One-Way Wide Module Joist Concrete Floor Design

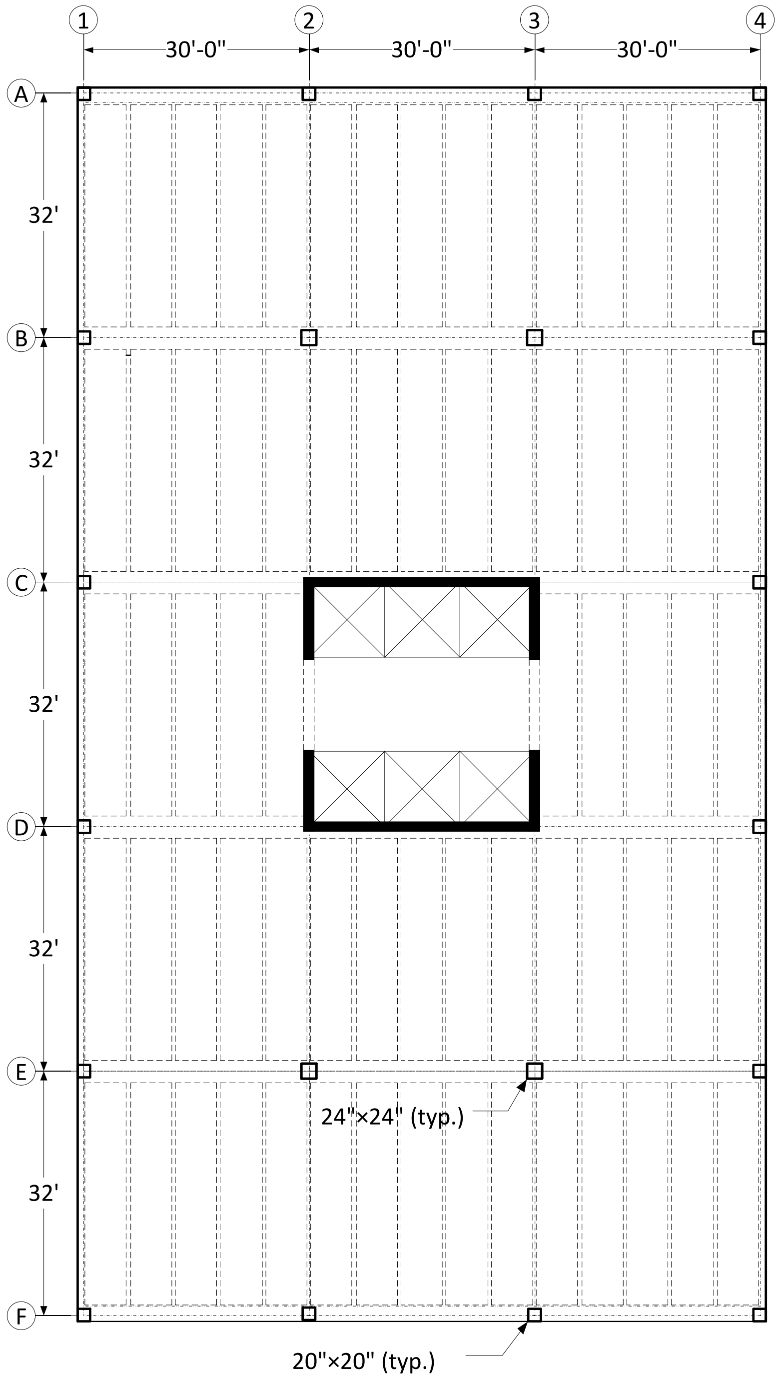


Figure 1 – One-Way Wide Module Joist Concrete Floor Framing System

Overview

A typical floor plan of a 5-story office building located in Los Angeles, CA is shown in Figure 1. The wide-module joist floor system with special reinforced concrete shear walls is selected as the structural system. As the building is assigned to the Seismic Design Category D, building frame is to be designed to resist the gravity loads only and the lateral load effects are resisted by the shear walls. 6 ft – Module with 66 in. pan width and 6 in. rib width shall be utilized.

Code

Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)
Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10)
International Code Council, 2009 International Building Code, Washington, D.C., 2009

Design Data

Floor Heights:

Typical Floor-to-Floor Height $= 12\text{ ft}$

First Story Height $= 16\text{ ft}$

Material Properties:

Concrete:

Unit weight of normal weight concrete, $w_c = 150\text{ pcf}$,

Specified compressive strength, $f'_c = 5000\text{ psi}$

Reinforcing Steel:

Specified yield strength of reinforcement, $f_y = 60000\text{ psi}$,

Specified yield strength of transverse reinforcement, $f_{yt} = 60000\text{ psi}$

Loads:

Dead Loads

Self-weight is to be determined.

Superimposed dead load, $= 20$ psf (Typical floor levels only)

Live Loads

Minimum uniformly distributed live loads, L_o, and minimum concentrated live loads are given in ASCE/SEI 7-10, Table 4-1.

Typical Floor Level, Live load, $L_o = 80\text{ psf}$ (Average value of 80 psf is considered by inspection of Table 4-1 for Office Buildings)

Minimum concentrated live load of 2000 lb uniformly distributed over an area of 2.5 ft^2 needs to be located so as to produce the maximum load effects in the structural members per ASCE/SEI 7-10, 4.3. For simplicity, this requirement is not considered in this example.

Roof Live Load, $L_o = 20$ psf (Ordinary flat roofs)

Required fire resistance rating = 2 hours

Solution

1. PRELIMINARY SIZING

1.1. Determine the preliminary slab and joist sizes for 6'-0" wide-module joist system

1.1.a.One-way Slab

In lieu of detailed calculation for the deflections, ACI 318 gives minimum thickness for one-way slabs in Table 9.5 (a).

$$\text{End Spans: } h = \frac{l}{18.5} = \frac{72}{18.5} = 3.9 \text{ in}$$

$$\text{Interior Spans: } h = \frac{l}{21} = \frac{72}{21} = 3.4 \text{ in}$$

The slab thickness for wide-module joists is generally governed by the fire rating. From IBC 2009, Table 720.1(3), for 2-hour fire rating, the minimum slab thickness is 4.6 in.
Therefore, select slab thickness as 5 in for all spans.

1.1.b.One-way Joist

Since the wide-module joist systems do not meet the limitations of ACI 318, 8.13.1 through 8.13.3, the structural members of this type of joist construction shall be designed as slabs and beams as stated in ACI 318, 8.13.4.
In lieu of detailed calculation for the deflections, ACI Code gives minimum thickness for non-prestressed beams in Table 9.5 (a).

$$\text{End Span: } h = \frac{l}{18.5} = \frac{384}{18.5} = 20.8 \text{ in (governs)}$$

$$\text{Interior Span: } h = \frac{l}{21} = \frac{384}{21} = 18.3 \text{ in}$$

Therefore, select pan depth of 16 in. which makes the total joist depth as 21 in.

1.2. Determine the preliminary column sizes for 6'-0" wide-module joist system

1.2.a.Interior Columns

Select a preliminary size based on the axial load demand. Therefore, the load take-down for an interior column is done as follows:

The governing load combination: $U = 1.2D + 1.6L + 0.5L_r$

where D = Dead Load; L= Live Load; L_r= Roof Live Load

<u>Typical Floor Level Loads</u>

No. of Floors $= 4$

Dead Loads, D

Self-weight of wide-module joist system (16 + 6 + 66) $= 497$ plf (From CRSI Design Handbook 2008, Table 8-3(b). This is equal to $497/6 = 82.83$ psf.

Superimposed dead load $= 20$ psf

Live Load, L: Calculate the live load reduction per ASCE/SEI 7-10, section 4.8.

$$L = L_o \times (0.25 + \frac{15}{\sqrt{K_{LL} A_T}}) \quad \text{ASCE/SEI 7-10, Eq (4-1)}$$

where

$L =$ reduced design live load per ft^2 of area supported by the member

$L_o =$ unreduced design live load per ft^2 of area supported by the member

$K_{LL} =$ live load element factor (ASCE 7-10, Table 4-2)

$A_T =$ tributary area in ft^2

Tributary Area $A_T = (30'\text{–}0'' \times 32'\text{–}0'') = 960 \text{ ft}^2$

$L_o = 80 \text{ psf}$

$$L = 80 \times (0.25 + \frac{15}{\sqrt{4 \times 960}}) = 39.4 \text{ psf}$$

which satisfies $0.40 \times L_o$ requirement for members supporting two or more floors per ASCE/SEI 7-10, section 4.8.1

Roof Level Loads

Dead Loads, D

Self-weight of wide-module joist system (16 + 6 + 66) = 497 plf (From CRSI Design Handbook 2008, Table 8-3(b). This is equal to $497/6 = 82.83$ psf.

No superimposed dead load at the roof

Roof Live Load, L_r: Calculate the roof live load reduction per ASCE/SEI 7-10, section 4.9.

$L_r = L_o \times R_1 \times R_2$ where $12 \le L_r \le 20$ ASCE/SEI 7-10, Eq (4-2)

$L_o = 20$ psf

$R_1 = 0.6$ since $A_T = 960 \text{ ft}^2 \ge 600 \text{ ft}^2$

$R_2 = 1$ for flat roof

$L_r = 20 \times 0.6 \times 1.0 = 12$ psf

Total Factored Load on 1st story interior column (@ 1st interior support)

Total Floor Load $= 4 \times [1.2 \times (82.83 + 20) + 1.6 \times 39.6] \times 960 = 717143 \text{ lb} = 717.1 \text{ kips}$

Total Roof Load $= [1.2 \times 82.83 + 1.6 \times 12] \times 960 = 113852 \text{ lb} = 113.9 \text{ kips}$

Assume 24 in square column with 4 – No. 11 vertical bars with design axial strength, $\phi P_{n,max}$ of

$$\phi P_{n,max} = 0.80\phi[0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$$

$$\phi P_{n,max} = 0.80 \times 0.65 \times [0.85 \times 5000 \times ((24 \times 24 - 4 \times 1.56) + 60000 \times 4 \times 1.56)] = 1453858 \text{ lb}$$

$$\phi P_{n,max} = 1454 \text{ kips}$$

$$\text{Column Self-weight} = \left[1.2 \times \left(\frac{24 \times 24}{144}\right) \times 0.15\right] \times (4 \times 12 + 16) = 46.1 \text{ kips}$$

Total Reaction @ 1st interior support $= 1.15 \times (717.1 + 113.9) + 46.1 = 1002 \text{ kips} < 1454 \text{ kips}$.

Therefore, the interior column size of 24x24 is adequate.

By utilizing the same procedure as outlined above, it is concluded that for the edge and corner columns 20x20 size shall be adequate.

2. DESIGN OF STRUCTURAL MEMBERS

Thedesign of the following structural members shall be performed:

2.1.One-way slab
2.2.One-way Joist
2.3.Interior Beam
2.4.Spandrel Beam
2.5.Interior Column

The computer program solutions shall also be represented for each structural member listed above.

2.1. One-way Slab Design

The typical floor slab design shall be performed. The slab is spanning between joists and designed to carry gravity loads. The unit strip of 1 ft shall be considered in the design. Note that ACI 318 does not allow live load reduction for one-way slabs.

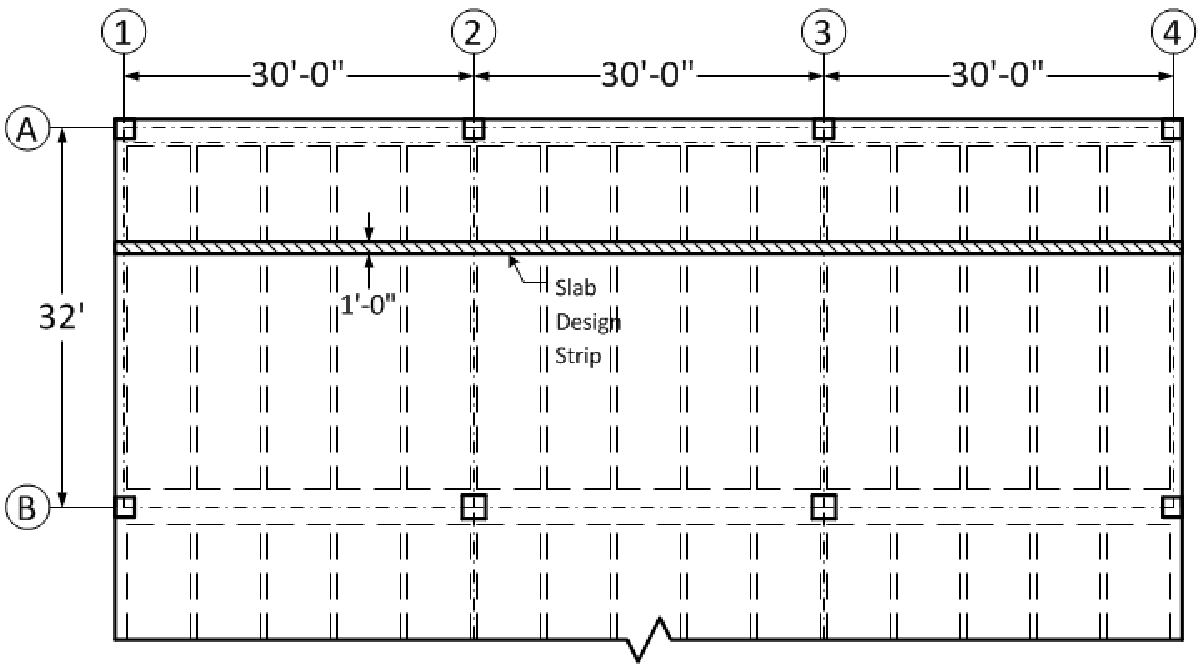


Figure 2.1 – Partial plan view illustrating slab design strip

The design involves the following steps:

2.1.1.Determination of span loads
2.1.2.Determination of design moments and shears
2.1.3.Flexural Design
2.1.4.Shear Design
2.1.5.Deflections
2.1.6.Computer Program Solution
2.1.7.Summary and comparison of design results
2.1.8.Conclusions and observations

2.1.1.Determination of span loads

ACI 318, 9.2.1 gives the following load combinations for structural members loaded with dead and live loads:

$U = 1.4D$ ACI 318, Eq. 9-1

$U = 1.2D + 1.6L$ ACI 318, Eq. 9-2

Factored total load per Eq. 9-1: $w_u = 1.4 \times \left(\left(\frac{5}{12} \times 0.15 \right) + 0.02 \right) = 0.116$ klf per ft

Factored total load per Eq. 9-2: $w_u = 1.2 \times \left(\left(\frac{5}{12} \times 0.15 \right) + 0.02 \right) + 1.6 \times 0.08 = 0.227$ klf per ft

The span loads are governed by load combination per Eq. 9-2.

2.1.2.Determination of design moments and shears

Using the approximate coefficients of ACI 318, 8.3.3, the factored moments, and shears are determined and summarized in the Tables 2.12.1, and 2.1.2.2 respectively below.

Table 2.1.2.1 – One-Way Slab Design Moments

Location		Design Moment Value
End Spans	Exterior Support Negative	$\frac{w_u l_n^2}{24} = \frac{0.227 \times 5.5^2}{24} = 0.29$ ft-kips/ft
	Mid-span Positive	$\frac{w_u l_n^2}{14} = \frac{0.227 \times 5.5^2}{14} = 0.49$ ft-kips/ft
	Interior Support Negative	$\frac{w_u l_n^2}{10} = \frac{0.227 \times 5.5^2}{10} = 0.69$ ft-kips/ft
Interior Spans	Mid-span Positive	$\frac{w_u l_n^2}{16} = \frac{0.227 \times 5.5^2}{16} = 0.43$ ft-kips/ft
	Support Negative	$\frac{w_u l_n^2}{11} = \frac{0.227 \times 5.5^2}{11} = 0.62$ ft-kips/ft

Table 2.1.2.2 – One-Way Slab Design Shears	
Location	Design Shear Value
End Span at Face of First Interior Support	$1.15\times\frac{w_u l_n}{2}=1.15\times\frac{0.227\times5.5}{2}=0.72\ \text{kips/ft}$
At Face of all other Supports	$\frac{w_u l_n}{2}=\frac{0.227\times5.5}{2}=0.62\ \text{kips/ft}$

2.1.3. Flexural Design

For the one-way slab of a wide-module joist system, single layer longitudinal reinforcement is to be provided. The first interior support negative moment governs the design as tabulated in Table 2.1.2.1. Therefore, it is favorable to place the single layer reinforcement closer to the top fiber of the concrete. The required reinforcement shall be calculated for the first interior support negative moment first. The required reinforcement for the end span positive moment shall also be calculated as the low effective depth due to the reinforcement location may govern the required reinforcement amount. Finally, the required reinforcement for design shall be checked against the shrinkage and temperature reinforcement requirement per ACI 318, 7.12.2.1.

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

$$M_u = 0.69\ \text{ft-kips/ft}$$

Assume tension-controlled section ($\phi = 0.9$). Note that this assumption shall be verified within the calculations below.

Unit strip width, $b = 12$ in

The one-way slab reinforcement is to be placed on top of the one-way joist top reinforcement. Assuming No. 3 bars for both the slab and wide-module joist stirrups and following the 1$^{1/2"}$ concrete cover to reinforcement requirement of beam stirrups per ACI 318, 7.7, the distance from extreme compression fiber to the centroid of longitudinal tension reinforcement, d, is calculated below:

$$d = 5-\left[1.5+0.5\times\left(\frac{3}{8}\right)\right]=3.31\ \text{in}$$

Since we are designing a slab (wide compression zone), select a moment arm, jd approximately equal to 0.95d. Assume that $jd = 0.95d = 0.95\times3.31 = 3.15$ in.

Required reinforcement @ initial trial, $A_s = \frac{M_u}{\phi fy(d-a/2)}=\frac{M_u}{\phi fy(jd)}=\frac{0.69\times12{,}000}{0.9\times60{,}000\times3.15}$

$$A_s = 0.049\ \text{in}^2/\text{ft}$$

Use one-iteration to refine this value by inserting it in the equation that finds the depth of the equivalent stress block.

Depth of equivalent stress block, $a = \frac{A_s\,fy}{0.85f'c\quad b}=\frac{0.049\times60{,}000}{0.85\times5{,}000\times12}=0.058$ in.

Neutral axis depth, $c = \frac{a}{\beta_1}=\frac{0.058}{0.85}=0.068$ in.

Strain at tensile reinforcement, $\varepsilon_t = (\frac{0.003}{c}) \quad d_t - 0.003 = (\frac{0.003}{0.068}) \times 3.31 - 0.003 = 0.143 > 0.005$

Therefore, the section is tension-controlled. Use the value of a, $(a = 0.058\text{in})$, to get an improved value for A_s.

$$A_s = \frac{M_u}{\phi fy(d - \frac{a}{2})} = \frac{0.69 \times 12{,}000}{0.9 \times 60{,}000(3.31 - \frac{0.058}{2})} = 0.047 \ \text{in}^2/\text{ft}$$

Calculate the required reinforcement to resist the positive moment

$M_u = 0.49$ ft-kips/ft

Assume tension-controlled section ($\phi = 0.9$). Note that this assumption shall be verified within the calculations below.

Unit strip width, $b = 12$ in

The distance from extreme compression fiber to the centroid of longitudinal tension reinforcement $d = 5 - 3.31 = 1.69$ in.

Since we are designing a slab (wide compression zone), select a moment arm, jd approximately equal to 0.95d. Assume that $jd = 0.95d = 0.95 \times 1.69 = 1.60$ in.

Required reinforcement @ initial trial, $A_s = \frac{M_u}{\phi fy(d - a/2)} = \frac{M_u}{\phi fy(jd)} = \frac{0.49 \times 12{,}000}{0.9 \times 60{,}000 \times 1.60}$

$A_s = 0.068 \ \text{in}^2/\text{ft}$

Use one-iteration to refine this value by inserting it in the equation that finds the depth of the equivalent stress block.

Depth of equivalent stress block, $a = \frac{A_s f_y}{0.85 f'_c \ b} = \frac{0.068 \times 60{,}000}{0.85 \times 5{,}000 \times 12} = 0.08$ in.

Neutral axis depth, $c = \frac{a}{\beta_1} = \frac{0.08}{0.85} = 0.094$ in.

Strain at tensile reinforcement, $\varepsilon_t = (\frac{0.003}{c}) \quad d_t - 0.003 = (\frac{0.003}{0.094}) \times 1.69 - 0.003 = 0.051 > 0.005$

Therefore, the section is tension-controlled. Use the value of a, $(a = 0.08\text{in})$, to get an improved value for A_s.

$$A_s = \frac{M_u}{\phi fy(d - \frac{a}{2})} = \frac{0.49 \times 12{,}000}{0.9 \times 60{,}000(1.69 - \frac{0.08}{2})} = 0.066 \ \text{in}^2/\text{ft}$$

The required reinforcement from analysis is 0.66 in^2/ft. In this example, positive moment value controls the design due to the placement of slab reinforcement near top concrete surface even though it is not the governing design moment for the slab.

Check the shrinkage and temperature reinforcement requirement per ACI 318, 7.12.2.1.

$A_s = 0.0018bh = 0.0018 \times 12 \times 5 = 0.108 \ \text{in}^2 / \text{ft}$

Check reinforcement spacing for crack control.

Per ACI 318, 10.6.4, the maximum spacing of the flexural reinforcement closest to the tension face of the slab shall be:

$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c$, but not greater than $12\left(\frac{40000}{f_s}\right)$

where

$s =$ maximum reinforcement spacing for crack control

$f_s =$ calculated stress in reinforcement closest to the tension face at service load

$c_c =$ the least distance from surface of reinforcement to the tension face.

ACI 318, 10.6.4 permits to take f_s as $2/3f_y$

Therefore, for Grade 60 steel, ACI 318 permits f_s to be taken as $2/3f_y$ = 40000 psi.

$c_c = 1.5$ in for reinforcement resisting negative moment at supports (i.e. tension at the top)

$c_c = 3.125$ in for reinforcement resisting positive moment at mid-span (i.e. tension at the bottom)

Thus,

At supports

$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c = 15\times\left(\frac{40000}{40000}\right) - 2.5\times1.5 = 11.25\,\text{in}$ (governs @ support)

But not greater than $s = 12\left(\frac{40000}{f_s}\right) = 12\times\left(\frac{40000}{40000}\right) = 12\,\text{in}$

At mid-span

$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c = 15\times\left(\frac{40000}{40000}\right) - 2.5\times3.125 = 7.19\,\text{in}$ (governs @ mid-span)

But not greater than $s = 12\left(\frac{40000}{f_s}\right) = 12\times\left(\frac{40000}{40000}\right) = 12\,\text{in}$

Therefore, for this one-way slab, the shrinkage and temperature reinforcement requirement per ACI 318, 7.12 governs the required reinforcement area ($A_s = 0.108\,\text{in}^2$ / ft) and crack control requirement per ACI 318, 10.6.4 governs the reinforcement spacing ($s = 7.19\,\text{in}$).

The most feasible reinforcement solution that meets both requirements mentioned above is to provide welded wire fabric reinforcement, 6 x 6-W5.5 x W5.5. Note that the welded wire reinforcement selected provides minimum shrinkage and temperature reinforcement in the slab direction parallel to the joists as well. Alternately, rebar can be utilized in lieu of welded wire fabric. As illustrated above, reinforcing spacing of 7 in shall meet the spacing requirement for crack control in the main direction. It should be noted that two conditions specific to this design contributes to having such a stringent spacing requirement. These are listed below:

- The 5 in. slab has a single layer reinforcement that is placed near the top surface (i.e. clear cover from the top surface to the reinforcement is 1.5 in. This result in a high c_c value for the calculation of reinforcement spacing for crack control due to positive moment.
- The stress in reinforcement closest to the tension face at service load, f_s, is taken as $2/3f_y$ as permitted by ACI 318 without calculation. It is very likely that under the loading considered, the stress in the steel be lower than $2/3f_y$. The f_s value is expected to be in the range of $1/3f_y$ to $1/2f_y$. Even it is assumed to be $1/2f_y$, s value will be 12 in. The designer may choose to calculate the actual f_s value which may justify utilizing No. 3 @ 12 in. in the main direction.

In the slab direction parallel to the joists No. 3 @ 12 in. shall suffice.

2.1.4. Shear Design

From Table 2.1.2.2 above, the shear value in end span at face of first interior support governs.

$$V_u = 1.15\frac{w_u l_n}{2} = (1.15\times 0.227\times 5.5)/2 = 0.72 \text{ kips / ft}$$

The design shear at a distance, d, away from the face of support,

$$V_u = 0.72 - 0.227\times\frac{1.69}{12} = 0.69 \text{ kips / ft}$$

Shear strength provided by concrete

$$\phi V_c = \phi(2\sqrt{f'_c}b_w d = 0.75\times\left(2\times 1.0\times\sqrt{5000}\times 12\times 1.69\right) = 2151 \text{ lb / ft } = 2.15 \text{ kips / ft}$$

$V_u = 0.69 \text{ kips / ft} < \phi V_c = 4.21 \text{ kips / ft}$. Therefore, the slab shear capacity is adequate.

2.1.5. Deflections

ACI 318 provides the minimum thickness of nonprestressed one-way slabs in Table 9.5(a) unless deflections are calculated. Since the preliminary slab thickness met this requirement, the deflection calculations are waived here. Typically, in hand solutions, the designer opts to follow ACI 318, 9.5.2 in lieu of deflection calculations which can be long and tedious. It should be noted that unless governed by fire rating requirements as in this example, in most other cases, lesser thicknesses and consequent cost savings can be achieved through deflection computations.

2.1.6. Computer Program Solution

spSlab Program is utilized to design the one-way slab system. The one-way slab is modeled as 1-ft unit strip. The exterior spandrel joists along grids 1 and 4 provide some rotational stiffness at the support. In spSlab solution, the rotational stiffness is conservatively ignored by modeling the exterior supports as pin supports. This results in exterior support moment value being zero and conservatively higher positive moment and interior support moment for the end spans. Also, for one-way slab run, the joists are omitted in the model and centerline moments are considered for the design moments. Note that spSlab allows a user-defined reinforcement sizes as well. In this example, user-defined bar size #2 is defined in lieu of welded wire fabric,W5.5, with the cross-sectional area of 0.055 in^2 (see Fig. 2.1.6).

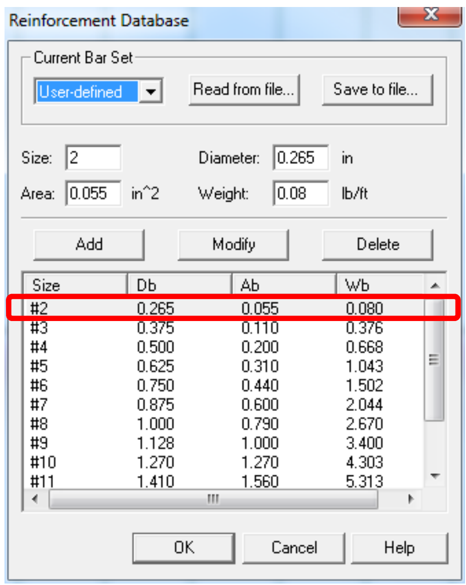


Figure 2.1.6 – spSlab Reinforcement Database – User-defined Bar Set

The program calculates the internal forces (shear force and bending moment), moment and shear capacities, immediate and long-term deflections. The graphical and text results are provided in the Appendix A for input and output of the spSlab program.

2.1.6.1 Isometric View of 15 span - 1-ft wide unit strip of One-way Slab from spSlab

spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\One-Way Slab.slb

Project: Wide-Module Joist System

Frame: One-Way Slab

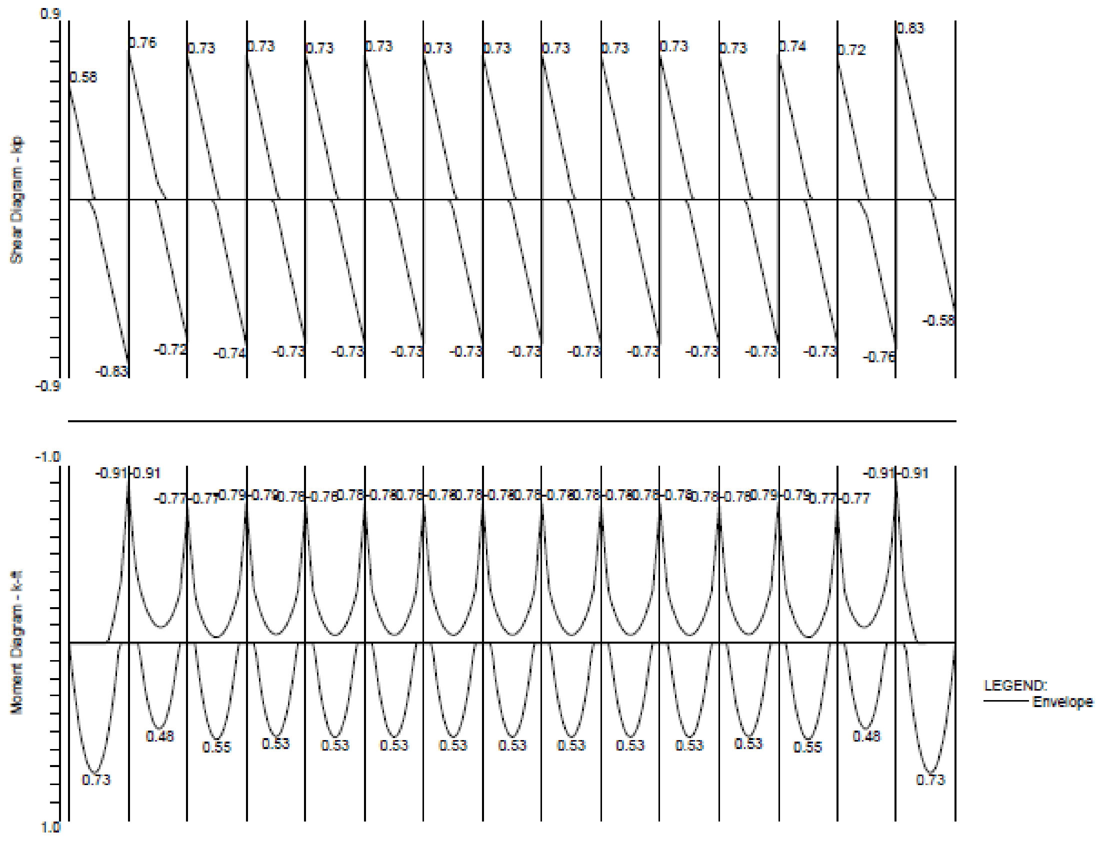
Engineer: MD

Code: ACI 318-11

Date: 11/19/13

Time: 11:42:15

2.1.6.2 spSlab Model Calculated Internal Forces (Shear Force & Bending Moment)



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File: C:\Projects\Wide-Module Joist System\One-Way Slab\One-Way Slab.slb

Project: Wide-Module Joist System

Frame: One-Way Slab

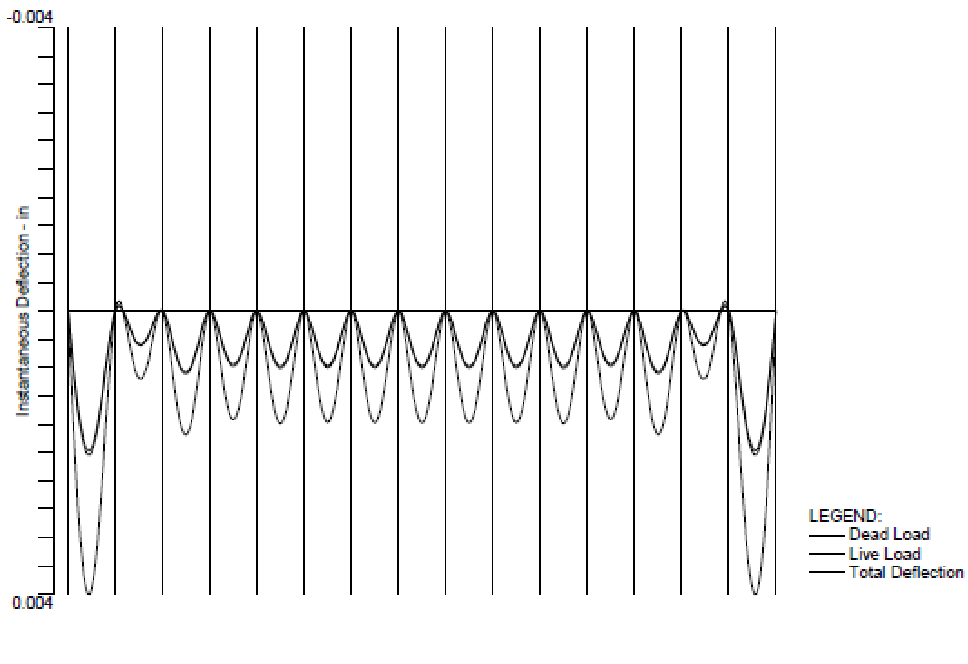
Engineer: MD

Code: ACI 318-11

Date: 12/05/13

Time: 11:45:19

2.1.6.3 spSlab Model Calculated Immediate Deflections



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\One-Way Slab.slb

Project: Wide-Module Joist System

Frame: One-Way Slab

Engineer: MD

Code: ACI 318-11

Date: 11/22/13

Time: 12:35:10

2.1.7. Summary and Comparison of Results

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================================================================================
[2] DESIGN RESULTS
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Top Reinforcement
=================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span Zone       Width       Mmax      Xmax     AsMin     AsMax     AsReq    SpProv     Bars
   ---- ------- -------- ---------- -------- -------- -------- -------- -------- -------
      1 Left        1.00       0.00    0.000    0.000    0.859    0.000    6.000   2-#2
        Midspan     1.00       0.01    3.900    0.108    0.859    0.001    6.000   2-#2 *3
        Right       1.00       0.91    6.000    0.108    0.859    0.061    6.000   2-#2 *3

      2 Left        1.00       0.91    0.000    0.108    0.859    0.061    6.000   2-#2 *3
        Midspan     1.00       0.16    2.100    0.108    0.859    0.010    6.000   2-#2 *3
        Right       1.00       0.77    6.000    0.108    0.859    0.051    6.000   2-#2 *3

      3 Left        1.00       0.77    0.000    0.108    0.859    0.051    6.000   2-#2 *3
        Midspan     1.00       0.08    3.900    0.108    0.859    0.005    6.000   2-#2 *3
        Right       1.00       0.79    6.000    0.108    0.859    0.053    6.000   2-#2 *3

      4 Left        1.00       0.79    0.000    0.108    0.859    0.053    6.000   2-#2 *3
        Midspan     1.00       0.09    2.100    0.108    0.859    0.006    6.000   2-#2 *3
        Right       1.00       0.78    6.000    0.108    0.859    0.052    6.000   2-#2 *3

      5 Left        1.00       0.78    0.000    0.108    0.859    0.052    6.000   2-#2 *3
        Midspan     1.00       0.08    3.900    0.108    0.859    0.006    6.000   2-#2 *3
        Right       1.00       0.78    6.000    0.108    0.859    0.052    6.000   2-#2 *3

Bottom Reinforcement
====================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span    Width       Mmax      Xmax     AsMin     AsMax     AsReq    SpProv     Bars
   ---- -------- ---------- -------- -------- -------- -------- -------- -------
      1     1.00       0.73    2.500    0.108    0.444    0.097    6.000   2-#2 *3

      2     1.00       0.48    3.000    0.108    0.444    0.063    6.000   2-#2 *3

      3     1.00       0.55    3.000    0.108    0.444    0.071    6.000   2-#2 *3

      4     1.00       0.53    3.000    0.108    0.444    0.069    6.000   2-#2 *3

      5     1.00       0.53    3.000    0.108    0.444    0.070    6.000   2-#2 *3

   NOTES:
   *3 - Design governed by minimum reinforcement.

Slab Shear Capacity
===================
   Units: b, d (in), Xu (ft), PhiVc, Vu(kip)
   Span        b        d    Vratio        PhiVc           Vu           Xu
   ---- -------- -------- -------- ------------ ------------ ------------
      1    12.00     1.74    1.000         2.22         0.80         5.85
      2    12.00     1.74    1.000         2.22         0.73         0.15
      3    12.00     1.74    1.000         2.22         0.70         5.85
      4    12.00     1.74    1.000         2.22         0.70         0.15
      5    12.00     1.74    1.000         2.22         0.70         5.85
```

Table 2.1.7.1 – Comparison of Hand Solution with spSlab Solution				
Span Location	**Reinforcement Required for Flexure (in^2/ft)**		**Minimum Reinforcement (in^2/ft) (Shrinkage & Temperature Reinforcement)**	
End Span	**Hand Solution**	**spSlab Solution**	**Hand Solution**	**spSlab Solution**
Interior Negative	0.047	0.058	0.108	0.108
Positive	0.066	0.083	0.108	0.108

2.1.8. Conclusions and Observations

In this design example, the modeling of the exterior support condition as pin compared to the hand solution which utilizes the approximate moments per ACI 318, 8.3.3 does not influence the design of flexural reinforcement due to the fact that the shrinkage and temperature reinforcement requirement governs the required reinforcement criteria. In general, the pin support assumption may be employed by the designer as it yields the conservative values for reinforcement. However, spSlab program also enables the user to enter the rotational support springs as boundary conditions for exterior support for proper assessment of rotational stiffness effects.

Typically, in wide-module joist construction, one-way slab is reinforced with single layer reinforcement placed near the top in the main direction. As seen in this example, this may create crack control criteria to govern the reinforcement spacing and consequently, it may warrant the use of welded wire fabric reinforcement instead of No. 3 rebar.

2.2. One-way Joist Design

The typical floor one-way joist design shall be performed. The wide-module joists are considered as beams per ACI 318, 8.13.4. Therefore, the design of the joist shall conform to the requirements of T-beams per ACI 318, 8.12.

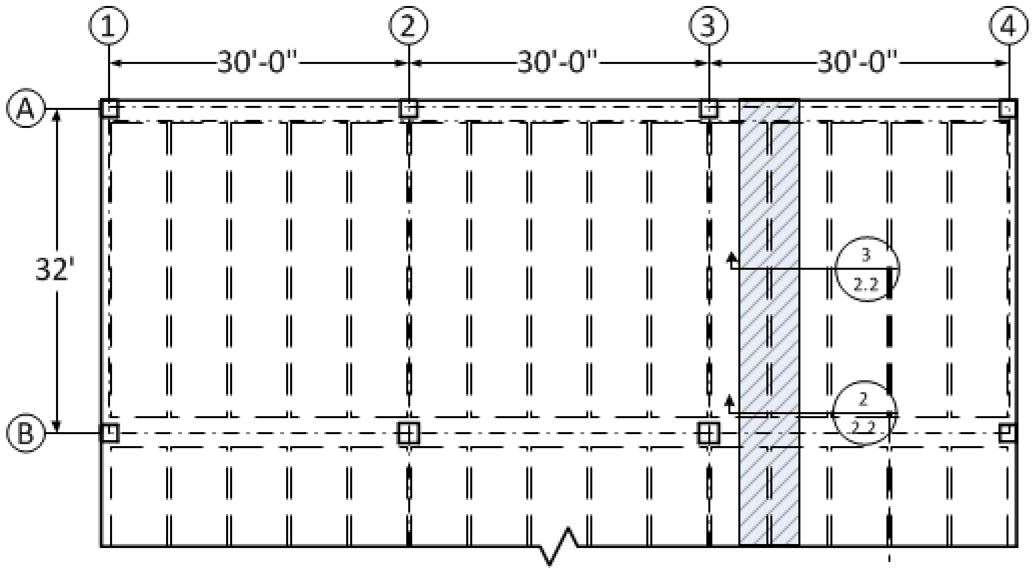


Figure 2.2 – Partial plan view illustrating one-way joist to be design

The design involves the following steps:

2.2.1. Determination of span loads
2.2.2. Determination of design moments and shears
2.2.3. Flexural Design
2.2.4. Shear Design
2.2.5. Deflections
2.2.6. Computer Program Solution
2.2.7. Summary and comparison of design results
2.2.8. Conclusions and observations

2.2.1. Determination of span loads

ACI 318, 9.2.1 gives the following load combinations for structural members loaded with dead and live loads:

$U = 1.4D$ ACI 318, Eq. 9-1

$U = 1.2D + 1.6L$ ACI 318, Eq. 9-2

Check Floor Live Load Reduction per ASCE 7-10, sections 4.8.

$$L = L_o \times (0.25 + \frac{15}{\sqrt{K_{LL} A_T}})$$ ASCE 7-10, Eq (4-1)

where

Live Load Element Factor, $K_{LL} = 2$ for interior beams [ASCE/SEI 7-10, Table 4-2]

Tributary Area $A_T = (6'-0''\times 32'-0'') = 192\ \text{ft}^2$

Since $K_{LL} \times A_T = 2\times 192 = 384\ \ \text{ft}^2 < 400\ \ \text{ft}^2$, live load reduction is not applicable.

Factored total load per Eq. 9-1:

$$w_u = 1.4\times\left[\left\{\left(\left(\frac{5}{12}\times 0.15\right)+0.02\right)\times 6\right\}+\left\{\left(\left(\frac{(6+8.66)/2}{12}\right)\times\frac{16}{12}\right)\times 0.15\right\}\right] = 0.86\ \text{klf}$$

Factored total load per Eq. 9-2:

$$w_u = 1.2\times\left[\left\{\left(\left(\frac{5}{12}\times 0.15\right)+0.02\right)\times 6\right\}+\left\{\left(\left(\frac{(6+8.66)/2}{12}\right)\times\frac{16}{12}\right)\times 0.15\right\}\right]+1.6\times 0.08\times 6$$

$$w_u = 1.51\,\text{klf}$$

The span loads are governed by load combination per Eq. 9-2.

2.2.2. Determination of design moments and shears

Using the approximate coefficients of ACI 318, 8.3.3, the factored moments, and shears are determined and summarized in the Tables 2.2.2.1, and 2.2.2.2 respectively below.

Table 2.2.2.1 – One-Way Joist Design Moments

Location		Design Moment Value
End Spans	Exterior Support Negative	$\frac{w_u l_n^2}{24} = \frac{1.51\times 30.17^2}{24} = 57.3$ ft-kips
	Mid-span Positive	$\frac{w_u l_n^2}{14} = \frac{1.51\times 30.17^2}{14} = 98.2$ ft-kips
	Interior Support Negative	$\frac{w_u l_n^2}{10} = \frac{1.51\times 30.08^2}{10} = 136.6$ ft-kips
Interior Spans	Mid-span Positive	$\frac{w_u l_n^2}{16} = \frac{1.51\times 30^2}{16} = 84.9$ ft-kips
	Support Negative	$\frac{w_u l_n^2}{11} = \frac{1.51\times 30^2}{11} = 123.5$ ft-kips

Table 2.2.2.2 – One-Way Joist Design Shears

Location	Design Shear Value
End Span at Face of First Interior Support	$1.15\times\frac{w_u l_n}{2} = 1.15\times\frac{1.51\times 30.17}{2} = 26.2\ \text{kips}$
At Face of all other Supports	$\frac{w_u l_n}{2} = \frac{1.51\times 30.17}{2} = 22.8\ \text{kips}$

*when support beam is wider than the column, the clear span, l_n, of the joists is measured from the face of the column. For calculating negative moments, l_n, is taken as the average of the adjacent clear spans per ACI 318, 8.3.3.

2.2.3. Flexural Design

For the one-way joist of a wide-module joist system, the end span moment values govern the design as tabulated in Table 2.2.2.1.

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

$$M_u = 136.6 \text{ ft-kips}$$

Assume tension-controlled section ($\phi = 0.9$). Note that this assumption shall be verified within the calculations below.

By assuming the maximum bar size for joist top reinforcement as No. 6, the distance from extreme compression fiber to the centroid of longitudinal tension reinforcement, d, is calculated below:

$$d = 21 - \left(1.5 + \frac{3}{8} + 0.5 \times \left(\frac{6}{8}\right)\right) = 18.75 \text{ in}$$

Since we are designing for the negative moment in a T-Beam (narrow compression zone), select a moment arm, jd approximately equal to 0.9d. Assume that $jd = 0.9d = 0.9 \times 18.75 = 16.88$ in.

Required reinforcement @ initial trial, $A_s = \dfrac{M_u}{\phi f_y (d - a/2)} = \dfrac{M_u}{\phi f_y (jd)} = \dfrac{136.6 \times 12{,}000}{0.9 \times 60{,}000 \times 16.88}$

$$A_s = 1.80 \text{ in}^2$$

Use one-iteration to refine this value by inserting it in the equation that finds the depth of the equivalent stress block.

Depth of equivalent stress block, $a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{1.80 \times 60{,}000}{0.85 \times 5{,}000 \times 7.33} = 3.47$ in.

Neutral axis depth, $c = \dfrac{a}{\beta_1} = \dfrac{3.47}{0.85} = 4.08$ in.

Strain at tensile reinforcement, $\varepsilon_t = (\dfrac{0.003}{c})d_t - 0.003 = (\dfrac{0.003}{4.08})18.75 - 0.003 = 0.0108 > 0.005$

Therefore, the section is tension-controlled. Use the value of a, $(a = 3.47 \text{in})$, to get an improved value for A_s.

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{136.6 \times 12{,}000}{0.9 \times 60{,}000 \times (18.75 - \frac{3.47}{2})} = 1.78 \text{ in}^2$$

According to ACI 318.10.5.1, the minimum reinforcement shall not be less than

$$A_s = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{5{,}000}}{60{,}000} \times 7.33 \times 18.75 = 0.49 \text{ in}^2$$

and not less than $\dfrac{200 b_w d}{f_y} = \dfrac{200 \times 7.33 \times 18.75}{60{,}000} = 0.46 \text{ in}^2$

ACI Code Section 10.6.6 states that "part" of the negative-moment steel shall be distributed over a width equal to the smaller of the effective flange width (72in) and $\dfrac{l}{10} = \dfrac{384}{10} = 38.4$ in.

Provide 6-No. 5 bars within 38.4 in width.

$$A_{s,prov} = (6 \times 0.31) = 1.86 \text{ in}^2 > 1.78 \text{ in}^2 \text{ O.K.}$$

Calculate the required reinforcement for the positive moment, $M_u = 98.2$ ft-kips

In the positive moment regions, the beam acts as a T-shaped beam. The effective width of the overhanging flange on each side of the beam web is the smallest of the following per ACI 318, 8.12.2

$$0.25l_n = 0.25 \times (30.17 \times 12) = 90.5 \text{ in}$$

$$b_w + 2 \times (8 \times 5) = 8.66 + 2 \times 40 = 88.66 \text{ in}$$

$$b_w + 5.5 \times 12 = 6 + 66 = 72 \text{ in}$$

Therefore, the effective flange width is 72 in

Assume tension-controlled section ($\phi = 0.9$). Note that this assumption shall be verified within the calculations below.

By assuming No. 3 bars for wide-module joist stirrups & the maximum bar size for joist bottom reinforcement as No. 7 and following the $1^{1/2"}$ concrete cover to reinforcement requirement of beam stirrups per ACI 318, 7.7, the distance from extreme compression fiber to the centroid of longitudinal tension reinforcement, d, is calculated below:

$$d = 21 - \left(1.5 + \frac{3}{8} + 0.5 \times \left(\frac{7}{8}\right)\right) = 18.69 \text{ in}$$

Since we are designing for the positive moment in a T-Beam (wide compression zone), select a moment arm, jd approximately equal to 0.95d.Assume that $jd = 0.95d = 0.95 \times 18.69 = 17.76$ in.

Required reinforcement @ initial trial, $A_s = \dfrac{M_u}{\phi f_y jd} = \dfrac{98.2 \times 12{,}000}{0.9 \times 60{,}000 \times 0.95 \times 18.69} = 1.23 \text{ in}^2$

Assume that the depth of the equivalent stress block, a, is less than the slab thickness, then, use one-iteration to refine this value by inserting it in the equation that finds the depth of the equivalent stress block.

Depth of equivalent stress block, $a = \dfrac{A_s f_y}{0.85 f'_c b}$

where b is the effective flange width of 72 in.

$a = \dfrac{1.23 \times 60}{0.85 \times 5 \times 72} = 0.241$ in which is less than the slab thickness. Therefore, it checks.

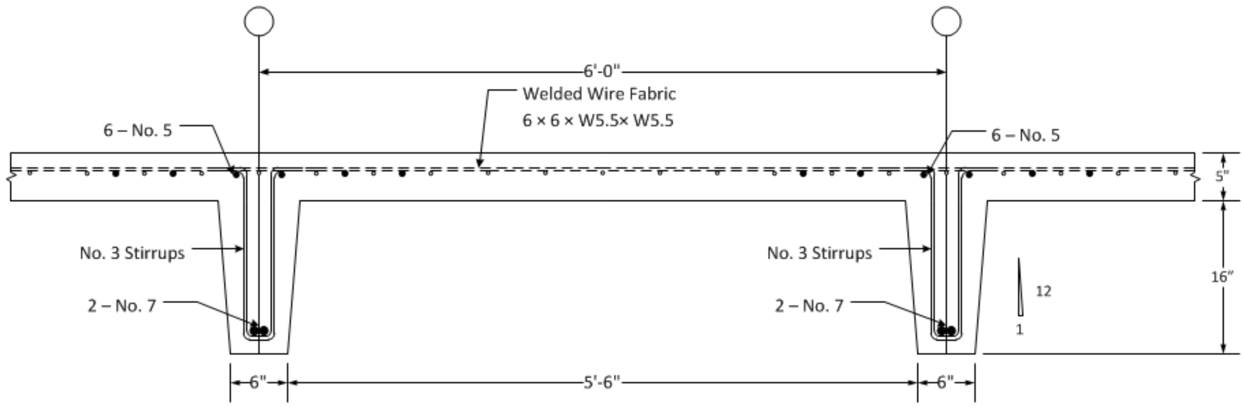
Neutral axis depth, $c = \dfrac{a}{\beta_1} = \dfrac{0.241}{0.85} = 0.284$ in.

Strain at tensile reinforcement, $\varepsilon_c = (\dfrac{0.003}{c})d_t - 0.003 = (\dfrac{0.003}{0.284})18.69 - 0.003 = 0.194 > 0.005$

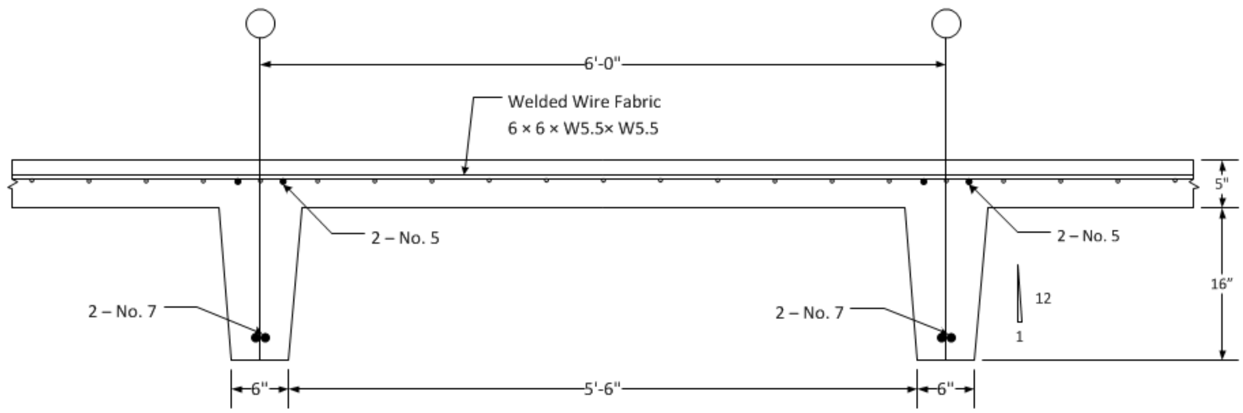
Therefore, the section is tension-controlled. Use the value of a, $(a = 0.241 \text{in})$, to get an improved value for A_s.

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{98.2 \times 12{,}000}{0.9 \times 60{,}000 \times (18.69 - \frac{0.241}{2})} = 1.18 \text{ in}^2$$

Use 2-No. 7 bundled bars with $A_s = 1.20 \text{ in}^2 > 1.18 \text{ in}^2$ O.K.



Section 2/2.2 Cross-sectional View at Joist mid-span



Section 3/2.2 Cross-sectional View at Joist near support face

2.2.4. Shear Design

From Table 2.2.2.2 above, the shear value in end span at face of first interior support governs.

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15 \times 1.51 \times 30.17}{2} = 26.2 \text{ kips}$$

The design shear at a distance, d, away from the face of support,

$$V_u = 26.2 - 1.51 \times \frac{18.69}{12} = 23.9 \text{ kips / ft}$$

Shear strength provided by concrete

$$\phi V_c = \phi(2\sqrt{f'_c} b_w d) = 0.75 \times \left(2 \times 1.0 \times \sqrt{5000} \times 7.33 \times 18.69\right) = 14531 \text{lb} = 14.5 \text{ kips}$$

Since $V_u > \frac{\phi V_c}{2}$, shear reinforcement is required.

Try No. 3, Grade 60 double-leg stirrups with a 90° hook.

The nominal shear strength required to be provided by steel is

$$V_s = V_n - V_c = \frac{V_u}{\phi} - V_c = (\frac{23.9}{0.75}) - 19.3 = 12.6 \text{ kips}$$

Check whether V_s is less than $8\sqrt{f'_c}b_w d$

If V_s is greater than $8\sqrt{f'_c}b_w d$, then the cross-section has to be increased as ACI 318-11.4.7.9 limits the shear capacity to be provided by stirrups to $8\sqrt{f'_c}b_w d$.

$$8\sqrt{f'_c}b_w d = 8\times\sqrt{5{,}000}\times 7.33\times 18.69 = 77498 \text{ lb } = 77.5 \text{ kips}$$

Since V_s does not exceed $8\sqrt{f'_c}b_w d$. The cross-section is adequate.

Assume No. 3 stirrups with two legs ($A_v = 0.22 \text{ in}^2$)

Calculate the required stirrup spacing as

$$s(\text{req'd}) = \frac{\phi A_v f_{yt} d}{V_u - \phi V_c} = \frac{0.75\times 0.22\times 60\times 18.69}{23.9 - 14.5} = 19.7 \text{ in.}$$

Check whether the required spacing based on the shear demand meets the spacing limits for shear reinforcement per ACI 318, 11.4.5.1.

Check whether V_s is less than $4\sqrt{f'_c}b_w d$

If V_s is greater than $4\sqrt{f'_c}b_w d$, then the maximum spacing of shear reinforcement given in 11.4.5.1 and 11.4.5.2 shall be reduced by one-half per ACI 318.11.4.5.3

$$4\sqrt{f'_c}b_w d = 4\times\sqrt{5{,}000}\times 7.33\times 18.69 = 38749 \text{ lb } = 38.75 \text{ kips}$$

Since V_s does not exceed $4\sqrt{f'_c}b_w d$, the maximum stirrups spacing limits per ACI 318.11.4.5.1 govern. Therefore, maximum stirrup spacing shall be the smallest of $\frac{d}{2}$ and 24 in.

$$s(\max) \le \text{Min}\left\{\frac{d}{2};24\right\} \text{ in } \le \text{Min}\left\{\frac{18.69}{2};24\right\} \text{ in}$$

$$s(\max) \le 9.35 \text{ in.}$$

This value governs over the required stirrup spacing of 19.7 in which was based on the demand.

For a wide-module joist, the minimum shear reinforcement requirements shall need to be checked as wide-module joists does not fit into the category of concrete joist construction defined by ACI 318, 8.13.

Check the maximum stirrup spacing based on minimum shear reinforcement

$$s(\max) \le \frac{A_v f_{yt}}{0.75\sqrt{f'_c}b_w} = \frac{0.22\times 60000}{0.75\times\sqrt{5000}\times 7.33} = 34 \text{ in (does not govern)}$$

$$s(\max) \le \frac{A_v f_{yt}}{50 b_w} = \frac{0.22\times 60000}{50\times 7.33} = 36 \text{ in (does not govern)}$$

Therefore, $s(\max)$ value is governed by the spacing limit per ACI 318, 11.4.5.1, and is equal to 9.35 in.

Use No. 3 @ 9 in. stirrups

$$V_n = \frac{A_v f_{yt} d}{s} + V_c = \frac{0.22\times 60\times 18.69}{9} + 19.3 = 27.4 + 19.3 = 46.7 \text{ kips}$$

$$\phi V_n = 0.75\times 46.7 = 35.0 \text{ kips} > V_u = 23.9 \text{ kips} \text{O.K.}$$

Compute where $\frac{V_u}{\phi}$ is equal to $\frac{V_c}{2}$, and the stirrups can be stopped

$$x = \frac{(\frac{V_u}{\phi}) - (\frac{V_c}{2})}{\frac{V_u}{\phi}} \times \frac{l_n}{2}$$

$$x = \frac{(\frac{23.9}{0.75}) - (\frac{19.3}{2})}{(\frac{23.9}{0.75})} \times \frac{30.17 \times 12}{2} = 126 \text{ in.}$$

At interior end of the exterior span, use 16-No. 3 @ 9 in o.c., Place 1st stirrup 2 in. from the face of supporting girder.

2.2.5. Deflections

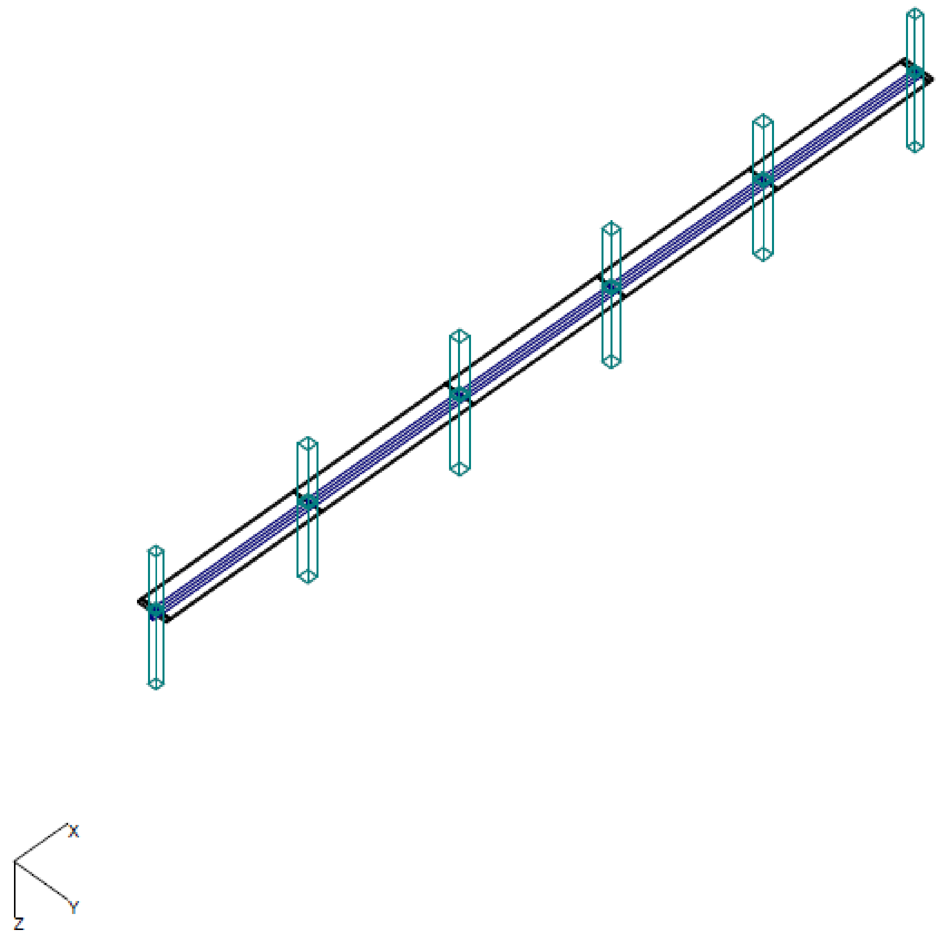
ACI 318 provides the minimum thickness of nonprestressed beams in Table 9.5(a) unless deflections are calculated. Since the preliminary beam depth met this requirement, the deflection calculations are waived here. Typically, in hand solutions, the designer opts to follow ACI 318, 9.5.2 in lieu of deflection calculations which can be long and tedious. It should be noted that, in most other cases, lesser thicknesses and consequent cost savings can be achieved through deflection computations.

2.2.6. Computer Program Solution

spSlab Program is utilized to design the one-way wide-module joist. The single wide-module joist is modeled as a T-beam.

The program calculates the internal forces (shear force and bending moment), moment and shear capacities, immediate and long-term deflection results, and required flexural reinforcement. The graphical and text results are provided below for both input and output of the spSlab model.

2.2.6.1 Isometric View of One-way Joist from spSlab



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\One-Way Joist\One-way Joist.slb

Project: Wide-Module Joist System

Frame: One-Way Joist

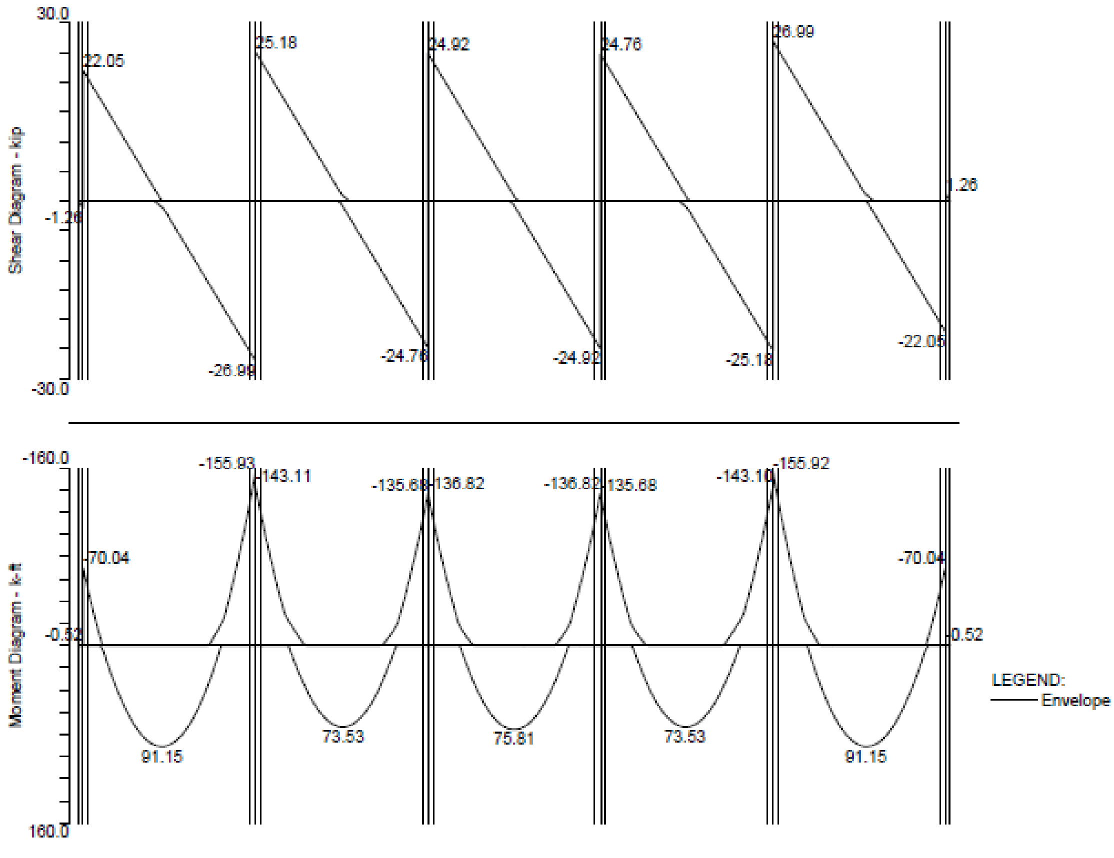
Engineer: MD

Code: ACI 318-11

Date: 11/25/13

Time: 14:54:39

2.2.6.2 spSlab Model Calculated Internal Forces (Shear Force & Bending Moment)



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\One-Way Joist\One-way Joist.slb

Project: Wide-Module Joist System

Frame: One-Way Joist

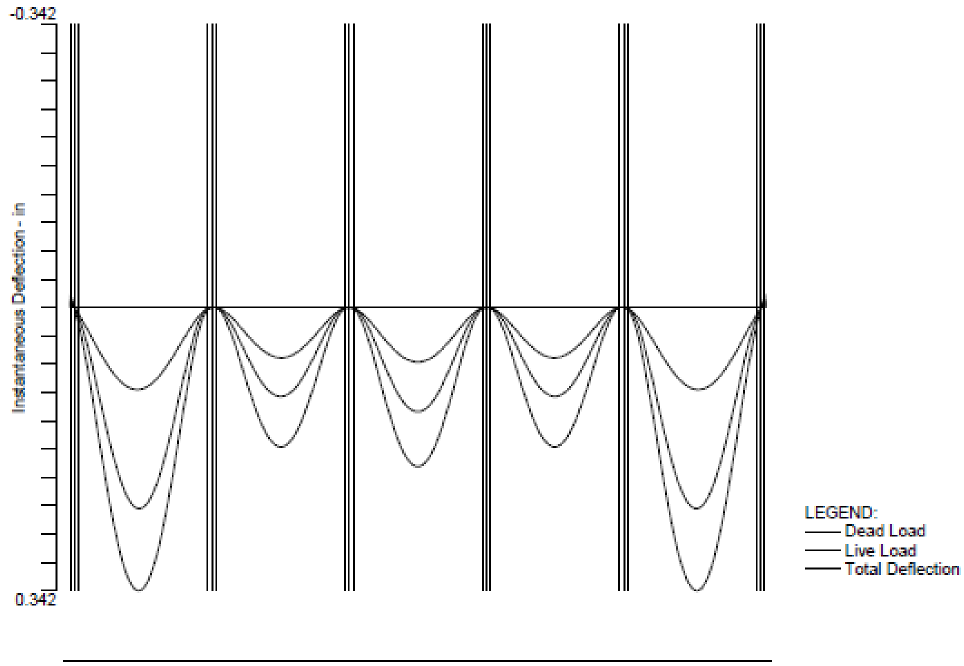
Engineer: MD

Code: ACI 318-11

Date: 12/05/13

Time: 12:02:18

2.2.6.3 spSlab Model Calculated Immediate Deflections



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\One-Way Joist\One-way Joist.slb

Project: Wide-Module Joist System

Frame: One-Way Joist

Engineer: MD

Code: ACI 318-11

Date: 12/04/13

Time: 14:32:41

2.2.7. Summary and Comparison of Design Results

```
==============================================================================================
[2] DESIGN RESULTS
==============================================================================================

Top Reinforcement
=================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span Zone        Width       Mmax      Xmax     AsMin    AsMax    AsReq   SpProv    Bars
   ---- ------- -------- ---------- -------- -------- -------- -------- -------- -------
      1 Left         3.20       0.00     0.000    0.197    2.989    0.000    8.442   5-#5 *3 *5
        Midspan      3.20       0.00     0.000    0.197    2.989    0.000    8.442   5-#5 *3 *5
        Right        3.20       0.00     0.000    0.197    2.989    0.000    8.442   5-#5 *3 *5

      2 Left         3.20      52.19     0.833    0.497    2.989    0.624    8.442   5-#5 *5
        Midspan      3.20       0.00    15.917    0.000    2.989    0.000    0.000    ---
        Right        3.20     129.69    31.000    0.497    2.989    1.636    6.754   6-#5

      3 Left         3.20     118.68     1.000    0.497    2.989    1.485    6.754   6-#5
        Midspan      3.20       0.00    16.000    0.000    2.989    0.000    0.000    ---
        Right        3.20     111.68    31.000    0.497    2.989    1.390    8.442   5-#5

      4 Left         3.20     112.65     1.000    0.497    2.989    1.404    8.442   5-#5
        Midspan      3.20       0.00    16.000    0.000    2.989    0.000    0.000    ---
        Right        3.20     112.65    31.000    0.497    2.989    1.404    8.442   5-#5


Bottom Reinforcement
====================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span    Width        Mmax      Xmax     AsMin    AsMax    AsReq   SpProv    Bars
   ---- -------- ---------- -------- -------- -------- -------- -------- -------
      1     0.61       0.00    0.000    0.000   25.893    0.000    0.000     ---

      2     0.61      91.15   14.710    0.494   25.873    1.068    2.522    2-#7

      3     0.61      73.53   16.300    0.496   25.883    0.858    2.610    2-#6

      4     0.61      75.81   16.000    0.494   25.873    0.888    2.522    2-#7


Longitudinal Beam Shear Reinforcement Details
=============================================
   Units: spacing & distance (in).
   Span Size Stirrups (2 legs each unless otherwise noted)
   ---- ---- ---------------------------------------------
     1   #3 --- None ---
     2   #3 12 @ 9.5 + <-- 92.5 --> + 17 @ 9.4
     3   #3 17 @ 9.4 + <-- 46.0 --> + 17 @ 9.4
     4   #3 17 @ 9.4 + <-- 46.0 --> + 17 @ 9.4
     5   #3 17 @ 9.4 + <-- 46.0 --> + 17 @ 9.4
     6   #3 17 @ 9.4 + <-- 92.5 --> + 12 @ 9.5
     7   #3 --- None ---
```

Table 2.2.7.1 – Comparison of Hand Solution with spSlab Solution				
Span Location	**Required Reinforcement Area for Flexure (in^2)**		**Reinforcement Provided**	
End Span	**Hand Solution**	**spSlab Solution**	**Hand Solution**	**spSlab Solution**
Interior Negative	1.78	1.610	6 – No. 5	6 – No. 5
Positive	1.18	1.008	2 – No. 7	2 – No. 7

2.2.8. Conclusions and Observations

In this design example, the one-way joist system is modeled as a continuous T-beam representing single one-way joist. There is a good agreement between the hand solution and computer solution.

2.3. Design of a Continuous Beam along Grid B (Interior Frame)

Design of the interior beam along grid B shall be performed. In the wide-module joist construction, the supporting beam depths shall be same as the overall joist depth. Therefore, the beam depth is set at 21 in. This depth satisfies the minimum thickness requirement of ACI 318, Table 9.5 (a) so that the deflection computations can be waived.

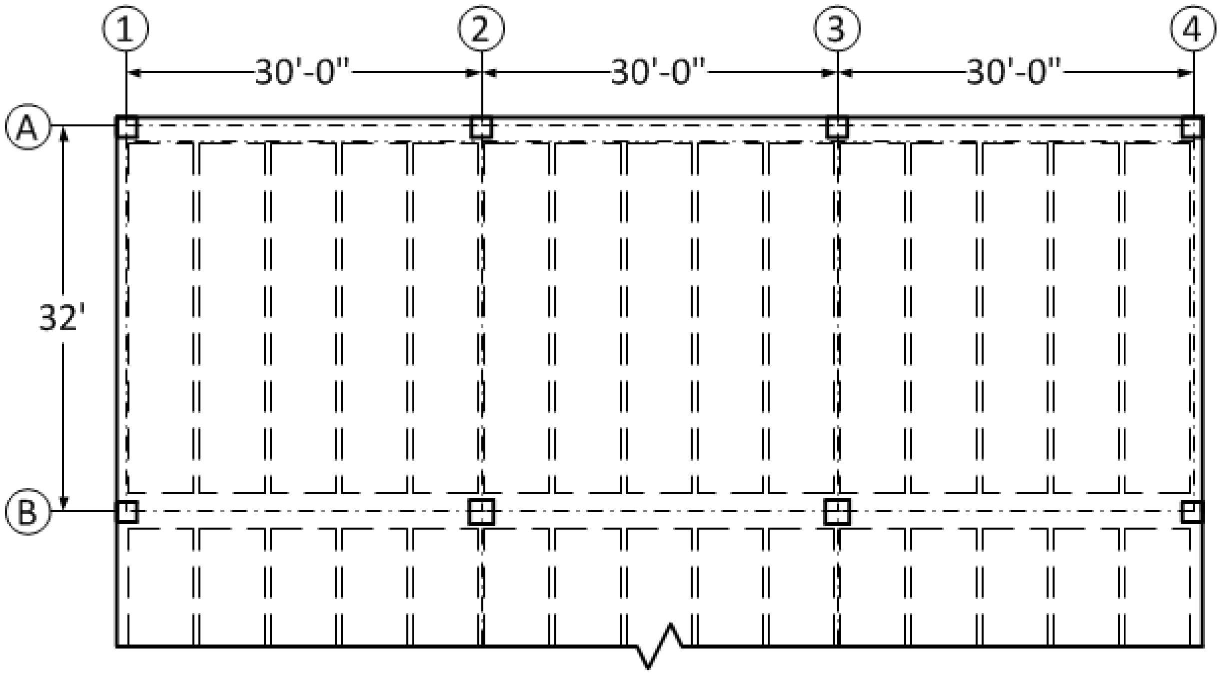


Figure 2.3 – Partial plan view showing interior beam along grid B

The design involves the following steps:

- 2.3.1.Determination of span loads
- 2.3.2.Determination of design moments and shears
- 2.3.3.Flexural Design
- 2.3.4.Shear Design
- 2.3.5.Deflections
- 2.3.6.Computer Program Solution
- 2.3.7.Summary and comparison of design results
- 2.3.8.Conclusions and observations

2.3.1.Determination of span loads

ACI 318, 9.2.1 gives the following load combinations for structural members loaded with dead and live loads:

$U = 1.4D$ ACI 318, Eq. 9-1

$U = 1.2D + 1.6L$ ACI 318, Eq. 9-2

Check Floor Live Load Reduction per ASCE 7-10, sections 4.8.

$$L = L_o \times (0.25 + \frac{15}{\sqrt{K_{LL} A_T}})$$ ASCE 7-10, Eq (4-1)

where

Live Load Element Factor, $K_{LL} = 2$ for interior beams [ASCE/SEI 7-10, Table 4-2]

Tributary Area $A_T = (30'\text{–}0'' \times 32'\text{–}0'') = 960 \text{ ft}^2$

$L_o = 80 \text{ psf}$

$L = 80 \times (0.25 + \frac{15}{\sqrt{2 \times 960}}) = 47.4 \text{ psf}$ which satisfies $0.50 \times L_o$ requirement for members supporting only one floor per ASCE 7-10, section 4.8.1

Live Load, $L = 0.0474 \times 32 = 1.52 \text{ klf}$

Try 36 in width for the beam (slightly larger than the column width that helps facilitate the forming, and reduces the beam longitudinal vs. column vertical bar interference)

$$\text{Joist \& Slab Weight} = \left[\frac{5}{12} + \left\{\left(\left(\frac{(6+8.66)/2}{12}\right) \times \frac{16}{12}\right)/6\right\}\right] \times 0.15 \times \left(32 - \frac{36}{12}\right) = 2.40 \text{ klf}$$

$$\text{Beam Weight} = \left(\frac{21}{12} \times \frac{36}{12}\right) \times 0.15 = 0.79 \text{ klf}$$

Superimposed Dead Load, SDL $= 0.02 \times 32 = 0.64 \text{ klf}$

Factored total load per Eq. 9-1: $U = 1.4 \times (2.40 + 0.79 + 0.64) = 5.36 \text{ klf}$

Factored total load per Eq. 9-2: $U = 1.2 \times (2.40 + 0.79 + 0.64) + 1.6 \times 0.0474 \times 32 = 7.02 \text{ klf}$

Span loads are governed by load combination per Eq. 9-2.

2.3.2. Determination of span loads

Using the approximate coefficients of ACI 318, 8.3.3, the factored moments, and shears are summarized in the Table 2.3.2.1, and 2.3.2.2 respectively below.

Table 2.3.2.1 –Interior Beam Design Moments

Location		Design Moment Value
End Spans	Exterior Support Negative	$\frac{w_u l_n^2}{24} = \frac{7.02 \times 28.17^2}{16} = 348.2$ ft-kips
	Mid-span Positive	$\frac{w_u l_n^2}{14} = \frac{7.02 \times 28.17^2}{14} = 397.9$ ft-kips
	Interior Support Negative	$\frac{w_u l_n^2}{10} = \frac{7.02 \times 28.08^2}{10} = 553.5$ ft-kips
Interior Spans	Mid-span Positive	$\frac{w_u l_n^2}{16} = \frac{7.02 \times 28^2}{16} = 344$ ft-kips
	Support Negative	$\frac{w_u l_n^2}{11} = \frac{7.02 \times 28^2}{11} = 500.3$ ft-kips

Table 2.3.2.2 – Interior Beam Design Shears	
Location	Design Shear Value
End Span at Face of First Interior Support	$1.15\times\frac{w_u l_n}{2}=1.15\times\frac{7.02\times 28.17}{2}=113.7\ \text{kips}$
At Face of all other Supports	$\frac{w_u l_n}{2}=\frac{7.02\times 28.17}{2}=98.9\ \text{kips}$

2.3.3. Flexural Design

For this interior beam, the end span moment values govern the design as tabulated in Table 2.3.2.1. Calculate the required reinforcement to resist the first interior support negative moment:

$$M_u = 553.5\ \text{ft-kips}$$

Assume tension-controlled section ($\phi = 0.9$).Note that this assumption shall be verified within the calculations below.

Clear cover to the stirrup is 1.5 in. per ACI 318, 7.7.1. However, in order to avoid interference with joist negative moment reinforcement, the clear cover to the girder top reinforcement is required to be increased by lowering the girder top reinforcement.

By assuming the maximum bar size for beam top reinforcement as No. 8, the distance from extreme compression fiber to the centroid of longitudinal tension reinforcement, d, is calculated below:

$$d = 21-\left(1.5+\frac{3}{8}+\frac{6}{8}+0.5\times\left(\frac{8}{8}\right)\right)=17.88\ \text{in}$$

Since we are designing for the negative moment in a Rectangular Beam (narrow compression zone), select a moment arm, jd approximately equal to 0.9d. Assume that $jd = 0.9d = 0.9\times 17.88 = 16.09$ in.

$$\text{Required reinforcement @ initial trial, } A_s = \frac{M_u}{\phi f_y\left(d-a/2\right)}=\frac{M_u}{\phi f_y\left(jd\right)}=\frac{553.5\times 12{,}000}{0.9\times 60{,}000\times 16.09}$$

$$A_s = 7.64\ \text{in}^2$$

Use one-iteration to refine this value by inserting it in the equation that finds the depth of the equivalent stress block.

$$\text{Depth of equivalent stress block, } a = \frac{A_s f_y}{0.85 f'_c b}=\frac{7.64\times 60{,}000}{0.85\times 5{,}000\times 36}=3.0\ \text{in.}$$

$$\text{Neutral axis depth, } c = \frac{a}{\beta_1}=\frac{3.0}{0.85}=3.53\ \text{in.}$$

$$\text{Strain at tensile reinforcement, } \beta_t = \left(\frac{0.003}{c}\right)d_t - 0.003 = \left(\frac{0.003}{3.53}\right)17.88-0.003=0.0122>0.005$$

Therefore, the section is tension-controlled. Use the value of a, $(a = 3.0\text{in})$, to get an improved value for A_s.

$$A_s = \frac{M_u}{\phi f_y\left(d-\frac{a}{2}\right)}=\frac{553.5\times 12{,}000}{0.9\times 60{,}000\times\left(17.88-\frac{3.0}{2}\right)}=7.51\ \text{in}^2$$

According to ACI 318.10.5.1, the minimum reinforcement shall not be less than

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{5{,}000}}{60{,}000} \times 36 \times 17.88 = 2.28 \text{ in}^2$$

and not less than $\dfrac{200 b_w d}{f_y} = \dfrac{200 \times 36 \times 17.88}{60{,}000} = 2.15 \ \text{ in}^2$

Provide 10 – No. 8 bars with provided reinforcement area of 7.90 in^2 which is greater than required reinforcement of 7.51 in^2.

Check the requirement for distribution of flexural reinforcement to control flexural cracking per ACI 318-11, 10.6.

Maximum spacing allowed,

$$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c \le 12\left(\frac{40000}{f_s}\right)$$

$$c_c = 21.0 - \left(17.88 + 0.5 \times (8/8)\right) = 2.62 \text{ in.}$$

Use $f_s = \dfrac{2}{3} f_y = 40$ ksi

$$s = 15 \times \left(\frac{40000}{40000}\right) - 2.5 \times 2.62 = 8.45 \text{ in. (governs)}$$

$$s = 12 \times \left(\frac{40000}{40000}\right) = 12 \text{ in.}$$

Spacing provided for 10-No. 8 bars $= \dfrac{1}{9} \times \{36 - 2 \times 2.625\} = 3.42$ in. < 8.45 in. O.K.

Check the spacing, s provided, is greater than the minimum center to center spacing, s_{min} where

$$s_{min} = d_b + \max\begin{Bmatrix} 1 \\ d_b \\ 1.33 \times \max.\text{agg.} \end{Bmatrix}$$

where maximum aggregate size is ¾”

$$s_{min} = 1 + 1 = 2 \text{ in}$$

Since the spacing provided is greater than 2 in. Therefore, 10 – No. 8 bars are O.K.

All the values on Table 2.3.3.1 are calculated based on the procedure outlined above.

Table 2.3.3.1 – Reinforcing Design Summary					
	End Span			**Interior Span**	
	Exterior Negative	Positive	Interior Negative	Positive	Negative
Design Moment, M_u (ft-kips)	348.2	397.9	553.5	344	500.3
Effective depth, d (in)	17.88*	19.0**	17.88	19.0	17.88
A_s req’d (in^2)	4.57	4.92	7.51	4.22	6.73
A_s min (in^2)	2.28	2.42	2.28	2.42	2.28
Reinforcement	6-No. 8	7-No. 8	10-No. 8	6-No. 8	9-No. 8

* The beam top bars are to be placed below the joist top bars.

** The beam bottom bars are to be placed at the bottom-most layer. The joist bottom bars, then, shall be spliced at joist-beam intersection.

2.3.4. Shear Design

From Table 2.3.2.2 above, the shear value in end span at face of first interior support governs.

Shear in end members at face of first interior support

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15 \times 7.02 \times 28.17}{2} = 113.7 \text{ kips}$$

The design shear at a distance, d, away from the face of support,

$$V_u = 113.7 - 7.02 \times \frac{17.88}{12} = 103.2 \text{ kips}$$

Shear strength provided by concrete

$$\phi V_c = \phi(2\sqrt{f'_c}\, b_w d) = 0.75 \times \left(2 \times 1.0 \times \sqrt{5000} \times 36 \times 17.88\right) = 68273 \text{ lb} = 68.3 \text{ kips}$$

Since $V_u > \frac{\phi V_c}{2}$, shear reinforcement is required.

Try No. 3, Grade 60 four-leg stirrups ($A_v = 0.44 \text{ in}^2$) with a 90° hook.

The nominal shear strength required to be provided by steel is

$$V_s = V_n - V_c = \frac{V_u}{\phi} - V_c = \left(\frac{103.2}{0.75}\right) - 91.1 = 46.5 \text{ kips}$$

Check whether V_s is less than $8\sqrt{f'_c}\, b_w d$

If V_s is greater than $8\sqrt{f'_c}\, b_w d$, then the cross-section has to be increased as ACI 318-11.4.7.9 limits the shear capacity to be provided by stirrups to $8\sqrt{f'_c}\, b_w d$.

$$8\sqrt{f'_c}\, b_w d = 8 \times \sqrt{5{,}000} \times 36 \times 17.88 = 364120 \text{ lb} = 364.1 \text{ kips}$$

Since V_s does not exceed $8\sqrt{f'_c}\, b_w d$. The cross-section is adequate.

Calculate the required stirrup spacing as

$$s(\text{req'd}) = \frac{\phi A_v f_{yt} d}{V_u - \phi V_c} = \frac{0.75 \times 0.44 \times 60 \times 17.88}{103.2 - 68.3} = 10.1 \text{ in.}$$

Check whether the required spacing based on the shear demand meets the spacing limits for shear reinforcement per ACI 318, 11.4.5.1.

Check whether V_s is less than $4\sqrt{f'_c}\, b_w d$

If V_s is greater than $4\sqrt{f'_c}\, b_w d$, then the maximum spacing of shear reinforcement given in 11.4.5.1 and 11.4.5.2 shall be reduced by one-half per ACI 318.11.4.5.3

$$4\sqrt{f'_c}\, b_w d = 4 \times \sqrt{5{,}000} \times 36 \times 17.88 = 182060 \text{ lb} = 182.1 \text{ kips}$$

Since V_s does not exceed $4\sqrt{f'_c}\, b_w d$, the maximum stirrups spacing limits per ACI 318.11.4.5.1 govern. Therefore, maximum stirrup spacing shall be the smallest of $\frac{d}{2}$ and 24 in.

$$s(max) \le \text{Min}\left\{\frac{d}{2};24\right\} \text{ in } \le \text{Min}\left\{\frac{17.88}{2};24\right\} \text{ in}$$

$$s(max) \le 8.94 \text{ in.}$$

This value governs over the required stirrup spacing of 10.1 in based on the demand.

Check the maximum stirrup spacing based on minimum shear reinforcement

$$s(max) \le \frac{A_v f_{yt}}{0.75\sqrt{f'_c}b_w} = \frac{0.44 \times 60000}{0.75 \times \sqrt{5000} \times 36} = 13.9 \text{ in (does not govern)}$$

$$s(max) \le \frac{A_v f_{yt}}{50b_w} = \frac{0.44 \times 60000}{50 \times 36} = 14.7 \text{ in (does not govern)}$$

Therefore, $s(max)$ value is governed by the spacing limit per ACI 318, 11.4.5.1, and is equal to 8.94 in.

Use No. 3 @ 8 in. stirrups

$$V_n = \frac{A_v f_{yt} d}{s} + V_c = \frac{0.44 \times 60 \times 17.88}{8} + 91.1 = 59.0 + 91.1 = 150.1 \text{ kips}$$

$$\phi V_n = 0.75 \times 150.1 = 112.6 \text{ kips} > V_u = 103.2 \text{ kips} \text{O.K.}$$

Compute where $\frac{V_u}{\phi}$ is equal to $\frac{V_c}{2}$, and the stirrups can be stopped

$$x = \frac{\left(\frac{V_u}{\phi}\right) - \left(\frac{V_c}{2}\right)}{\frac{V_u}{\phi}} \times \frac{l_n}{2}$$

$$x = \frac{\left(\frac{103.2}{0.75}\right) - \left(\frac{91.1}{2}\right)}{\left(\frac{103.2}{0.75}\right)} \times \frac{28.17 \times 12}{2} = 113 \text{ in.}$$

At interior end of the exterior span, use 16-No. 3 @ 8 in o.c., Place 1st stirrup 2 in. from the face of the column.

2.3.5. Deflections

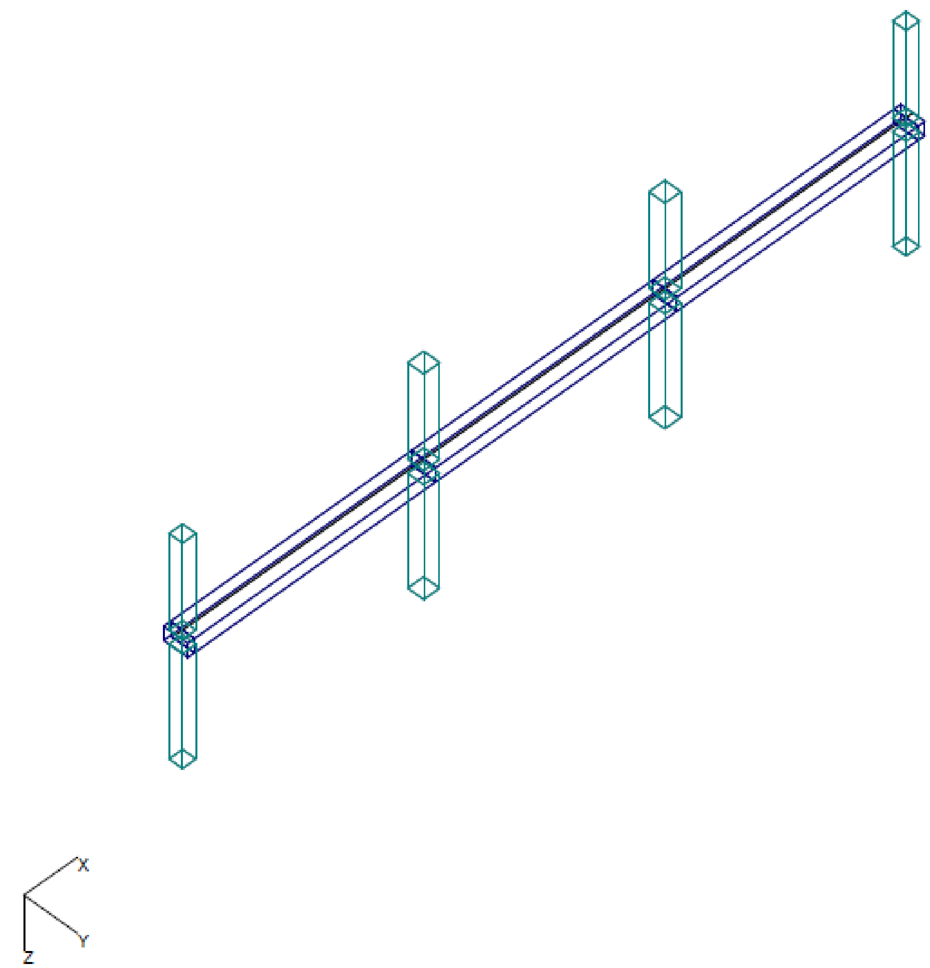
ACI 318 provides the minimum thickness of nonprestressed beams in Table 9.5(a) unless deflections are calculated. Since the preliminary beam depth met this requirement, the deflection calculations are waived here. Typically, in hand solutions, the designer opts to follow ACI 318, 9.5.2 in lieu of deflection calculations which can be long and tedious. It should be noted that, in most other cases, lesser thicknesses and consequent cost savings can be achieved through deflection computations.

2.3.6. Computer Program Solution

spSlab Program is utilized to design the interior beam. The interior beam is modeled as a rectangular beam.

The program calculates the internal forces (shear force and bending moment), moment and shear capacities, immediate and long-term deflection results, and required flexural reinforcement. The graphical and text results are provided below for both input and output of the spSlab model.

2.3.6.1. Isometric View of Interior Beam from spSlab



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\Interior Beam Design\Interior Beam Design.slb

Project: Wide-Module Joist System

Frame: Interior Beam

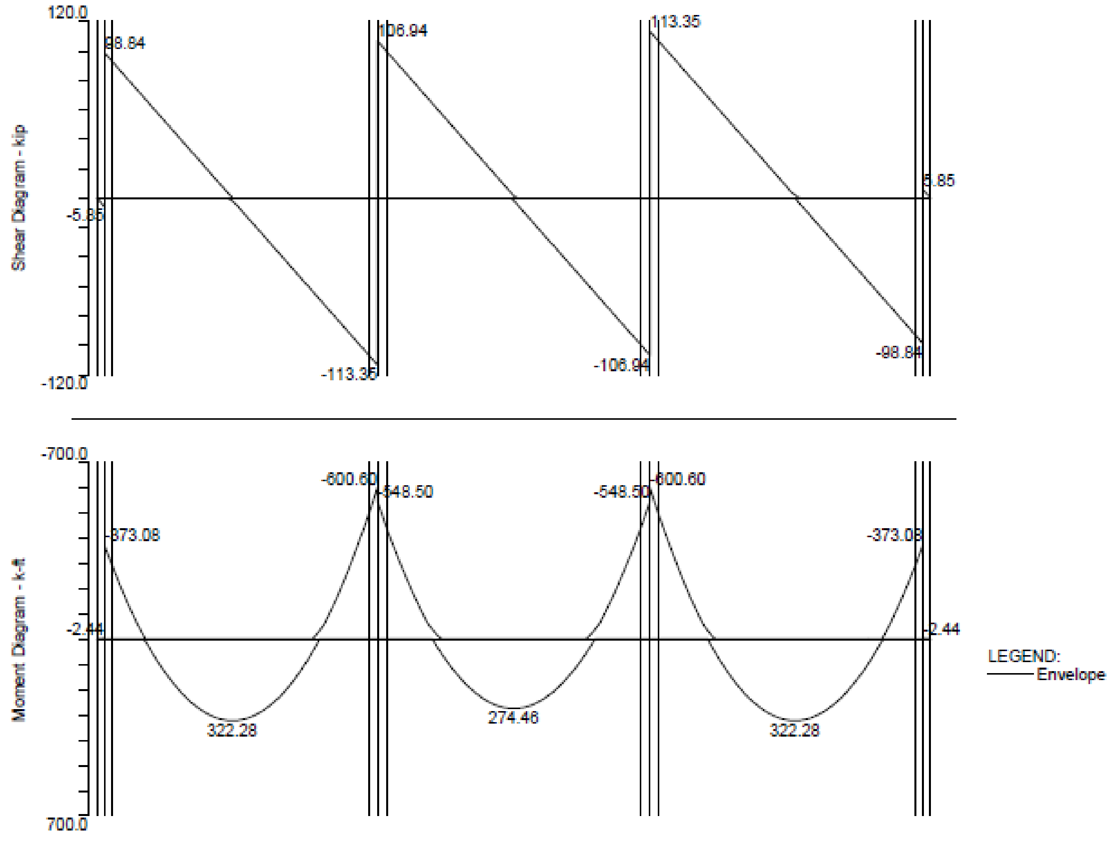
Engineer:

Code: ACI 318-11

Date: 12/05/13

Time: 15:37:12

2.3.6.2. spSlab Model Calculated Internal Forces (Shear Force & Bending Moment)



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\Interior Beam Design\Interior Beam Design.slb

Project: Wide-Module Joist System

Frame: Interior Beam

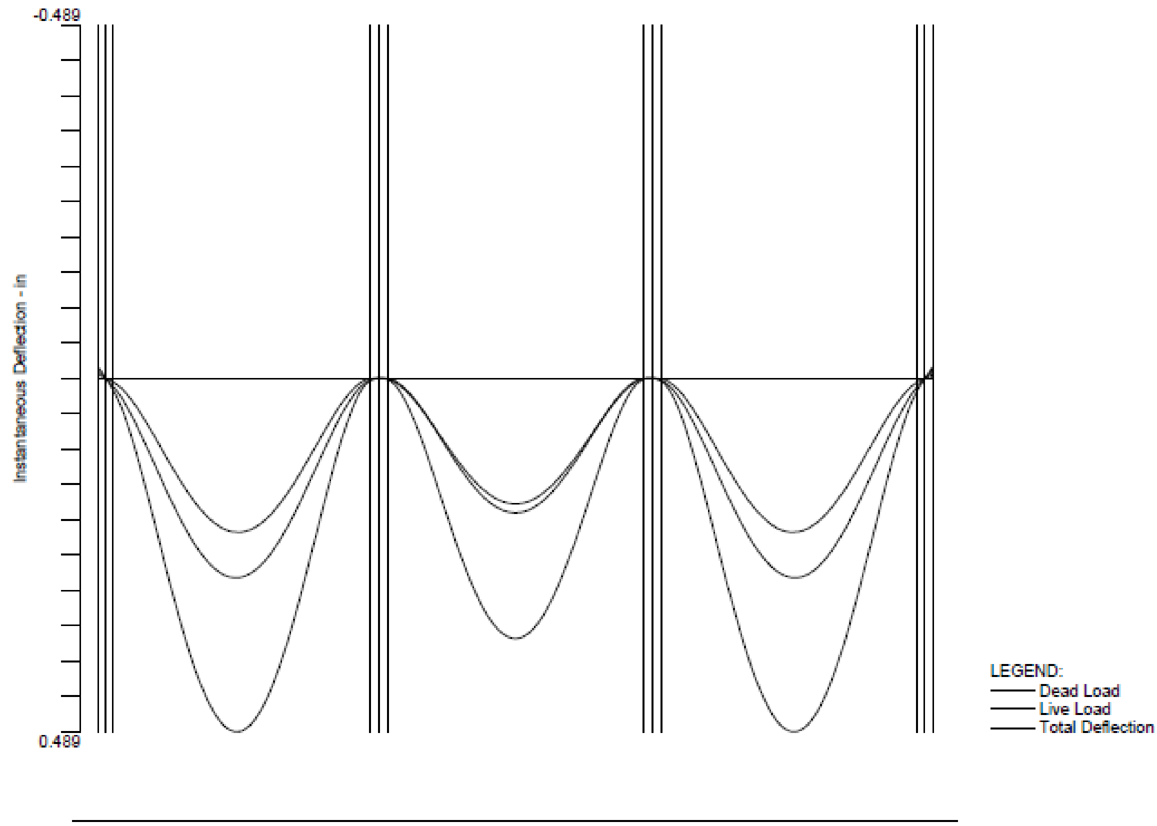
Engineer:

Code: ACI 318-11

Date: 12/05/13

Time: 15:52:55

2.3.6.3. spSlab Model Calculated Immediate Deflections



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\Interior Beam Design\Interior Beam Design.slb

Project: Wide-Module Joist System

Frame: Interior Beam

Engineer:

Code: ACI 318-11

Date: 12/05/13

Time: 16:03:27

2.3.7. Summary and Comparison of Design Results

```
==============================================================================================
[2] DESIGN RESULTS
==============================================================================================

Top Reinforcement
=================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span Zone      Width        Mmax     Xmax    AsMin    AsMax    AsReq   SpProv    Bars
   ---- ------- -------- ---------- -------- -------- -------- -------- -------- -------
      1 Left       3.00        0.00    0.000    0.901   13.674    0.000    7.776   5-#8 *3 *5
        Midspan    3.00        0.00    0.000    0.901   13.674    0.000    7.776   5-#8 *3 *5
        Right      3.00        0.00    0.000    0.901   13.674    0.000    7.776   5-#8 *3 *5

      2 Left       3.00      293.15    0.833    2.275   13.674    3.803    7.776   5-#8
        Midspan    3.00        0.00   14.917    0.000   13.674    0.000    0.000   ---
        Right      3.00      490.77   29.000    2.275   13.674    6.576    3.888   9-#8

      3 Left       3.00      445.07    1.000    2.275   13.674    5.917    3.888   9-#8
        Midspan    3.00        0.00   15.000    0.000   13.674    0.000    0.000   ---
        Right      3.00      445.07   29.000    2.275   13.674    5.917    3.888   9-#8
```

```
Bottom Reinforcement
====================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span    Width        Mmax      Xmax    AsMin    AsMax    AsReq   SpProv    Bars
   ---- -------- ---------- -------- -------- -------- -------- -------- -------
      1     3.00        0.00    0.000    0.000   14.153    0.000    0.000     ---

      2     3.00      322.28   14.072    2.355   14.153    4.045    6.221    6-#8

      3     3.00      274.46   15.000    2.355   14.153    3.421    7.776    5-#8
```

```
Longitudinal Beam Shear Reinforcement Details
=============================================
   Units: spacing & distance (in).
   Span Size Stirrups (2 legs each unless otherwise noted)
   ---- ---- -----------------------------------------
      1   #3 --- None ---
      2   #3 12 @ 8.8 [4L] + <-- 86.4 --> + 17 @ 8.8 [4L]
      3   #3 17 @ 8.7 [4L] + <-- 42.9 --> + 17 @ 8.7 [4L]
      4   #3 17 @ 8.8 [4L] + <-- 86.4 --> + 12 @ 8.8 [4L]
      5   #3 --- None ---
```

Table 2.3.7.1 – Comparison of Hand Solution with spSlab Solution

Span Location	**Required Reinforcement Area for Flexure (in^2)**		**Reinforcement Provided**	
End Span	**Hand Solution**	**spSlab Solution**	**Hand Solution**	**spSlab Solution**
Interior Negative	7.51	6.576	10 – No. 8	9 – No. 8
Positive	4.92	4.045	7 – No. 8	6 – No. 8

2.3.8. Conclusions and Observations

In this design example, the interior beam is modeled as a continuous rectangular beam. There is a good agreement between the hand solution and computer solution.

2.4. Design of a Continuous Beam along Grid A (Exterior Frame)

Design of the interior beam along grid B shall be performed. In the wide-module joist construction, the supporting beam depths shall be same as the overall joist depth. Therefore, the beam depth is set at 21 in. This depth satisfies the minimum thickness requirement of ACI 318, Table 9.5 (a) so that the deflection computations can be waived. The beams of the exterior frame shall be designed and detailed for the combined effects of flexure, shear, and torsion according to ACI 318.

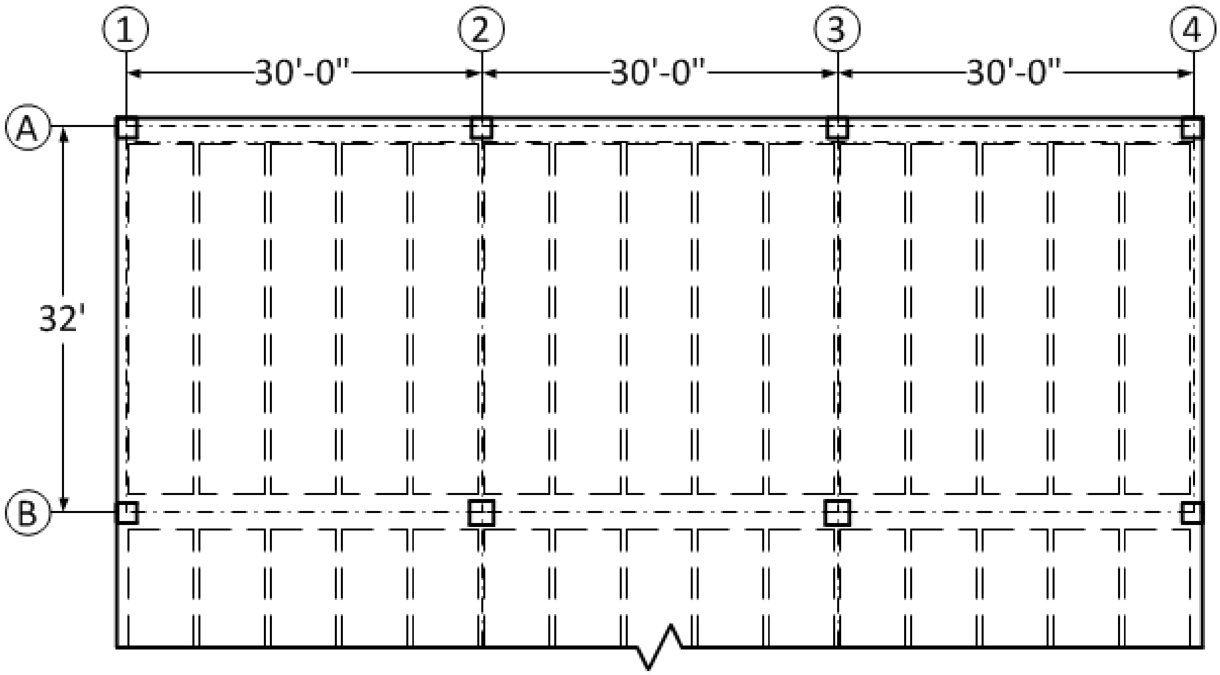


Figure 2.4 – Partial plan view showing exterior beam along grid A

The design involves the following steps:

2.4.1.Determination of span loads
2.4.2.Determination of design moments and shears
2.4.3.Flexural and Torsion Design
2.4.4.Shear Design
2.4.5.Deflections
2.4.6.Computer Program Solution
2.4.7.Summary and comparison of design results
2.4.8.Conclusions and observations

2.4.1 Determination of span loads

ACI 318, 9.2.1 gives the following load combinations for structural members loaded with dead and live loads:

$U = 1.4D$ ACI 318, Eq. 9-1

$U = 1.2D + 1.6L$ ACI 318, Eq. 9-2

Check Floor Live Load Reduction per ASCE 7-10, sections 4.8.

$$L = L_o \times (0.25 + \frac{15}{\sqrt{K_{LL} A_T}})\ \text{ASCE 7-10, Eq (4-1)}$$

where

$K_{LL} = 2$ for edge beams without cantilever slabs

Tributary Area $A_T = (30'\text{–}0" \times 16'\text{–}10") = 505\ \text{ft}^2$

$L_o = 80\ \text{psf}$

$L = 80 \times (0.25 + \frac{15}{\sqrt{2 \times 505}}) = 57.8$ psf which satisfies $0.50 \times L_o$ requirement for members supporting only one floor per ASCE 7-10, section 4.8.1

$$\text{Live Load, L} = 0.0578 \times \left(16 + \frac{10}{12}\right) = 0.97\ \text{klf}$$

Try 24 in width for the beam (slightly larger than the column width that helps facilitate the forming, and reduces the beam longitudinal vs. column vertical bar interference)

$$\text{Joist \& Slab Weight} = \left[\frac{5}{12} + \left\{\left(\left(\frac{(6+8.66)/2}{12}\right) \times \frac{16}{12}\right)/6\right\}\right] \times 0.15 \times \left(16 - \frac{24}{2 \times 12}\right) = 1.24\ \text{klf}$$

$$\text{Beam Weight} = \left(\frac{21}{12} \times \frac{24}{12}\right) \times 0.15 = 0.525\ \text{klf}$$

Superimposed Dead Load, SDL $= 0.02 \times (16 + 10/12) = 0.34$ klf

Factored total load per Eq. 9-1: $U = 1.4 \times (1.24 + 0.525 + 0.34) = 2.95$ klf

Factored total load per Eq. 9-2: $U = 1.2 \times (1.24 + 0.525 + 0.34) + 1.6 \times 0.0578 \times \left(16 + \frac{10}{12}\right) = 4.08$ klf

Span loads are governed by load combination per Eq. 9-2.

2.4.2 Determination of span loads

Using the approximate coefficients of ACI 318, 8.3.3, the factored moments, and shears are summarized in the Table 2.4.2.1, and 2.4.2.2 respectively below.

Table 2.4.2.1 –Exterior Beam Design Moments

Location		Design Moment Value
End Spans	Exterior Support Negative	$\frac{w_u l_n^2}{24} = \frac{4.08 \times 28.17^2}{16} = 202.4$ ft-kips
	Mid-span Positive	$\frac{w_u l_n^2}{14} = \frac{4.08 \times 28.17^2}{14} = 231.3$ ft-kips
	Interior Support Negative	$\frac{w_u l_n^2}{10} = \frac{4.08 \times 28.08^2}{10} = 321.7$ ft-kips
Interior Spans	Mid-span Positive	$\frac{w_u l_n^2}{16} = \frac{4.08 \times 28^2}{16} = 199.9$ ft-kips
	Support Negative	$\frac{w_u l_n^2}{11} = \frac{4.08 \times 28^2}{11} = 290.8$ ft-kips

Table 2.4.2.2 – Exterior Beam Design Shears	
Location	Design Shear Value
End Span at Face of First Interior Support	$1.15\times\frac{w_u l_n}{2}=1.15\times\frac{4.08\times 28.17}{2}=66.1\text{ kips}$
At Face of all other Supports	$\frac{w_u l_n}{2}=\frac{4.08\times 28.17}{2}=57.5\text{ kips}$

2.4.3. Flexural, Shear, and Torsion Design

For this exterior beam, the end span moment values govern the design as tabulated in Table 2.4.2.1.

Calculate the required reinforcement for negative flexural moment, $M_u = 321.7$ ft-kips

Assume tension-controlled section ($\phi = 0.9$).Note that this assumption shall be verified within the calculations below.

Clear cover to the stirrup is 1.5 in. per ACI 318, 7.7.1. However, in order to avoid interference with joist negative moment reinforcement, the clear cover to the girder top reinforcement is required to be increased by lowering the girder top reinforcement.

By assuming the maximum bar size for beam top reinforcement as No. 8, the distance from extreme compression fiber to the centroid of longitudinal tension reinforcement, d, is calculated below:

$$d = 21-\left(1.5+\frac{3}{8}+\frac{6}{8}+0.5\times\left(\frac{8}{8}\right)\right)=17.88\text{ in}$$

Since we are designing for the negative moment in a Rectangular Beam (narrow compression zone), select a moment arm, jd approximately equal to 0.9d.Assume that $jd = 0.9d = 0.9\times 17.88 = 16.09$ in.

$$\text{Required reinforcement @ initial trial, } A_s = \frac{M_u}{\phi f_y\left(d-a/2\right)}=\frac{M_u}{\phi f_y\left(jd\right)}=\frac{321.7\times 12{,}000}{0.9\times 60{,}000\times 16.09}$$

$$A_s = 4.44\text{ in}^2$$

Use one-iteration to refine this value by inserting it in the equation that finds the depth of the equivalent stress block.

$$\text{Depth of equivalent stress block, } a = \frac{A_s f_y}{0.85 f_c' b}=\frac{4.44\times 60{,}000}{0.85\times 5{,}000\times 24}=2.61\text{ in.}$$

$$\text{Neutral axis depth, } c = \frac{a}{\beta_1}=\frac{2.61}{0.85}=3.07\text{ in.}$$

$$\text{Strain at tensile reinforcement, } \beta_t = \left(\frac{0.003}{c}\right)d_t - 0.003 = \left(\frac{0.003}{3.07}\right)17.88-0.003 = 0.014 > 0.005$$

Therefore, the section is tension-controlled. Use the value of a, $(a = 2.61\text{in})$, to get an improved value for A_s.

$$A_s = \frac{M_u}{\phi f_y\left(d-\frac{a}{2}\right)}=\frac{321.7\times 12{,}000}{0.9\times 60{,}000\times\left(17.88-\frac{2.61}{2}\right)}=4.31\text{ in}^2$$

According to ACI 318.10.5.1, the minimum reinforcement shall not be less than

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d = \frac{3\sqrt{5{,}000}}{60{,}000} \times 24 \times 17.88 = 1.52\,\text{in}^2$$

and not less than $\dfrac{200 b_w d}{f_y} = \dfrac{200 \times 24 \times 17.88}{60{,}000} = 1.51\ \text{in}^2$

All the values on Table 2.4.3.1 are calculated based on the procedure outlined above.

Table 2.4.3.1 – Reinforcing Design Summary (Flexure only)					
	End Span			**Interior Span**	
	Exterior Negative	Positive	Interior Negative	Positive	Negative
Design Moment, M_u (ft-kips)	202.4	231.3	321.7	199.9	290.8
Effective depth, d (in)	17.88	19.0	17.88	19.0	17.88
A_s req'd (in^2)	2.64	2.84	4.31	2.44	3.87
A_s min (in^2)	1.52	1.61	1.52	1.61	1.52

The actual selection of reinforcing bars will be delayed until the torsion requirements for longitudinal steel have been determined.

Calculate the factored Torsional Moment, T_u

Since beam between grids 1 and 2 is part of an indeterminate framing system in which redistribution of internal forces can occur following torsional cracking, the maximum factored torsional moment T_u at the critical section located at a distance d from the face of the support can be determined from the following per ACI 318, 11.5.2.2 (a):

$$T_u = \phi 4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

This type of torsion is referred to as compatibility torsion, the magnitude of which is greater than the factored torsional moment, $T_{u,min}$, below which torsional effects can be neglected, where

$$T_{u,min} = \phi\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

Since the beams are cast monolithically with slab and joists, A_{cp} and p_{cp} for the beam can include a portion of the adjoining slab. The effective width, b_e, of the overhanging flange must conform to ACI 318, 13.2.4:

$$b_e = h - h_f = 21 - 5 = 16.0\ \text{in. (governs)}$$

$$b_e = 4h_f = 4 \times 5 = 20.0\ \text{in.}$$

$$A_{cp} = (21 \times 24) + (16 \times 5) = 584\,\text{in}^2$$

$$p_{cp} = 2 \times (21 + 24 + 16) = 122\ \text{in.}$$

$$A_{cp}^2 / p_{cp} = 2796 \ \text{in.}^3$$

The torsional properties of the beam ignoring the overhanging flange are the following:

$$A_{cp} = (21 \times 24) = 504 \, \text{in.}^2$$

$$p_{cp} = 2 \times (21 + 24) = 90 \, \text{in.}$$

$$A_{cp}^2 / p_{cp} = 2822 \ \text{in}^3 > 2796 \ \text{in}^3$$

Therefore, ignore flange per ACI 318, 11.5.1.1.

$$T_u = 0.75 \times 4\sqrt{5{,}000} \times 2822 = 598637 \ \text{in.-lbs} = 49.9 \ \text{ft-kips}$$

It is assumed that the torsional loading on the beam is uniformly distributed along the span.

Determine the Adequacy of Cross-sectional Dimensions for the Torsion

For solid sections, the limit on shear and torsion is given by:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7 A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f_c^{'}}\right)$$

Using $d = 17.88$ in., the factored shear force at the critical section located at a distance d from the face of the support is:

$$V_u = 66.1 - 4.08 \times \frac{17.88}{12} = 60 \ \text{kips.}$$

Also, the nominal shear strength provided by the concrete is:

$$V_c = 2\lambda\sqrt{f_c^{'}} \ b_w d$$

Assuming a 1.5-in. clear cover to No. 4 closed stirrups at bottom and 2.125 in clear cover to No. 4 closed stirrups at top.

$$A_{oh} = [21 - ((2.125 + 1.5) + 0.5)] \times [24 - ((2.125 + 1.5) + 0.5)] = 335.4 \, \text{in}^2$$

$$p_h = 2 \times \{[21 - (2.125 + 1.5) + 0.5] + [24 - (2.125 + 1.5) + 0.5]\} = 73.5 \, \text{in}$$

$$\sqrt{\left(\frac{60000}{24 \times 17.88}\right)^2 + \left(\frac{49.9 \times 12000 \times 73.5}{1.7 \times 335.4^2}\right)^2} = 269.3 \ \text{psi}$$

$$< 0.75 \times [(\frac{2 \times \sqrt{5000} \times 24 \times 17.88}{24 \times 17.88}) + 8 \times \sqrt{5{,}000}] = 530.3 \ \text{psi}$$

Therefore, the section is adequate.

Determine the Transverse Reinforcement Required for Torsion

$$\frac{A_t}{s} = \frac{T_u}{\phi 2 A_o f_{yt} \cot\theta}$$

where

$$A_o = 0.85 \times A_{oh} = 0.85 \times 335.4 = 285.1 \, \text{in}^2$$

$$\theta = 45°$$

Therefore,

$$\frac{A_t}{s} = \frac{49.9 \times 12000}{0.75 \times 2 \times 285.1 \times 60000 \times \cot 45°} = 0.0233 \, \text{in}^2 / \text{in per leg}$$

Determine the Transverse Reinforcement Required for Shear

From Table 2.4.2.2 above, the shear value in end span at face of first interior support governs.

Shear in end members at face of first interior support

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15 \times 4.08 \times 28.17}{2} = 66.1 \text{ kips}$$

The design shear at a distance, d, away from the face of support,

$$V_u = 66.1 - 4.08 \times \frac{17.88}{12} = 60 \text{ kips}$$

Nominal shear strength provided by concrete:

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2 \times 1.0 \times \sqrt{5000} \times 24 \times 17.88 = 60687 \text{ lb } = 60.7 \text{ kips}$$

Design shear strength provided by concrete:

$$\phi V_c = \phi(2\lambda\sqrt{f'_c}\, b_w d) = 0.75 \times \left(2 \times 1.0 \times \sqrt{5000} \times 24 \times 17.88\right) = 45515 \text{ lb } = 45.5 \text{ kips}$$

Since the factored shear force, V_u, exceeds one-half of the design shear strength provided by concrete, $\frac{\phi V_c}{2}$, shear reinforcement is required per ACI 318, 11.4.6.1.

Since the factored shear force, V_u, exceeds the design shear strength provided by concrete, ϕV_c, minimum shear reinforcement per ACI 318, 11.4.6.3 shall not be sufficient and the shear reinforcement shall be provided to satisfy ACI 318, Eq. 11-1 and 11-2.

$$\phi V_n \geq V_u$$

$$V_n = V_c + V_s$$

Therefore, the nominal shear strength required to be provided by steel:

$$V_s = V_n - V_c = \frac{V_u}{\phi} - V_c = (\frac{60}{0.75}) - 60.7 = 19.3 \text{ kips}$$

If V_s is greater than $8\sqrt{f'_c}\, b_w d$, then the cross-section has to be increased as ACI 318-11.4.7.9 limits the shear capacity to be provided by stirrups to $8\sqrt{f'_c}\, b_w d$.

$$8\sqrt{f'_c}\, b_w d = 8 \times \sqrt{5000} \times 24 \times 17.88 = 242747 \text{ lb } = 242.7 \text{ kips}$$

Since V_s does not exceed $8\sqrt{f'_c}\, b_w d$. The cross-section is adequate.

The required transverse reinforcement for shear is:

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_{yt} d} = \frac{60 - 45.5}{0.75 \times 60 \times 17.88} = 0.018 \text{ in}^2 / \text{in}$$

Calculate total required transverse reinforcement for shear and torsion

$$\frac{A_v}{s} + 2\frac{A_t}{s} = 0.018 + 2 \times 0.0233 \frac{\text{in.}^2/\text{in.}}{\text{leg}} = 0.0647 \text{ in.}^2/\text{in.}$$

Minimum transverse reinforcement for shear and torsion is calculated per ACI 318, 11.5.5.2

$$\left(\frac{A_v}{s} + 2\frac{A_t}{s}\right) = 0.75\sqrt{f'_c}\frac{b_w}{f_{yt}} = 0.75 \times \sqrt{5000} \times \frac{24}{60000} = 0.0212 \text{ in.}^2/\text{in.}$$

but shall not be less than $50 b_w / f_{yt} = 50 \times 24 / 60000 =$ 0.020 in.^{2}/in.

Provide $\frac{A_v}{s} + 2\frac{A_t}{s} = 0.0647$ in.^{2}/in. as closed stirrups.

Maximum spacing of transverse torsion reinforcement per ACI 318, 11.5.6.1 is:

Minimum of $(p_h / 8;12) = \text{Min}(73.5/8,12) = \text{Min}(9.19;12) = 9.19$ in

Spacing limits for shear reinforcement per ACI 318, 11.4.5.1 is:

Minimum of $(d/2;24) = \text{Min}(17.88/2;24) = \text{Min}(8.94;24) = 8.94$ in (governs)

Assuming No. 4 closed stirrups with 2 legs (area per leg $= 0.20\,\text{in}^2$), the required spacing, s, at the critical section is

$$s = 0.40/0.0647 = 6.18 \text{ in } < 8.94 \text{ in}$$

Provide No. 4 closed stirrups spaced at 6 in. on center.

In view of the shear and torsion distribution along the span length, this same reinforcement and spacing can be provided throughout the span length.

Determine the Longitudinal Reinforcement Required for Torsion per ACI 318, 11.5.3.7

$$A_\ell = \left(\frac{A_t}{s}\right) p_h \left(\frac{f_{yt}}{f_y}\right) \cot^2\theta = 0.0233 \times 73.5 \times \left(\frac{60}{60}\right) \times \cot^2 45° = 1.71\,\text{in}^2$$

The minimum total area of longitudinal torsional reinforcement per ACI 318, 11.5.5.3

$$A_{\ell,min} = \frac{5\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right) p_h \frac{f_{yt}}{f_y}$$

where

$$A_t/s = 0.0233 \text{ in.}^2\text{/in} > 25b_w/f_{yt} = 25 \times 24/60000 = 0.010\,\text{in}^2\text{/in}$$

$$A_{\ell,min} = \frac{5 \times \sqrt{5000} \times 504}{60000} - \left(0.0233 \times 73.5 \times \frac{60}{60}\right) = 1.26 \text{ in}^2 < A_\ell = 1.71\,\text{in}^2$$

Use $A_\ell = 1.71\,\text{in}^2$

According to ACI 318, 11.5.6.2, the longitudinal reinforcement is to be distributed around the perimeter of the stirrups, with a maximum spacing of 12 in. There shall be at least one longitudinal bar in each corner of the stirrups. And longitudinal bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than 3/8 in. In order to meet maximum spacing requirement a rebar is to be provided between corner bars at all four sides. This configuration leads to eight-bars; three at top, three at bottom, and one at each side. Therefore, the reinforcement are per bar is $A_\ell = \frac{1.71}{8} = 0.22 \text{ in}^2$.

Use No. 5 bars for longitudinal bars which also meets minimum bar diameter requirement of 3/8 in. At the negative moment regions (support-top) and positive moment region (midspan-bottom), the required longitudinal torsional reinforcement shall be provided as addition to the area of required flexural reinforcement. At midspan-top region where flexural reinforcement is not required by analysis, 3-No. 5 bars shall be provided. Class B lap splice is to be provided.

Table 2.4.3.2 – Reinforcing Design Summary (Flexure + Torsion)					
	End Span			Interior Span	
	Exterior Negative	Positive	Interior Negative	Positive	Negative
Required Longitudinal Reinforcement (in^2)	2.64	2.84	4.31	2.44	3.87
Required Torsional Longitudinal Reinforcement (in^2)	0.66	0.66	0.66	0.66	0.66
Required Total Longitudinal Reinforcement (in^2)	3.30	3.50	4.97	3.10	4.53
Reinforcement	5-No. 8	5-No. 8	7–No. 8	4-No. 8	6-No. 8

Check the requirement for distribution of flexural reinforcement to control flexural cracking per ACI 318-11, 10.6.

Maximum spacing allowed,

$$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c \le 12\left(\frac{40000}{f_s}\right)$$

$$c_c = 21.0 - (17.88 + 0.5 \times (8/8)) = 2.62 \text{ in.}$$

$$\text{Use } f_s = \frac{2}{3} f_y = 40 \text{ ksi}$$

$$s = 15 \times \left(\frac{40000}{40000}\right) - 2.5 \times 2.62 = 8.45 \text{ in. (governs)}$$

$$s = 12 \times \left(\frac{40000}{40000}\right) = 12 \text{ in.}$$

Spacing provided for 4-No. 8 bars, (governs the maximum spacing check) is

$$\frac{1}{3} \times \{24 - 2 \times 3.0\} = 6 \text{ in.} < 8.45 \text{ in. O.K.}$$

Check the spacing, s provided, is greater than the minimum center to center spacing, s_{min} where

$$s_{min} = d_b + \max\begin{Bmatrix} 1 \\ d_b \\ 1.33 \times \max.\text{agg.} \end{Bmatrix}$$

where maximum aggregate size is ¾”

$$s_{min} = 1 + 1 = 2 \text{ in}$$

Spacing provided for 7-No. 8 bars, (governs the minimum spacing check) is

$$\frac{1}{6} \times \{24 - 2 \times 3.0\} = 3 \text{ in.} > 2.0 \text{ in. O.K.}$$

Therefore, the reinforcement selections in Table 2.4.3.2 meet the spacing requirements.

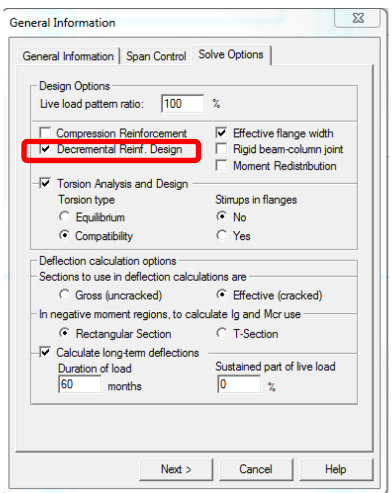
2.4.4. Deflections

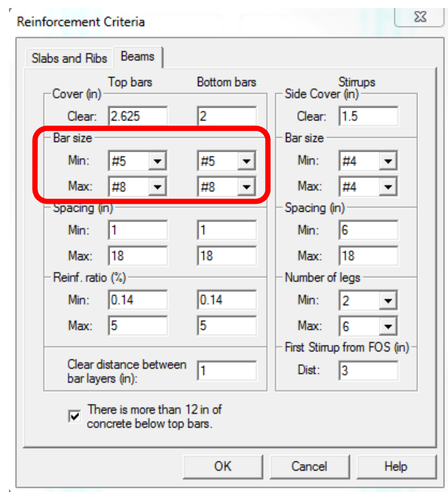
ACI 318 provides the minimum thickness of nonprestressed beams in Table 9.5(a) unless deflections are calculated. Since the preliminary beam depth met this requirement, the deflection calculations are waived here. Typically, in hand solutions, the designer opts to follow ACI 318, 9.5.2 in lieu of deflection calculations which can be long and tedious. It should be noted that, in most other cases, lesser thicknesses and consequent cost savings can be achieved through deflection computations.

2.4.5. Computer Program Solution

spSlab Program is utilized to design the exterior beam. The exterior beam is modeled as a rectangular beam.

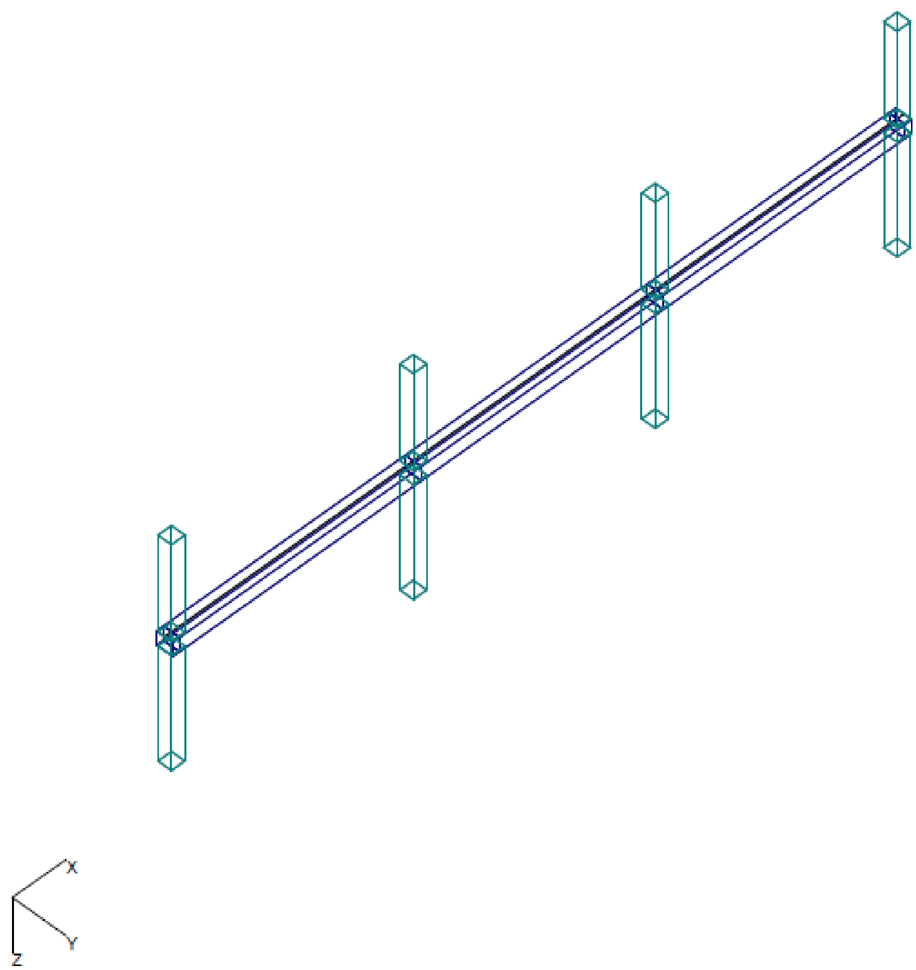
The program calculates the internal forces (shear force and bending moment), moment and shear capacities, immediate and long-term deflection results, and required flexural reinforcement. The graphical and text results are provided below for both input and output of the spSlab model.





Decremental reinforcement design solve option is utilized. This option makes the program start from the Max. Bar Size under Reinforcement Criteria for designing the reinforcement and works its way down to Min. Bar Size. The use of this option results in 5-No. 8 bars top reinforcement at the first interior support @ end span. If this option is unchecked, the program shall provide 9-No. 6 bars.

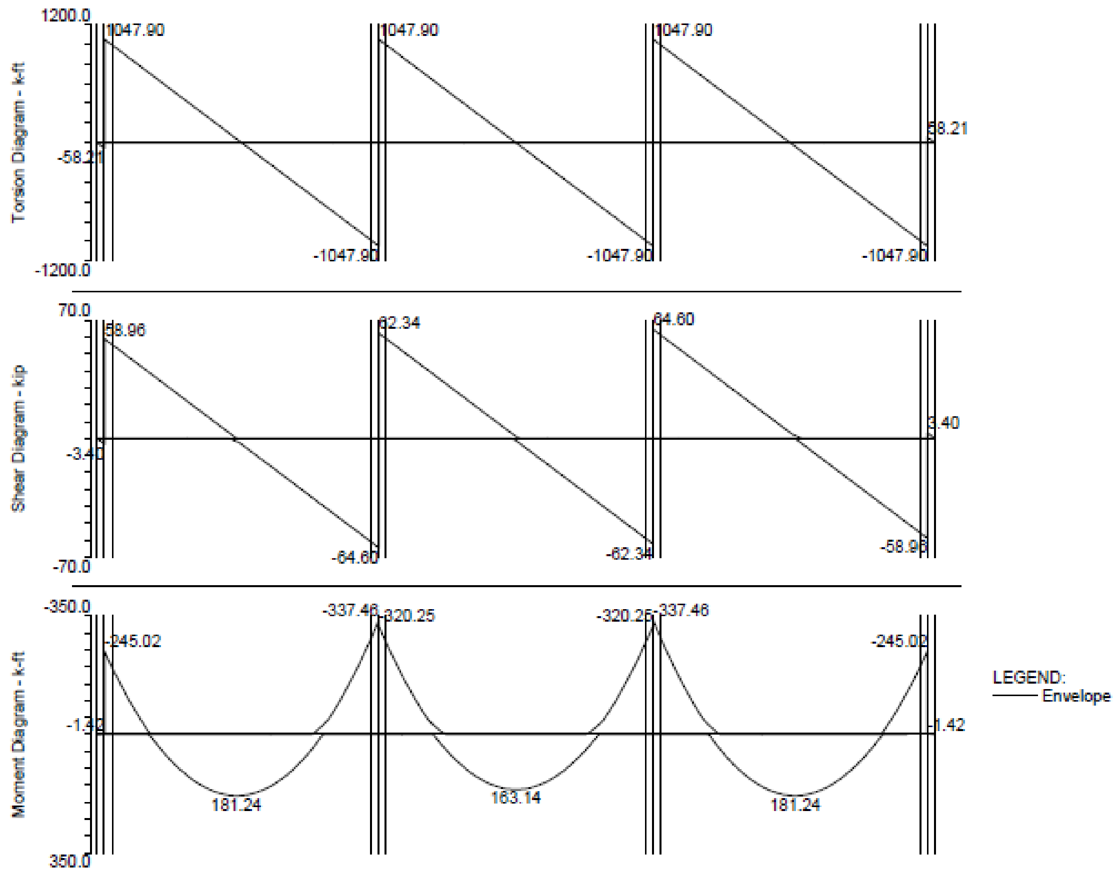
2.4.5.1. Isometric View of Interior Beam from spSlab



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\Exterior Beam Design\Exterior Beam Design.slb
Project: Wide-Module Joist System
Frame: Exterior Beam
Engineer: MD
Code: ACI 318-11
Date: 12/10/13
Time: 09:28:45

2.4.5.2. spSlab Model Calculated Internal Forces (Shear Force & Bending Moment)



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\Exterior Beam Design\Exterior Beam Design.slb

Project: Wide-Module Joist System

Frame: Exterior Beam

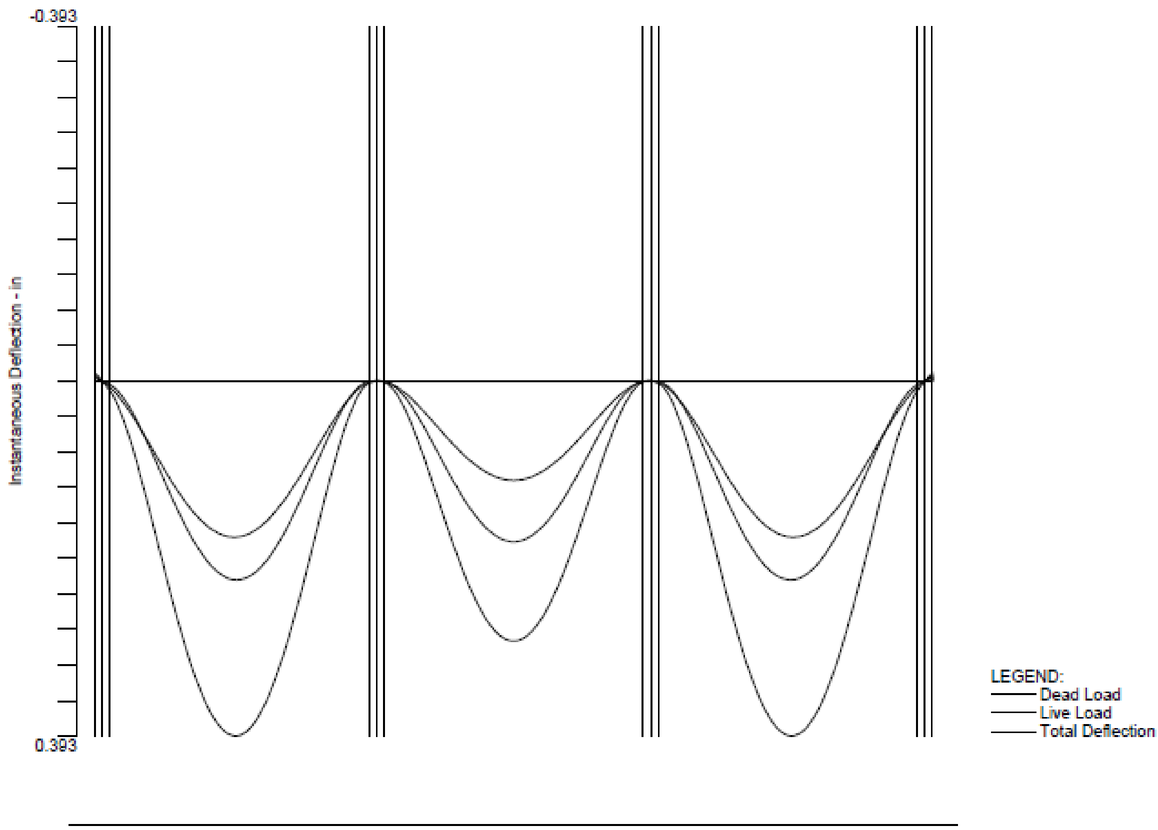
Engineer: MD

Code: ACI 318-11

Date: 12/10/13

Time: 09:32:51

2.4.5.3. spSlab Model Calculated Immediate Deflections



spSlab v3.60. Licensed to: StructurePoint. License ID: 00000-0000000-4-2A05D-2471B

File: C:\Projects\Wide-Module Joist System\Exterior Beam Design\Exterior Beam Design.slb

Project: Wide-Module Joist System

Frame: Exterior Beam

Engineer: MD

Code: ACI 318-11

Date: 12/10/13

Time: 09:42:10

2.4.6. Summary and Comparison of Design Results

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[2] DESIGN RESULTS

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Top Reinforcement

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Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	AsReq	SpProv	Bars
1	Left	2.00	0.00	0.000	0.601	9.116	0.000	6.236	4-#8 *3 *5
	Midspan	2.00	0.00	0.000	0.601	9.116	0.000	6.236	4-#8 *3 *5
	Right	2.00	0.00	0.000	0.601	9.116	0.000	6.236	4-#8 *3 *5
2	Left	2.00	197.30	0.833	1.517	9.116	2.561	6.236	4-#8
	Midspan	2.00	0.00	15.000	0.000	9.116	0.000	0.000	---
	Right	2.00	285.04	29.167	1.517	9.116	3.779	4.677	5-#8
3	Left	2.00	269.72	0.833	1.517	9.116	3.562	4.677	5-#8
	Midspan	2.00	0.00	15.000	0.000	9.116	0.000	0.000	---
	Right	2.00	269.72	29.167	1.517	9.116	3.562	4.677	5-#8
4	Left	2.00	285.04	0.833	1.517	9.116	3.779	4.677	5-#8
	Midspan	2.00	0.00	15.000	0.000	9.116	0.000	0.000	---
	Right	2.00	197.30	29.167	1.517	9.116	2.561	6.236	4-#8
5	Left	2.00	0.00	0.833	0.601	9.116	0.000	6.236	4-#8 *3 *5
	Midspan	2.00	0.00	0.833	0.601	9.116	0.000	6.236	4-#8 *3 *5
	Right	2.00	0.00	0.833	0.601	9.116	0.000	6.236	4-#8 *3 *5

NOTES:

*3 - Design governed by minimum reinforcement.

*5 - Number of bars governed by maximum allowable spacing.

Top Bar Details

================

Units: Length (ft)

	Left				Continuous		Right			
Span	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	---		---		4-#8	0.83	---		---	
2	2-#8	6.85	2-#8	3.72	---		3-#8	9.12	2-#8	4.09
3	3-#8	9.12	2-#8	3.87	---		3-#8	9.12	2-#8	3.87
4	3-#8	9.12	2-#8	4.09	---		2-#8	6.85	2-#8	3.72
5	---		---		4-#8	0.83	---		---	

```
Bottom Reinforcement
====================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span   Width       Mmax      Xmax     AsMin     AsMax     AsReq    SpProv     Bars
   ---- --------- ---------- --------- --------- --------- --------- --------- --------
     1     2.00       0.00     0.000     0.000     9.435     0.000     0.000      ---

     2     2.00     181.24    14.433     1.570     9.435     2.258     9.354     3-#8

     3     2.00     163.14    15.000     1.570     9.435     2.025     9.354     3-#8

     4     2.00     181.24    15.567     1.570     9.435     2.258     9.354     3-#8

     5     2.00       0.00     0.833     0.000     9.435     0.000     0.000      ---

Bottom Bar Details
==================
   Units: Start (ft), Length (ft)
        _______Long Bars_______  ______Short Bars_______
   Span   Bars   Start   Length     Bars   Start   Length
   ---- -------- ------- -------- -------- ------- -------
     1     ---                        ---

     2    2-#8    0.00    30.00     1-#8    0.00    21.58

     3    2-#8    0.00    30.00     1-#8    9.05    11.90

     4    2-#8    0.00    30.00     1-#8    8.42    21.58

     5     ---                        ---

 Required transverse reinforcement
 ---------------------------------
 Units: Start, End, Xu (ft), Vu (kip), Tu (k-ft), vf (ksi)
        Av/s, At/s, A(v+2t)/s (in^2/in)
 Span    Start      End       Vu       Tu       vf        Xu Comb/Patt     Av/s      At/s A(v+2t)/s
 ---- --------- --------- --------- --------- --------- --------- --------- --------- --------- ---------
   1     0.000     0.000      0.00      0.00     0.000      0.00    U1/All    0.0000    0.0000    0.0000 *2

   2     1.083     5.945     49.49     49.89     0.248      2.32   U2/Even    0.0050    0.0226    0.0502 *4
         5.945     9.567     24.29     49.89     0.227      5.94    U1/All    0.0000    0.0226    0.0452 *4
         9.567    13.189     13.61     49.89     0.222      9.57    U1/All    0.0000    0.0226    0.0452 *4
        13.189    16.811      7.73     49.89     0.221     16.81    U1/All    0.0000    0.0226    0.0452 *4
        16.811    20.433     18.41     49.89     0.224     20.43    U1/All    0.0000    0.0226    0.0452 *4
        20.433    24.055     29.08     49.89     0.230     24.06    U1/All    0.0000    0.0226    0.0452 *4
        24.055    28.917     55.13     49.89     0.255     27.68     U2/S2    0.0120    0.0226    0.0572 *4

   3     1.083     5.945     52.86     49.89     0.252      2.32     U2/S2    0.0092    0.0226    0.0544 *4
         5.945     9.567     26.69     49.89     0.229      5.94    U1/All    0.0000    0.0226    0.0452 *4
         9.567    13.189     16.01     49.89     0.223      9.57    U1/All    0.0000    0.0226    0.0452 *4
        13.189    16.811      5.34     49.89     0.220     16.81    U1/All    0.0000    0.0226    0.0452 *4
        16.811    20.433     16.01     49.89     0.223     20.43    U1/All    0.0000    0.0226    0.0452 *4
        20.433    24.055     26.69     49.89     0.229     24.06    U1/All    0.0000    0.0226    0.0452 *4
        24.055    28.917     52.86     49.89     0.252     27.68     U2/S3    0.0092    0.0226    0.0544 *4

   4     1.083     5.945     55.13     49.89     0.255      2.32     U2/S3    0.0120    0.0226    0.0572 *4
         5.945     9.567     29.08     49.89     0.230      5.94    U1/All    0.0000    0.0226    0.0452 *4
         9.567    13.189     18.41     49.89     0.224      9.57    U1/All    0.0000    0.0226    0.0452 *4
        13.189    16.811      7.73     49.89     0.221     13.19    U1/All    0.0000    0.0226    0.0452 *4
        16.811    20.433     13.61     49.89     0.222     20.43    U1/All    0.0000    0.0226    0.0452 *4
        20.433    24.055     24.29     49.89     0.227     24.06    U1/All    0.0000    0.0226    0.0452 *4
        24.055    28.917     49.49     49.89     0.248     27.68   U2/Even    0.0050    0.0226    0.0502 *4

   5     0.833     0.833      0.00      0.00     0.000      0.83    U1/All    0.0000    0.0000    0.0000 *2
 NOTES:
 *2 - Torsion ignored (Tu < PhiTcr/4).
 *4 - Design torsional moment reduced to PhiTcr due to compatibility torsion.
```

```
Required longitudinal reinforcement
-----------------------------------
Units: Start, End, Xu (ft), Tu (k-ft), Al (in^2)
Span    Start       End        Tu        Xu Comb/Patt       Al
---- --------- --------- --------- --------- --------- ---------
   1     0.000     0.000      0.00      0.00    U1/All     0.000 *2

   2     1.083     5.945     49.89      2.32    U1/All     1.691 *4
         5.945     9.567     49.89      5.94    U1/All     1.691 *4
         9.567    13.189     49.89      9.57    U1/All     1.691 *4
        13.189    16.811     19.79     14.72    U1/All     2.222 *5
        16.811    20.433     49.89     16.81    U1/All     1.691 *4
        20.433    24.055     49.89     20.43    U1/All     1.691 *4
        24.055    28.917     49.89     24.06    U1/All     1.691 *4

   3     1.083     5.945     49.89      2.32    U1/All     1.691 *4
         5.945     9.567     49.89      5.94    U1/All     1.691 *4
         9.567    13.189     49.89      9.57    U1/All     1.691 *4
        13.189    16.811     19.79     14.72    U1/All     2.222 *5
        16.811    20.433     49.89     16.81    U1/All     1.691 *4
        20.433    24.055     49.89     20.43    U1/All     1.691 *4
        24.055    28.917     49.89     24.06    U1/All     1.691 *4

   4     1.083     5.945     49.89      2.32    U1/All     1.691 *4
         5.945     9.567     49.89      5.94    U1/All     1.691 *4
         9.567    13.189     49.89      9.57    U1/All     1.691 *4
        13.189    16.811     19.79     14.72    U1/All     2.222 *5
        16.811    20.433     49.89     16.81    U1/All     1.691 *4
        20.433    24.055     49.89     20.43    U1/All     1.691 *4
        24.055    28.917     49.89     24.06    U1/All     1.691 *4

   5     0.833     0.833      0.00      0.83    U1/All     0.000 *2
NOTES:
*2 - Torsion ignored (Tu < PhiTcr/4).
*4 - Design torsional moment reduced to PhiTcr due to compatibility torsion.
*5 - Minimum longitudinal reinforcement required.
```

Table 2.4.6.1 – Comparison of Hand Solution with spSlab Solution

Span Location	Required Reinforcement Area for Flexure + Torsion (in^2)		Reinforcement Provided for Flexure + Torsion	
End Span	**Hand Solution**	**spSlab Solution**	**Hand Solution**	**spSlab Solution**
Interior Negative	4.97	3.779+1.691*(3/8)=4.413	7 – No. 8	5 – No. 8 + 1 – No. 8
Positive	3.50	2.258+1.691*(3/8)=2.892	5 – No. 8	3 – No. 8 + 1 – No. 8

2.4.7. Conclusions and Observations

In this design example, the exterior beam is modeled as a continuous rectangular beam representing. There is a good agreement between the hand solution and computer solution.

2.5. Design of Interior Column by spColumn

2.5.1 Determine Service Loads

Total service loads on 1st story interior column (@ 1st interior support) are reorganized based on the calculations on section 1.2:

Total service dead load $= 1.15 \times \left[\left(4 \times \left(82.83 + 20\right) + 1 \times 82.83\right) \times 960\right] / 1000 + 46.1 = 591.6$ kips

Total service live load $= 1.15 \times \left[\left(4 \times \left(39.6\right) + 1 \times 12\right) \times 960\right] / 1000 = 188.1$ kips

2.5.2 Moment Interaction Diagram

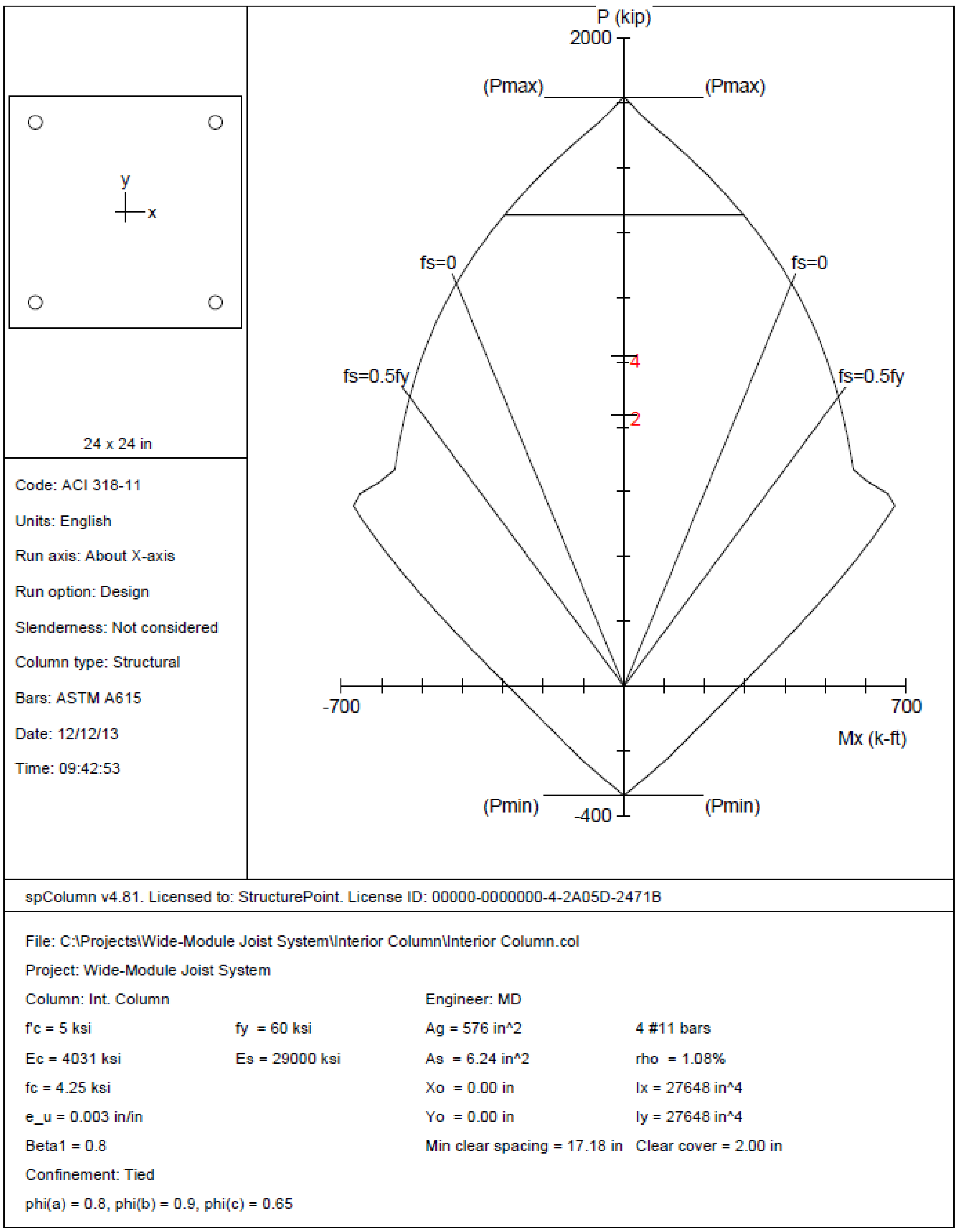

Figure 2.5 – P-M Diagram for Interior Column from spColumn Software Program