times \frac{0.257 \times 13.08}{2} = 1.93 \ \frac{\text{kips}}{\text{ft}}$$

Shear strength provided by concrete:

$$\phi V_c = \phi \times \left(2 \times \lambda \times \sqrt{f_c'} \times b_w \times d\right)$$ **ACI 318-14 (Eq. 22.5.5.1)**

$$\phi V_c = 0.75 \times \left(2.00 \times 1.00 \times \sqrt{4,000} \times 12 \times 6\right) = 6830.52 \ \frac{\text{lb}}{\text{ft}} = 6.83 \ \frac{\text{kips}}{\text{ft}}$$

Where:

λ = 1.0 (for Normal Weight) **ACI 318-14 (Table 19.2.4.2)**

ϕ = 0.75 (for Shear) **ACI 318-14 (Table 21.2.1)**

$$V_u = 1.93 \ \frac{\text{kips}}{\text{ft}} < \phi V_c = 6.83 \ \frac{\text{kips}}{\text{ft}}$$

Therefore, the slab shear capacity is adequate.

7. Design the Transverse Top Steel at Girders

ACI 318-19 (7.5.2.3) requires that top transverse reinforcement be designed for the slab to carry the factored floor load acting on the effective width of the overhanging flange (slab), which is assumed to act as a cantilevered beam. The definitions for the width of the overhanging slab are given in ***ACI 318-19 (Table 6.3.2.1)*** for interior and exterior girders. For this floor system, the Reference provides additional discussion.

8. Deflections

Since the preliminary slab thickness met minimum thickness requirement, the deflection calculations are not required except for the exterior bay. Deflection values are calculated and provided for every model created by spBeam Program and can be used by the engineer to make additional optimization decisions for all the spans including the exterior bay. Refer to spBeam Results Section 1.5.1 for notes related to the minimum slab thickness, and Section 3.2.1 for instantaneous deflection values to assess deflection against applicable limits.

9. One-Way Slabs Analysis and Design – spBeam Software

spBeam is widely used for analysis, design and investigation of beams, and one-way slab systems (including standard and wide module joist systems) per American (ACI 318-14) and Canadian (CSA A23.3-14) codes. spBeam can be used for new designs or investigation of existing structural members subjected to flexure, shear, and torsion loads. With capacity to integrate up to 20 spans and two cantilevers of wide variety of floor system types, spBeam is equipped to provide cost-effective, accurate, and fast solutions to engineering challenges.

spBeam provides top and bottom bar details including development lengths and material quantities, as well as live load patterning and immediate and long-term deflection results. Using the moment redistribution feature engineers can deliver safe designs with savings in materials and labor. Engaging this feature allows up to 20% reduction of negative moments over supports reducing reinforcement congestions in these areas.

For illustration and comparison purposes, the following figures provide a sample of the results obtained from an spBeam model created for the one way slab discussed in this example.

spbeam (TM)

spBeam v5.50
A Computer Program for Analysis, Design, and Investigation of
Reinforced Concrete Beams and One-way Slab Systems
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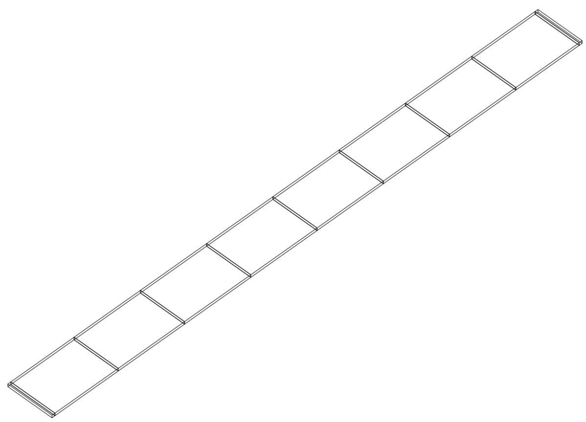

StructurePoint
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Contents

1. Input Echo

1.1. General Information

File Name	F:\StructurePoint\...\One-Way-Slab-ACI-318-14.slb
Project	One-Way Slab
Frame	Interior Frame
Engineer	StructurePoint
Code	ACI 318-14
Reinforcement Database	ASTM A615
Mode	Design
Number of supports =	9 + Left cantilever + Right cantilever
Floor System	One-Way/Beam

1.2. Solve Options

Live load pattern ratio = 100%
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.
Moment redistribution NOT selected.
Effective flange width calculations NOT selected.
Rigid beam-column joint NOT selected.
Torsion analysis and design NOT selected.

1.3. Material Properties

1.3.1. Concrete: Slabs / Beams

w_c	150	lb/ft³
f'_c	4	ksi
E_c	3834.3	ksi
f_r	0.47434	ksi

1.3.2. Concrete: Columns

w_c	150	lb/ft³
f'_c	4	ksi
E_c	3834.3	ksi
f_r	0.47434	ksi

1.3.3. Reinforcing Steel

f_y	60	ksi
f_{yt}	60	ksi
E_s	29000	ksi
Epoxy coated bars	No	

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1.4. Reinforcement Database

Size	Db	Ab	Wb	Size	Db	Ab	Wb
	in	in^2	lb/ft		in	in^2	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				

The program checks one-way slab or beam thickness based on minimum requirement for ACI318-14 code as specified in Tables 7.3.1.1 and 9.3.1.1.

1.5. Span Data

1.5.1. Slabs

Notes:
*a - Deflection check required for panels where slab thickness (t) is less than minimum (Hmin).

Span	Loc	L1	t	wL	wR	H_{min}
		ft	in	ft	ft	in
1	Int	0.667	7.00	5.000	5.000	0.80 LC
2	Int	14.333	7.00	5.000	5.000	7.17 *a
3	Int	15.000	7.00	5.000	5.000	6.43
4	Int	15.000	7.00	5.000	5.000	6.43
5	Int	15.000	7.00	5.000	5.000	6.43
6	Int	15.000	7.00	5.000	5.000	6.43
7	Int	15.000	7.00	5.000	5.000	6.43
8	Int	15.000	7.00	5.000	5.000	6.43
9	Int	14.333	7.00	5.000	5.000	7.17 *a
10	Int	0.667	7.00	5.000	5.000	0.80 RC

1.6. Support Data

1.6.1. Columns

Support	c1a	c2a	Ha	c1b	c2b	Hb	Red %
	in	in	ft	in	in	ft	
1	0.00	0.00	0.000	16.00	120.00	0.000	100
2	0.00	0.00	0.000	14.00	120.00	0.000	100
3	0.00	0.00	0.000	14.00	120.00	0.000	100
4	0.00	0.00	0.000	14.00	120.00	0.000	100
5	0.00	0.00	0.000	14.00	120.00	0.000	100
6	0.00	0.00	0.000	14.00	120.00	0.000	100
7	0.00	0.00	0.000	14.00	120.00	0.000	100
8	0.00	0.00	0.000	14.00	120.00	0.000	100
9	0.00	0.00	0.000	16.00	120.00	0.000	100

1.6.2. Boundary Conditions

Support	Spring		Far End	
	K_z	K_{ry}	Above	Below
	kip/in	kip-in/rad		
1	0	3.572e+005	Fixed	Fixed
2	0	0	Fixed	Fixed
3	0	0	Fixed	Fixed
4	0	0	Fixed	Fixed
5	0	0	Fixed	Fixed
6	0	0	Fixed	Fixed
7	0	0	Fixed	Fixed
8	0	0	Fixed	Fixed

Rotational stiffness can be used to simulate the end support condition (for this example, the end support is a spandrel beam integral to the external span end). The ACI code approximates the moment at spandrel beam end support to $(w_u \times l_n^2)/24$ (ACI 318-14 Table 6.5.2). This can be used as a basis to estimate the rotational stiffness of the spandrel beam with a few iterations.

Support	Spring		Far End	
	K_z	K_{ry}	Above	Below
	kip/in	kip-in/rad		
9	0	3.572e+005	Fixed	Fixed

1.7. Load Data

1.7.1. Load Cases and Combinations

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

1.7.2. Area Loads

Case/Patt	Span	Wa
		lb/ft²
SELF	1	87.50
	2	87.50
	3	87.50
	4	87.50
	5	87.50
	6	87.50
	7	87.50
	8	87.50
	9	87.50
	10	87.50
Dead	2	20.00
	3	20.00
	4	20.00
	5	20.00
	6	20.00
	7	20.00
	8	20.00
	9	20.00
	1	20.00
	10	20.00
Live	2	80.00
	3	80.00
	4	80.00
	5	80.00
	6	80.00
	7	80.00
	8	80.00
	9	80.00
	1	80.00
	10	80.00
Live/Odd	3	80.00
	5	80.00
	7	80.00
	9	80.00
	1	80.00
Live/Even	2	80.00
	4	80.00
	6	80.00
	8	80.00
	10	80.00
Live/S1	2	80.00

Case/Patt	Span	Wa
		lb/ft^2
	1	80.00
Live/S2	2	80.00
	3	80.00
Live/S3	3	80.00
	4	80.00
Live/S4	4	80.00
	5	80.00
Live/S5	5	80.00
	6	80.00
Live/S6	6	80.00
	7	80.00
Live/S7	7	80.00
	8	80.00
Live/S8	8	80.00
	9	80.00
Live/S9	9	80.00
	10	80.00

1.8. Reinforcement Criteria

1.8.1. Slabs and Ribs

	Units	Top Bars		Bottom Bars	
		Min.	Max.	Min.	Max.
Bar Size		#4	#4	#4	#4
Bar spacing	in	1.00	18.00	1.00	18.00
Reinf ratio	%	0.14	5.00	0.14	5.00
Clear Cover	in	0.75		0.75	

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

1.8.2. Beams

	Units	Top Bars		Bottom 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

2.1. Top Reinforcement

Notes:
*3 - Design governed by minimum reinforcement.
*5 - Number of bars governed by maximum allowable spacing.

Span	Zone	Width	M_{max}	X_{max}	$A_{s,min}$	$A_{s,max}$	$A_{s,req}$	Sp_{Prov}	Bars	
		ft	k-ft	ft	in^2	in^2	in^2	in		
1	Left	10.00	0.00	0.000	1.512	13.005	0.000	12.000	10-#4	*3 *5
	Midspan	10.00	0.00	0.000	1.512	13.005	0.000	12.000	10-#4	*3 *5
	Right	10.00	0.00	0.000	1.512	13.005	0.000	12.000	10-#4	*3 *5

Span	Zone	Width ft	M_{max} k-ft	X_{max} ft	$A_{s,min}$ in^2	$A_{s,max}$ in^2	$A_{s,req}$ in^2	Sp_{Prov} in	Bars	
2	Left	10.00	18.40	0.667	1.512	13.005	0.687	12.000	10-#4	*3 *5
	Midspan	10.00	0.00	7.208	0.000	13.005	0.000	0.000	---	
	Right	10.00	42.81	13.750	1.512	13.005	1.617	12.000	10-#4	*5
3	Left	10.00	42.81	0.583	1.512	13.005	1.618	12.000	10-#4	*5
	Midspan	10.00	2.29	9.575	1.512	13.005	0.085	12.000	10-#4	*3 *5
	Right	10.00	42.67	14.417	1.512	13.005	1.612	12.000	10-#4	*5
4	Left	10.00	42.61	0.583	1.512	13.005	1.610	12.000	10-#4	*5
	Midspan	10.00	2.36	9.575	1.512	13.005	0.088	12.000	10-#4	*3 *5
	Right	10.00	43.15	14.417	1.512	13.005	1.631	12.000	10-#4	*5
5	Left	10.00	43.16	0.583	1.512	13.005	1.631	12.000	10-#4	*5
	Midspan	10.00	2.60	9.575	1.512	13.005	0.096	12.000	10-#4	*3 *5
	Right	10.00	43.01	14.417	1.512	13.005	1.625	12.000	10-#4	*5
6	Left	10.00	43.01	0.583	1.512	13.005	1.625	12.000	10-#4	*5
	Midspan	10.00	2.60	5.425	1.512	13.005	0.096	12.000	10-#4	*3 *5
	Right	10.00	43.16	14.417	1.512	13.005	1.631	12.000	10-#4	*5
7	Left	10.00	43.15	0.583	1.512	13.005	1.631	12.000	10-#4	*5
	Midspan	10.00	2.36	5.425	1.512	13.005	0.088	12.000	10-#4	*3 *5
	Right	10.00	42.61	14.417	1.512	13.005	1.610	12.000	10-#4	*5
8	Left	10.00	42.67	0.583	1.512	13.005	1.612	12.000	10-#4	*5
	Midspan	10.00	2.29	5.425	1.512	13.005	0.085	12.000	10-#4	*3 *5
	Right	10.00	42.82	14.417	1.512	13.005	1.618	12.000	10-#4	*5
9	Left	10.00	42.81	0.583	1.512	13.005	1.618	12.000	10-#4	*5
	Midspan	10.00	0.00	7.125	0.000	13.005	0.000	0.000	---	
	Right	10.00	18.40	13.666	1.512	13.005	0.687	12.000	10-#4	*3 *5
10	Left	10.00	0.00	0.667	1.512	13.005	0.000	12.000	10-#4	*3 *5
	Midspan	10.00	0.00	0.667	1.512	13.005	0.000	12.000	10-#4	*3 *5
	Right	10.00	0.00	0.667	1.512	13.005	0.000	12.000	10-#4	*3 *5

2.2. Top Bar Details

	Left				Continuous		Right			
Span	Bars	Length ft	Bars	Length ft	Bars	Length ft	Bars	Length ft	Bars	Length ft
1	---		---		10-#4	0.67	---		---	
2	8-#4	4.98	2-#4	3.28	---		8-#4	6.09	2-#4	3.20
3	---		---		10-#4	15.00	---		---	
4	---		---		10-#4	15.00	---		---	
5	---		---		10-#4	15.00	---		---	
6	---		---		10-#4	15.00	---		---	
7	---		---		10-#4	15.00	---		---	

Span	Left Bars	Left Length ft	Left Bars	Left Length ft	Continuous Bars	Continuous Length ft	Right Bars	Right Length ft	Right Bars	Right Length ft
8	---		---		10-#4	15.00	---		---	
9	8-#4	6.09	2-#4	3.20	---		8-#4	4.98	2-#4	3.28
10	---		---		10-#4	0.67	---		---	

2.3. Top Bar Development Lengths

Span	Left Bars	Left DevLen in	Left Bars	Left DevLen in	Continuous Bars	Continuous DevLen in	Right Bars	Right DevLen in	Right Bars	Right DevLen in
1	---		---		10-#4	12.00	---		---	
2	8-#4	12.00	2-#4	12.00	---		8-#4	12.00	2-#4	12.00
3	---		---		10-#4	12.00	---		---	
4	---		---		10-#4	12.00	---		---	
5	---		---		10-#4	12.00	---		---	
6	---		---		10-#4	12.00	---		---	
7	---		---		10-#4	12.00	---		---	
8	---		---		10-#4	12.00	---		---	
9	8-#4	12.00	2-#4	12.00	---		8-#4	12.00	2-#4	12.00
10	---		---		10-#4	12.00	---		---	

2.4. Bottom Reinforcement

Notes:
*3 - Design governed by minimum reinforcement.
*5 - Number of bars governed by maximum allowable spacing.

Span	Width ft	M_{max} k-ft	X_{max} ft	$A_{s,min}$ in^2	$A_{s,max}$ in^2	$A_{s,req}$ in^2	Sp_{Prov} in	Bars
1	10.00	0.00	0.334	0.000	13.005	0.000	0.000	---
2	10.00	34.00	7.085	1.512	13.005	1.279	12.000	10-#4 *3 *5
3	10.00	33.83	7.747	1.512	13.005	1.273	12.000	10-#4 *3 *5
4	10.00	35.88	7.500	1.512	13.005	1.351	12.000	10-#4 *3 *5
5	10.00	35.91	7.500	1.512	13.005	1.353	12.000	10-#4 *3 *5
6	10.00	35.91	7.500	1.512	13.005	1.353	12.000	10-#4 *3 *5
7	10.00	35.88	7.500	1.512	13.005	1.351	12.000	10-#4 *3 *5
8	10.00	33.83	7.253	1.512	13.005	1.273	12.000	10-#4 *3 *5

Span	Width	M_{max}	X_{max}	$A_{s,min}$	$A_{s,max}$	$A_{s,req}$	Sp_{Prov}	Bars
	ft	k-ft	ft	in²	in²	in²	in	
9	10.00	34.00	7.248	1.512	13.005	1.279	12.000	10-#4 *3 *5
10	10.00	0.00	0.334	0.000	13.005	0.000	0.000	---

2.5. Bottom Bar Details

	Long Bars			Short Bars		
Span	Bars	Start	Length	Bars	Start	Length
		ft	ft		ft	ft
1	---			---		
2	8-#4	0.00	14.33	2-#4	0.00	8.09
3	8-#4	0.00	15.00	2-#4	6.75	2.00
4	8-#4	0.00	15.00	2-#4	6.50	2.00
5	8-#4	0.00	15.00	2-#4	6.50	2.00
6	8-#4	0.00	15.00	2-#4	6.50	2.00
7	8-#4	0.00	15.00	2-#4	6.50	2.00
8	8-#4	0.00	15.00	2-#4	6.25	2.00
9	8-#4	0.00	14.33	2-#4	6.25	8.09
10	---			---		

2.6. Bottom Bar Development Lengths

	Long Bars		Short Bars	
Span	Bars	DevLen	Bars	DevLen
		in		in
1	---		---	
2	8-#4	12.00	2-#4	12.00
3	8-#4	12.00	2-#4	12.00
4	8-#4	12.00	2-#4	12.00
5	8-#4	12.00	2-#4	12.00
6	8-#4	12.00	2-#4	12.00
7	8-#4	12.00	2-#4	12.00
8	8-#4	12.00	2-#4	12.00
9	8-#4	12.00	2-#4	12.00
10	---		---	

2.7. Flexural Capacity

	Top						Bottom				
Span	x	$A_{s,top}$	ΦM_n-	M_u-	Comb Pat	Status	$A_{s,bot}$	ΦM_n+	M_u+	Comb Pat	Status
	ft	in^2	k-ft	k-ft			in^2	k-ft	k-ft		
1	0.000	2.00	-52.68	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK
	0.334	2.00	-52.68	-0.16	U1 All	---	0.00	0.00	0.00	U1 All	OK
	0.667	2.00	-52.68	-0.57	U1 All	---	0.00	0.00	0.00	U1 All	OK
2	0.000	2.00	-52.68	-29.92	U1 Even	---	2.00	52.68	0.00	U1 All	OK
	0.667	2.00	-52.68	-18.40	U1 Even	OK	2.00	52.68	0.00	U1 All	OK
	2.283	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	6.71	U1 S2	OK
	3.283	1.60	-42.35	0.00	U1 All	OK	2.00	52.68	15.84	U1 S2	OK
	3.984	1.60	-42.35	0.00	U1 All	OK	2.00	52.68	21.88	U1 Even	OK
	4.984	0.00	0.00	0.00	U1 All	OK	2.00	52.68	28.48	U1 Even	OK
	5.246	0.00	0.00	0.00	U1 All	OK	2.00	52.68	29.79	U1 Even	OK
	7.085	0.00	0.00	0.00	U1 All	OK	2.00	52.68	34.00	U1 Even	OK
	7.086	0.00	0.00	0.00	U1 All	OK	2.00	52.68	34.00	U1 Even	OK
	7.167	0.00	0.00	0.00	U1 All	OK	1.97	51.85	33.97	U1 Even	OK
	8.086	0.00	0.00	0.00	U1 All	OK	1.60	42.35	32.63	U1 Even	OK
	8.242	0.00	0.00	0.00	U1 All	OK	1.60	42.35	32.17	U1 Even	OK
	9.171	1.49	-39.39	0.00	U1 All	OK	1.60	42.35	28.22	U1 Even	OK
	9.242	1.60	-42.35	0.00	U1 All	OK	1.60	42.35	27.83	U1 Even	OK
	11.133	1.60	-42.35	-11.71	U1 Odd	OK	1.60	42.35	12.59	U1 Even	OK
	12.133	2.00	-52.68	-19.92	U1 Odd	OK	1.60	42.35	0.82	U1 Even	OK
	13.750	2.00	-52.68	-42.81	U1 S2	OK	1.60	42.35	0.00	U1 All	OK
	14.333	2.00	-52.68	-54.36	U1 S2	---	1.60	42.35	0.00	U1 All	OK
3	0.000	2.00	-52.68	-54.36	U1 S2	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-42.81	U1 S2	OK	1.60	42.35	0.00	U1 All	OK
	5.425	2.00	-52.68	-1.56	U1 Even	OK	1.60	42.35	27.12	U1 Odd	OK
	6.746	2.00	-52.68	0.00	U1 All	OK	1.60	42.35	32.64	U1 Odd	OK
	7.500	2.00	-52.68	0.00	U1 All	OK	1.90	50.15	33.78	U1 Odd	OK
	7.746	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	33.83	U1 Odd	OK
	7.747	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	33.83	U1 Odd	OK
	7.748	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	33.83	U1 Odd	OK
	8.748	2.00	-52.68	-0.36	U1 Even	OK	1.60	42.35	32.44	U1 Odd	OK
	9.575	2.00	-52.68	-2.29	U1 Even	OK	1.60	42.35	29.34	U1 Odd	OK
	14.417	2.00	-52.68	-42.67	U1 S3	OK	1.60	42.35	0.00	U1 All	OK
	15.000	2.00	-52.68	-54.14	U1 S3	---	1.60	42.35	0.00	U1 All	OK
4	0.000	2.00	-52.68	-54.14	U1 S3	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-42.61	U1 S3	OK	1.60	42.35	0.00	U1 All	OK
	5.425	2.00	-52.68	-1.77	U1 Odd	OK	1.60	42.35	30.23	U1 Even	OK
	6.499	2.00	-52.68	0.00	U1 All	OK	1.60	42.35	34.54	U1 Even	OK
	7.499	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.88	U1 Even	OK
	7.500	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.88	U1 Even	OK
	7.501	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.88	U1 Even	OK
	8.501	2.00	-52.68	-0.07	U1 Odd	OK	1.60	42.35	34.63	U1 Even	OK
	9.575	2.00	-52.68	-2.36	U1 Odd	OK	1.60	42.35	30.42	U1 Even	OK
	14.417	2.00	-52.68	-43.15	U1 S4	OK	1.60	42.35	0.00	U1 All	OK
	15.000	2.00	-52.68	-54.72	U1 S4	---	1.60	42.35	0.00	U1 All	OK
5	0.000	2.00	-52.68	-54.72	U1 S4	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-43.16	U1 S4	OK	1.60	42.35	0.00	U1 All	OK
	5.425	2.00	-52.68	-2.59	U1 Even	OK	1.60	42.35	30.29	U1 Odd	OK
	6.499	2.00	-52.68	-0.45	U1 Even	OK	1.60	42.35	34.59	U1 Odd	OK
	7.499	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.91	U1 Odd	OK

Span	x	$A_{s,top}$	ΦM_n-	M_u-	Comb Pat	Status	$A_{s,bot}$	ΦM_n+	M_u+	Comb Pat	Status
		Top					Bottom				
	ft	in²	k-ft	k-ft			in²	k-ft	k-ft		
	7.500	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.91	U1 Odd	OK
	7.501	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.91	U1 Odd	OK
	8.501	2.00	-52.68	-0.46	U1 Even	OK	1.60	42.35	34.66	U1 Odd	OK
	9.575	2.00	-52.68	-2.60	U1 Even	OK	1.60	42.35	30.43	U1 Odd	OK
	14.417	2.00	-52.68	-43.01	U1 S5	OK	1.60	42.35	0.00	U1 All	OK
	15.000	2.00	-52.68	-54.56	U1 S5	---	1.60	42.35	0.00	U1 All	OK
6	0.000	2.00	-52.68	-54.56	U1 S5	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-43.01	U1 S5	OK	1.60	42.35	0.00	U1 All	OK
	5.425	2.00	-52.68	-2.60	U1 Odd	OK	1.60	42.35	30.43	U1 Even	OK
	6.499	2.00	-52.68	-0.46	U1 Odd	OK	1.60	42.35	34.66	U1 Even	OK
	7.499	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.91	U1 Even	OK
	7.500	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.91	U1 Even	OK
	7.501	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.91	U1 Even	OK
	8.501	2.00	-52.68	-0.45	U1 Odd	OK	1.60	42.35	34.59	U1 Even	OK
	9.575	2.00	-52.68	-2.59	U1 Odd	OK	1.60	42.35	30.29	U1 Even	OK
	14.417	2.00	-52.68	-43.16	U1 S6	OK	1.60	42.35	0.00	U1 All	OK
	15.000	2.00	-52.68	-54.72	U1 S6	---	1.60	42.35	0.00	U1 All	OK
7	0.000	2.00	-52.68	-54.72	U1 S6	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-43.15	U1 S6	OK	1.60	42.35	0.00	U1 All	OK
	5.425	2.00	-52.68	-2.36	U1 Even	OK	1.60	42.35	30.42	U1 Odd	OK
	6.499	2.00	-52.68	-0.07	U1 Even	OK	1.60	42.35	34.63	U1 Odd	OK
	7.499	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.88	U1 Odd	OK
	7.500	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.88	U1 Odd	OK
	7.501	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	35.88	U1 Odd	OK
	8.501	2.00	-52.68	0.00	U1 All	OK	1.60	42.35	34.54	U1 Odd	OK
	9.575	2.00	-52.68	-1.77	U1 Even	OK	1.60	42.35	30.23	U1 Odd	OK
	14.417	2.00	-52.68	-42.61	U1 S7	OK	1.60	42.35	0.00	U1 All	OK
	15.000	2.00	-52.68	-54.14	U1 S7	---	1.60	42.35	0.00	U1 All	OK
8	0.000	2.00	-52.68	-54.14	U1 S7	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-42.67	U1 S7	OK	1.60	42.35	0.00	U1 All	OK
	5.425	2.00	-52.68	-2.29	U1 Odd	OK	1.60	42.35	29.34	U1 Even	OK
	6.252	2.00	-52.68	-0.36	U1 Odd	OK	1.60	42.35	32.44	U1 Even	OK
	7.252	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	33.83	U1 Even	OK
	7.253	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	33.83	U1 Even	OK
	7.254	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	33.83	U1 Even	OK
	7.500	2.00	-52.68	0.00	U1 All	OK	1.90	50.15	33.78	U1 Even	OK
	8.254	2.00	-52.68	0.00	U1 All	OK	1.60	42.35	32.64	U1 Even	OK
	9.575	2.00	-52.68	-1.56	U1 Odd	OK	1.60	42.35	27.12	U1 Even	OK
	14.417	2.00	-52.68	-42.82	U1 S8	OK	1.60	42.35	0.00	U1 All	OK
	15.000	2.00	-52.68	-54.36	U1 S8	---	1.60	42.35	0.00	U1 All	OK
9	0.000	2.00	-52.68	-54.36	U1 S8	---	1.60	42.35	0.00	U1 All	OK
	0.583	2.00	-52.68	-42.81	U1 S8	OK	1.60	42.35	0.00	U1 All	OK
	2.200	2.00	-52.68	-19.92	U1 Even	OK	1.60	42.35	0.82	U1 Odd	OK
	3.200	1.60	-42.35	-11.72	U1 Even	OK	1.60	42.35	12.59	U1 Odd	OK
	5.091	1.60	-42.35	0.00	U1 All	OK	1.60	42.35	27.83	U1 Odd	OK
	5.162	1.49	-39.39	0.00	U1 All	OK	1.60	42.35	28.22	U1 Odd	OK
	6.091	0.00	0.00	0.00	U1 All	OK	1.60	42.35	32.17	U1 Odd	OK
	6.247	0.00	0.00	0.00	U1 All	OK	1.60	42.35	32.63	U1 Odd	OK
	7.167	0.00	0.00	0.00	U1 All	OK	1.97	51.85	33.97	U1 Odd	OK
	7.247	0.00	0.00	0.00	U1 All	OK	2.00	52.68	34.00	U1 Odd	OK

	Top						Bottom				
Span	x	$A_{s,top}$	ΦM_n-	M_u-	Comb Pat	Status	$A_{s,bot}$	ΦM_n+	M_u+	Comb Pat	Status
	ft	in²	k-ft	k-ft			in²	k-ft	k-ft		
	7.248	0.00	0.00	0.00	U1 All	OK	2.00	52.68	34.00	U1 Odd	OK
	9.087	0.00	0.00	0.00	U1 All	OK	2.00	52.68	29.79	U1 Odd	OK
	9.349	0.00	0.00	0.00	U1 All	OK	2.00	52.68	28.49	U1 Odd	OK
	10.349	1.60	-42.35	0.00	U1 All	OK	2.00	52.68	21.89	U1 Odd	OK
	11.050	1.60	-42.35	0.00	U1 All	OK	2.00	52.68	15.84	U1 S8	OK
	12.050	2.00	-52.68	0.00	U1 All	OK	2.00	52.68	6.71	U1 S8	OK
	13.666	2.00	-52.68	-18.40	U1 Odd	OK	2.00	52.68	0.00	U1 All	OK
	14.333	2.00	-52.68	-29.92	U1 Odd	---	2.00	52.68	0.00	U1 All	OK
10	0.000	2.00	-52.68	-0.57	U1 S9	---	0.00	0.00	0.00	U1 All	OK
	0.334	2.00	-52.68	-0.16	U1 S9	---	0.00	0.00	0.00	U1 All	OK
	0.667	2.00	-52.68	0.00	U1 All	OK	0.00	0.00	0.00	U1 All	OK

2.8. Slab Shear Capacity

Span	b	d	V_{ratio}	ΦV_c	V_u	X_u
	in	in		kip	kip	ft
1	120.00	6.00	1.000	68.31	0.00	0.00
2	120.00	6.00	1.000	68.31	17.77	13.25
3	120.00	6.00	1.000	68.31	17.76	1.08
4	120.00	6.00	1.000	68.31	17.81	13.92
5	120.00	6.00	1.000	68.31	17.79	1.08
6	120.00	6.00	1.000	68.31	17.79	13.92
7	120.00	6.00	1.000	68.31	17.81	1.08
8	120.00	6.00	1.000	68.31	17.76	13.92
9	120.00	6.00	1.000	68.31	17.77	1.08
10	120.00	6.00	1.000	68.31	0.00	0.00

2.9. Material TakeOff

2.9.1. Reinforcement in the Direction of Analysis

Top Bars	745.8 lb	<=>	6.22 lb/ft	<=>	0.622 lb/ft²
Bottom Bars	671.8 lb	<=>	5.60 lb/ft	<=>	0.560 lb/ft²
Stirrups	0.0 lb	<=>	0.00 lb/ft	<=>	0.000 lb/ft²
Total Steel	1417.6 lb	<=>	11.81 lb/ft	<=>	1.181 lb/ft²
Concrete	700.0 ft³	<=>	5.83 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).

Span	Zone	M_{+ve}			M_{-ve}		
		I_g	I_{cr}	M_{cr}	I_g	I_{cr}	M_{cr}
		in⁴	in⁴	k-ft	in⁴	in⁴	k-ft
1	Left	3430	0	38.74	3430	416	-38.74
	Midspan	3430	0	38.74	3430	416	-38.74
	Right	3430	0	38.74	3430	416	-38.74
2	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	0	-38.74
	Right	3430	343	38.74	3430	416	-38.74

Span	Zone	M_{+ve} I_g in⁴	M_{+ve} I_{cr} in⁴	M_{+ve} M_{cr} k-ft	M_{-ve} I_g in⁴	M_{-ve} I_{cr} in⁴	M_{-ve} M_{cr} k-ft
3	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	416	-38.74
	Right	3430	343	38.74	3430	416	-38.74
4	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	416	-38.74
	Right	3430	343	38.74	3430	416	-38.74
5	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	416	-38.74
	Right	3430	343	38.74	3430	416	-38.74
6	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	416	-38.74
	Right	3430	343	38.74	3430	416	-38.74
7	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	416	-38.74
	Right	3430	343	38.74	3430	416	-38.74
8	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	416	-38.74
	Right	3430	343	38.74	3430	416	-38.74
9	Left	3430	343	38.74	3430	416	-38.74
	Midspan	3430	343	38.74	3430	0	-38.74
	Right	3430	343	38.74	3430	416	-38.74
10	Left	3430	0	38.74	3430	416	-38.74
	Midspan	3430	0	38.74	3430	416	-38.74
	Right	3430	0	38.74	3430	416	-38.74

3.1.2. Frame Effective Section Properties

Span	Zone	Weight	Load Level Dead M_{max} k-ft	Dead I_e in⁴	Sustained M_{max} k-ft	Sustained I_e in⁴	Dead+Live M_{max} k-ft	Dead+Live I_e in⁴
1	Right	1.000	-0.24	3430	-0.24	3430	-0.42	3430
	Span Avg	----	----	3430	----	3430	----	3430
2	Middle	0.850	12.00	3430	12.00	3430	20.93	3430
	Right	0.150	-21.33	3430	-21.33	3430	-37.21	3430
	Span Avg	----	----	3430	----	3430	----	3430
3	Left	0.150	-21.33	3430	-21.33	3430	-37.21	3430
	Middle	0.700	9.65	3430	9.65	3430	16.83	3430
	Right	0.150	-19.84	3430	-19.84	3430	-34.60	3430
	Span Avg	----	----	3430	----	3430	----	3430
4	Left	0.150	-19.84	3430	-19.84	3430	-34.60	3430
	Middle	0.700	10.19	3430	10.19	3430	17.78	3430
	Right	0.150	-20.25	3430	-20.25	3430	-35.31	3430
	Span Avg	----	----	3430	----	3430	----	3430
5	Left	0.150	-20.25	3430	-20.25	3430	-35.31	3430
	Middle	0.700	10.06	3430	10.06	3430	17.54	3430
	Right	0.150	-20.11	3430	-20.11	3430	-35.08	3430
	Span Avg	----	----	3430	----	3430	----	3430
6	Left	0.150	-20.11	3430	-20.11	3430	-35.08	3430
	Middle	0.700	10.06	3430	10.06	3430	17.54	3430
	Right	0.150	-20.25	3430	-20.25	3430	-35.31	3430
	Span Avg	----	----	3430	----	3430	----	3430
7	Left	0.150	-20.25	3430	-20.25	3430	-35.31	3430
	Middle	0.700	10.19	3430	10.19	3430	17.78	3430

Span	Zone	Weight	Load Level Dead M_{max} k-ft	Dead I_e in⁴	Sustained M_{max} k-ft	Sustained I_e in⁴	Dead+Live M_{max} k-ft	Dead+Live I_e in⁴
	Right	0.150	-19.84	3430	-19.84	3430	-34.60	3430
	Span Avg	----	----	3430	----	3430	----	3430
8	Left	0.150	-19.84	3430	-19.84	3430	-34.60	3430
	Middle	0.700	9.65	3430	9.65	3430	16.83	3430
	Right	0.150	-21.33	3430	-21.33	3430	-37.21	3430
	Span Avg	----	----	3430	----	3430	----	3430
9	Left	0.150	-21.33	3430	-21.33	3430	-37.21	3430
	Middle	0.850	12.00	3430	12.00	3430	20.93	3430
	Span Avg	----	----	3430	----	3430	----	3430
10	Left	1.000	-0.24	3430	-0.24	3430	-0.42	3430
	Span Avg	----	----	3430	----	3430	----	3430

3.2. Instantaneous Deflections

3.2.1. Extreme Instantaneous Frame Deflections and Corresponding Locations

Span	Direction	Value	Units	Dead	Live Sustained	Live Unsustained	Live Total	Total Sustained	Total Dead+Live
1	Down	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
	Up	Def	in	-0.003	---	-0.002	-0.002	-0.003	-0.005
		Loc	ft	0.000	---	0.000	0.000	0.000	0.000
2	Down	Def	in	0.024	---	0.018	0.018	0.024	0.042
		Loc	ft	6.591	---	6.591	6.591	6.591	6.591
	Up	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
3	Down	Def	in	0.017	---	0.013	0.013	0.017	0.030
		Loc	ft	7.500	---	7.500	7.500	7.500	7.500
	Up	Def	in	0.000	---	0.000	0.000	0.000	0.000
		Loc	ft	0.194	---	0.194	0.194	0.194	0.194
4	Down	Def	in	0.019	---	0.014	0.014	0.019	0.033
		Loc	ft	7.500	---	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
5	Down	Def	in	0.019	---	0.014	0.014	0.019	0.032
		Loc	ft	7.500	---	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
6	Down	Def	in	0.019	---	0.014	0.014	0.019	0.032
		Loc	ft	7.500	---	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
7	Down	Def	in	0.019	---	0.014	0.014	0.019	0.033
		Loc	ft	7.500	---	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
8	Down	Def	in	0.017	---	0.013	0.013	0.017	0.030
		Loc	ft	7.500	---	7.500	7.500	7.500	7.500
	Up	Def	in	0.000	---	0.000	0.000	0.000	0.000
		Loc	ft	14.806	---	14.806	14.806	14.806	14.806
9	Down	Def	in	0.024	---	0.018	0.018	0.024	0.042
		Loc	ft	7.742	---	7.742	7.742	7.742	7.742
	Up	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---

					Live			Total	
Span	Direction	Value	Units	Dead	Sustained	Unsustained	Total	Sustained	Dead+Live
10	Down	Def	in	---	---	---	---	---	---
		Loc	ft	---	---	---	---	---	---
	Up	Def	in	-0.003	---	-0.002	-0.002	-0.003	-0.005
		Loc	ft	0.667	---	0.667	0.667	0.667	0.667

3.3. Long-term Deflections

3.3.1. Long-term 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_{+ve}					M_{-ve}				
Span	Zone	$A_{s,top}$	b	d	Rho'	Lambda	$A_{s,bot}$	b	d	Rho'	Lambda
		in^2	in	in	%		in^2	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	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
6	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
7	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
8	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
9	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
10	Left	----	----	----	0.000	2.000	----	----	----	0.000	2.000

3.3.2. Extreme Long-term Frame Deflections and Corresponding Locations

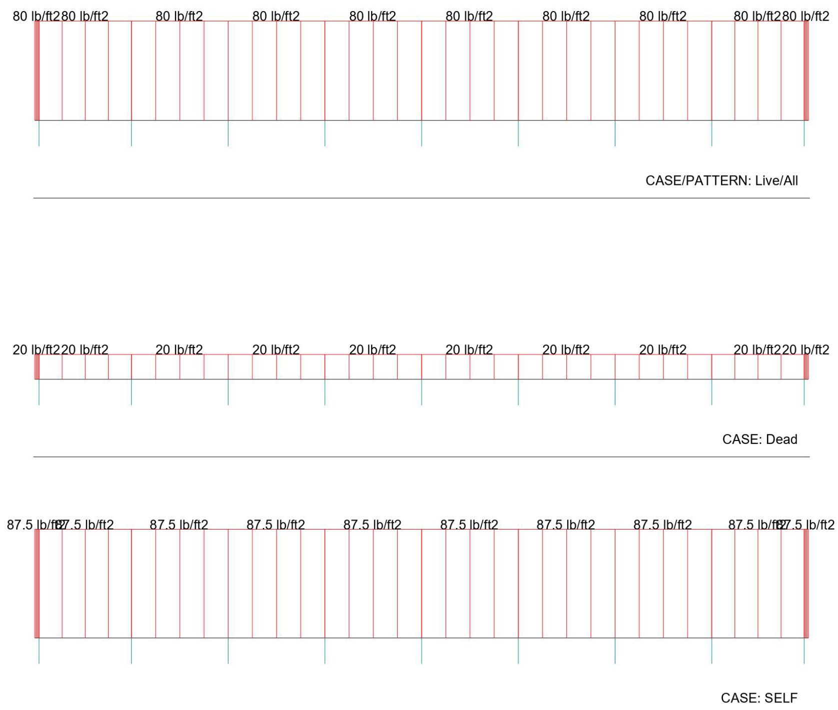
Notes:
Incremental deflections due to creep and shrinkage (cs) based on sustained load level values.
Incremental deflections after partitions are installed can be estimated by deflections due to:
- creep and shrinkage plus unsustained live load (cs+lu), if live load applied before partitions,
- creep 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	-0.005	-0.007	-0.007	-0.010
		Loc	ft	0.000	0.000	0.000	0.000
2	Down	Def	in	0.049	0.067	0.067	0.091
		Loc	ft	6.591	6.591	6.591	6.591
	Up	Def	in	---	---	---	---
		Loc	ft	---	---	---	---
3	Down	Def	in	0.034	0.047	0.047	0.064
		Loc	ft	7.500	7.500	7.500	7.500
	Up	Def	in	0.000	0.000	0.000	0.000
		Loc	ft	0.194	0.194	0.194	0.194
4	Down	Def	in	0.038	0.052	0.052	0.071
		Loc	ft	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---
		Loc	ft	---	---	---	---
5	Down	Def	in	0.037	0.051	0.051	0.069
		Loc	ft	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---
		Loc	ft	---	---	---	---

Span	Direction	Value	Units	cs	cs+lu	cs+l	Total
6	Down	Def	in	0.037	0.051	0.051	0.069
		Loc	ft	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---
		Loc	ft	---	---	---	---
7	Down	Def	in	0.038	0.052	0.052	0.071
		Loc	ft	7.500	7.500	7.500	7.500
	Up	Def	in	---	---	---	---
		Loc	ft	---	---	---	---
8	Down	Def	in	0.034	0.047	0.047	0.064
		Loc	ft	7.500	7.500	7.500	7.500
	Up	Def	in	0.000	0.000	0.000	0.000
		Loc	ft	14.806	14.806	14.806	14.806
9	Down	Def	in	0.049	0.067	0.067	0.091
		Loc	ft	7.742	7.742	7.742	7.742
	Up	Def	in	---	---	---	---
		Loc	ft	---	---	---	---
10	Down	Def	in	---	---	---	---
		Loc	ft	---	---	---	---
	Up	Def	in	-0.005	-0.007	-0.007	-0.010
		Loc	ft	0.667	0.667	0.667	0.667

4. Diagrams

4.1. Loads



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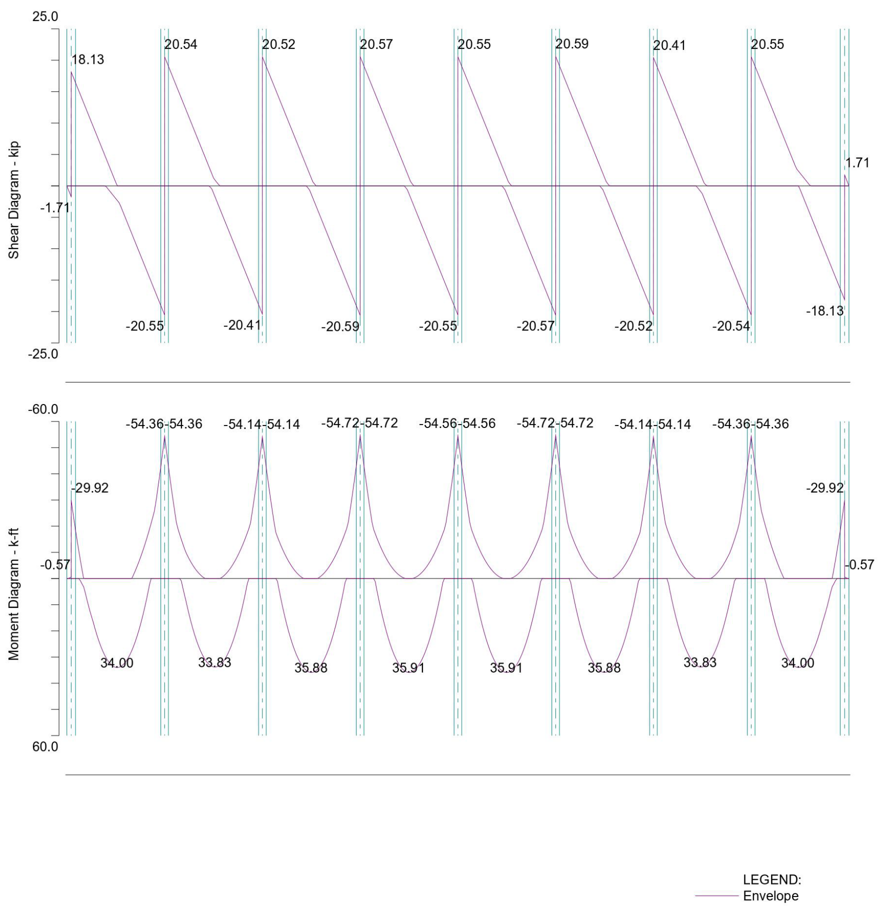
Project: One-Way Slab
Frame: Interior Frame
Engineer: StructurePoint
Code: ACI 318-14
Date: 05/08/24
Time: 12:49:42

4.2. Internal Forces

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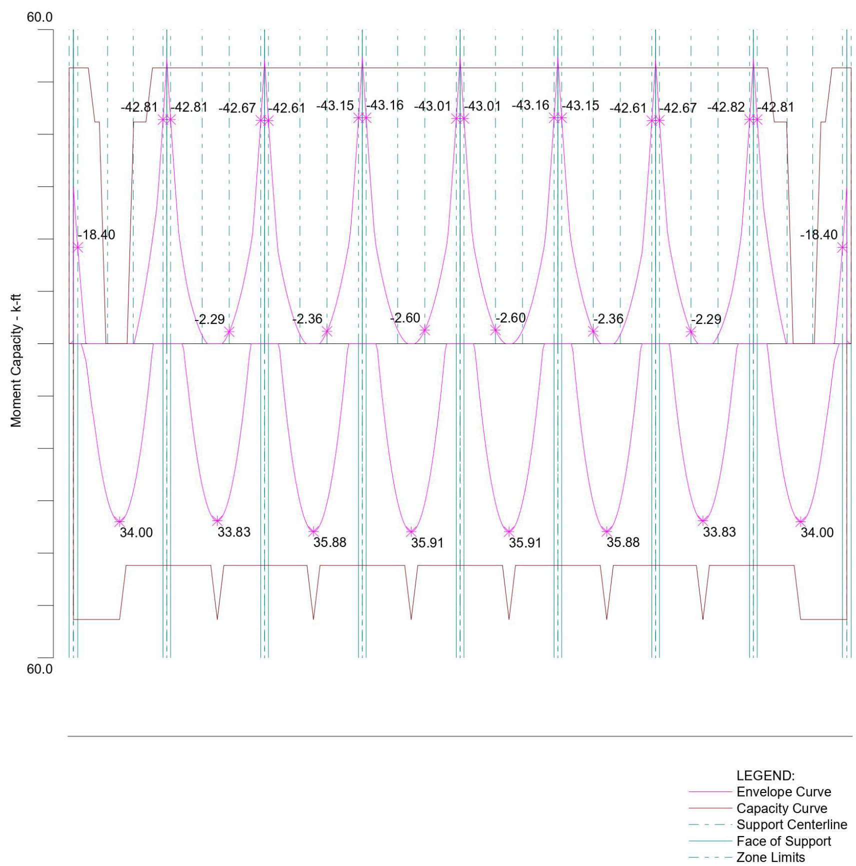
4.3. Moment Capacity

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Engineer: StructurePoint
Code: ACI 318-14
Date: 05/08/24
Time: 12:49:42
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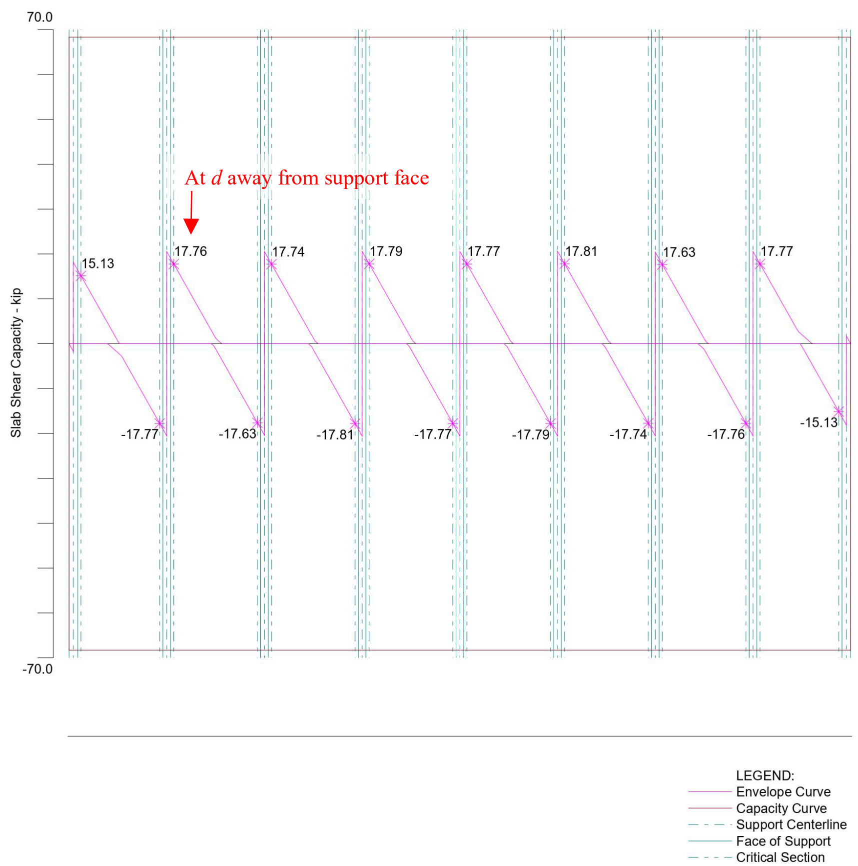
4.4. Shear Capacity



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Project: One-Way Slab
Frame: Interior Frame
Engineer: StructurePoint
Code: ACI 318-14
Date: 05/08/24
Time: 12:49:42

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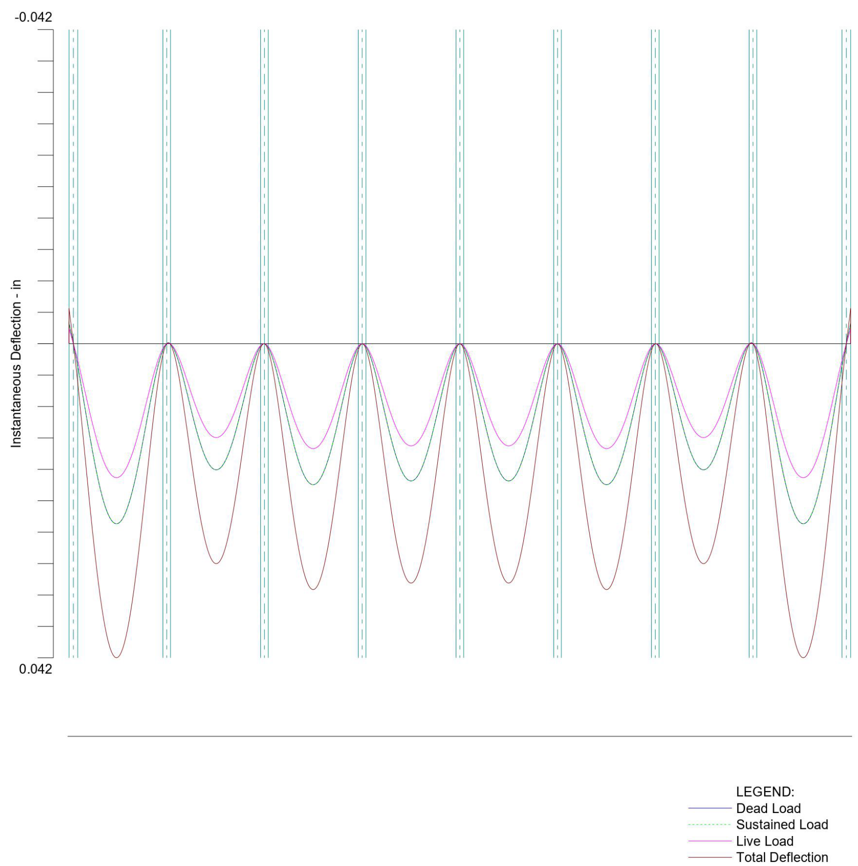
4.5. Deflection



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Project: One-Way Slab

Frame: Interior Frame

Engineer: StructurePoint

Code: ACI 318-14

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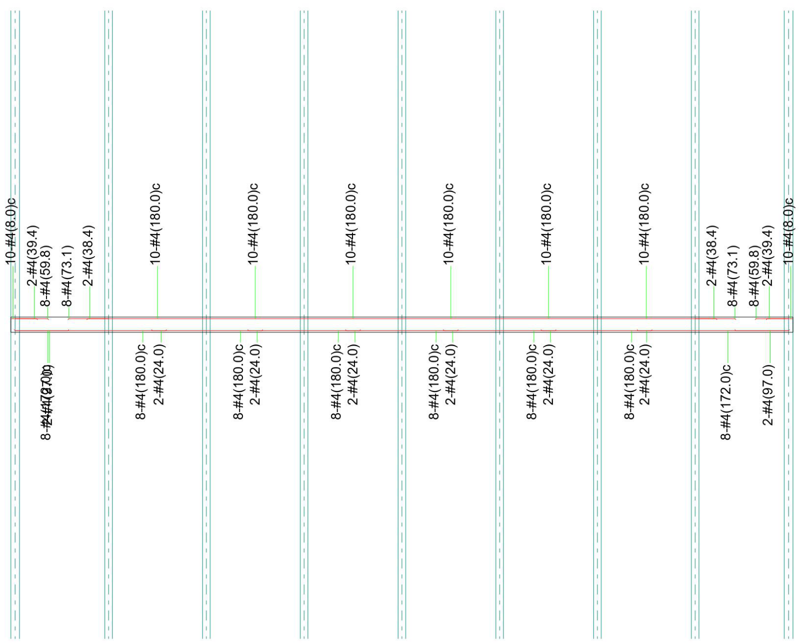
4.6. Reinforcement



Flexural and Transverse Reinforcement

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File: F:\StructurePoint\spBeam\One-Way-Slab-ACI-318-14.slb

Project: One-Way Slab

Frame: Interior Frame

Engineer: StructurePoint

Code: ACI 318-14

Date: 05/08/24

Time: 12:49:42

10. Comparison of Design Results

Table 4 - Comparison of Design Moment Values (kips-ft/ft)			
Span Location	**Reference**	**Hand**	**spBeam†**
Exterior Span			
Exterior Negative*	1.84	1.83	1.84
Positive	3.16	3.14	3.40
Interior Negative*	4.70	4.65	4.28
Interior Span			
Interior Negative*	4.47	4.47	4.28
Positive	3.51**	3.07	3.38

* Negative moments are taken at the faces of supports

** Reference used 1/14 incorrectly instead of 1/16 for interior midspan moment coefficient

† Moment values are divided by strip width (10 ft) to convert units from kips-ft to kips-ft/ft

Table 5 - Comparison of Shear Values (kips/ft)			
	Reference	**Hand**	**spBeam**
V_u (At face of support)	1.94	1.93	1.91
V_u (At d from support face)	N.A.*	N.A.*	1.78
ϕV_c	6.83	6.83	6.83

* ***ACI 318-14 (Table 6.5.4)*** provides only the approximate shear values at the support face, making it difficult to calculate V_u at the critical section (distance d from support face). See Section 11.3 for more information.

Table 6 - Comparison of Reinforcement Results									
Span Location	$A_{s,required}$ (in.^{2}/ft)			$A_{s,min}$ (in.^{2}/ft)			Reinforcement		
	Reference	Hand	spBeam	Reference	Hand	spBeam	Reference	Hand	spBeam
Exterior Span									
Exterior Negative	0.070	0.068	0.069	0.151	0.151	0.151	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.
Positive	0.120	0.118	0.128	0.151	0.151	0.151	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.
Interior Negative	0.178	0.176	0.162	0.151	0.151	0.151	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.
Interior Span									
Interior Negative	0.169	0.169	0.162	0.151	0.151	0.151	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.
Positive	0.133	0.115	0.127	0.151	0.151	0.151	No. 4 at 12 in.	No. 4 at 12 in.	No. 4 at 12 in.

In all of the hand calculations and the reference illustrated above, the results are in close or exact agreement with the automated analysis and design results obtained from the spBeam model.

11. Observations & Discussions

11.1. Modeling End Supports Conditions

In continuous one-way concrete slabs, the moment values along the exterior spans vary based on the rotational resistance offered by the exterior support. Engineers typically encounter two common exterior support conditions:

1. Unrestrained exterior support: In this condition, the exterior support is not built integrally with the slab and thus offers no resistance to rotations at the slab end. The support provides bearing resistance to carry the vertical loads and is considered a pin support. When employing spBeam to model a continuous one-way slab, this condition is the default setting when utilizing an exterior pin support, with the rotational stiffness (K_{ry}) is set to zero by default.
2. Integral exterior support: In this condition, the exterior support is built integrally with the slab and thus offers resistance to rotations at the slab end. The support provides bearing resistance along moment resistance and is considered a semi-rigid connection. When employing spBeam to model a continuous one-way slab, rotational stiffness can be used to simulate this end support condition. For this example, the end support is a spandrel girder integral to the external span end, the ACI code approximates the moment at spandrel girder end support to be ($w_u \times l_n^2$)/24 (***ACI 318-14 (Table 6.5.2)***). This can be used as one method to estimate the rotational stiffness of the spandrel girder by trial and error.

11.2. Modeling Transverse Beams

In spBeam the user can incorporate the physical dimensions of the transverse beams/girders to incorporate the added stiffness provided at the support locations. This takes advantage of any additional beam stiffness derived from increased depth and reduces the length of the span segment attributed to just the slab thickness. Modeling these additional support details produces a model with closer representation of the design conditions and potentially removes some conservatism. Engineering judgment is required to determine whether to include transverse beams/girders in the one-way slab modeling as it increases the parameters involved in determining both analysis and design results. The following figure shows the effect of modeling the transverse beams on the design moment values as compared with a simple pin support at the support center.

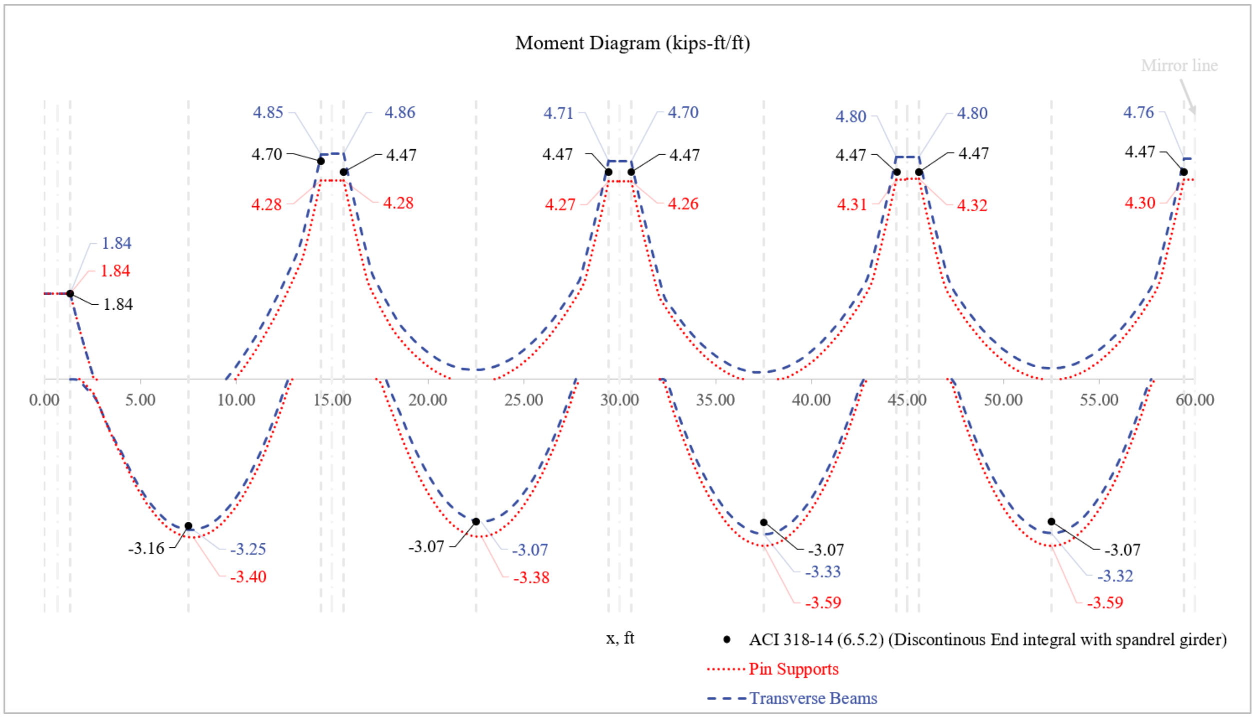

Figure 4 – Moment Diagram Comparison

11.3. One-Way (Beam Action) Shear Critical Section

Using the ACI shear coefficients, the one-way shear is calculated and checked at face of support for simplicity. When using spBeam, the one-way (beam action) shear is checked at a critical section (*d* away from support face) as permitted by ***ACI 318-14 (7.4.3.2)*** (Figure 5). Nevertheless, the shear force at any location along the span (including the support face) can be acquired from the "Internal Forces: M-V-Envelopes" section under "Detailed Results" as required by the user (Figure 6).

2.8. Slab Shear Capacity

Span	b in	d in	V_{ratio}	ΦV_c kip	V_u kip	X_u ft
1	120.00	6.00	1.000	68.31	0.00	0.00
2	120.00	6.00	1.000	68.31	17.77	13.25
3	120.00	6.00	1.000	68.31	17.76	1.08
4	120.00	6.00	1.000	68.31	17.81	13.92
5	120.00	6.00	1.000	68.31	17.79	1.08
6	120.00	6.00	1.000	68.31	17.79	13.92
7	120.00	6.00	1.000	68.31	17.81	1.08
8	120.00	6.00	1.000	68.31	17.76	13.92
9	120.00	6.00	1.000	68.31	17.77	1.08
10	120.00	6.00	1.000	68.31	0.00	0.00

Figure 5 – Slab Shear Capacity (spBeam)

Detailed Results - Internal Forces: M - V - Internal Forces: M - V - Envelopes

Span	x	M-	Comb.	M+	Comb.	V-	Comb.	V+	Comb.
	ft	k-ft		k-ft		kip		kip	
Face of	13.256	-33.71	U1	0.00	U1	-17.79	U1	0.00	U1
Support	13.503	-38.18	U1	0.00	U1	-18.42	U1	0.00	U1
	13.750	-42.81	U1	0.00	U1	-19.05	U1	0.00	U1
	13.750	-42.81	U1	0.00	U1	-19.05	U1	0.00	U1
	13.944	-46.56	U1	0.00	U1	-19.55	U1	0.00	U1
	14.139	-50.41	U1	0.00	U1	-20.05	U1	0.00	U1

Figure 6 – Shear at the Support Face (spBeam)