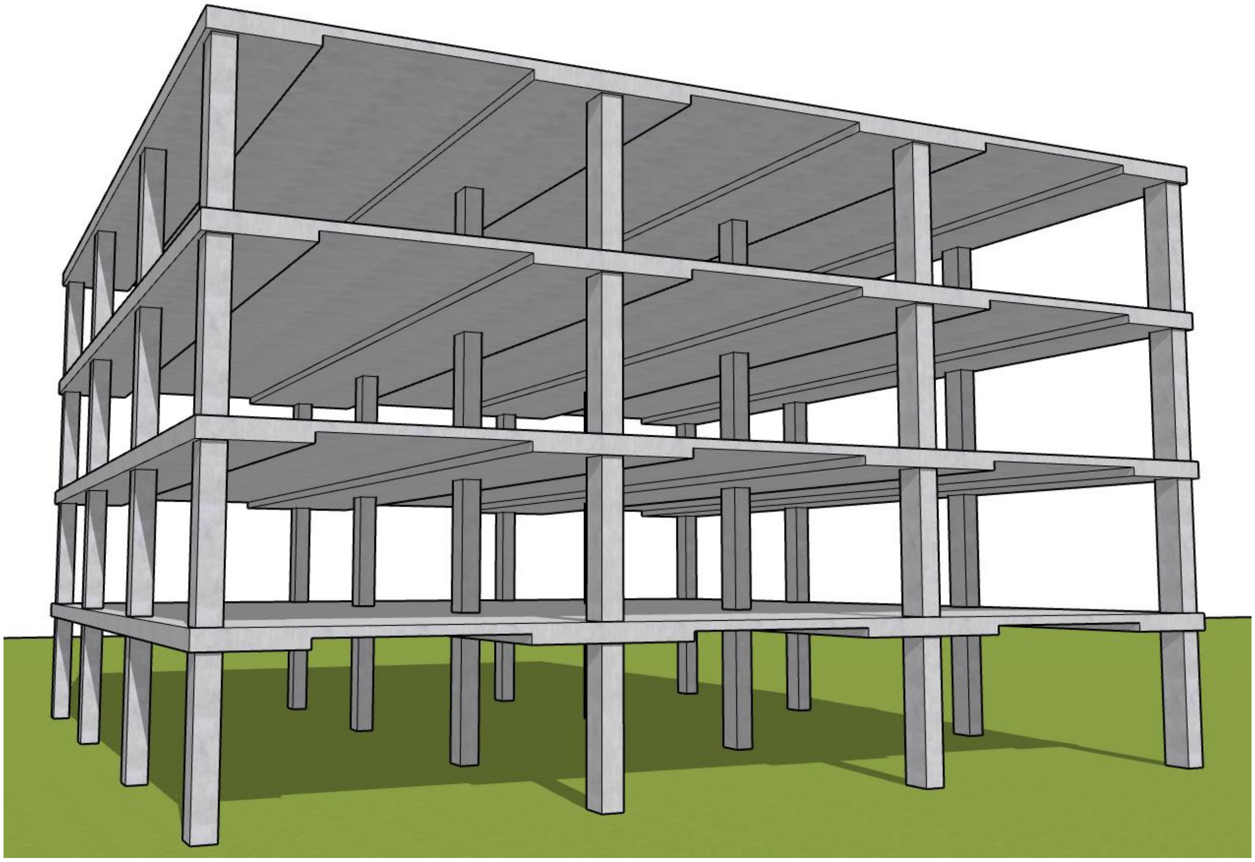
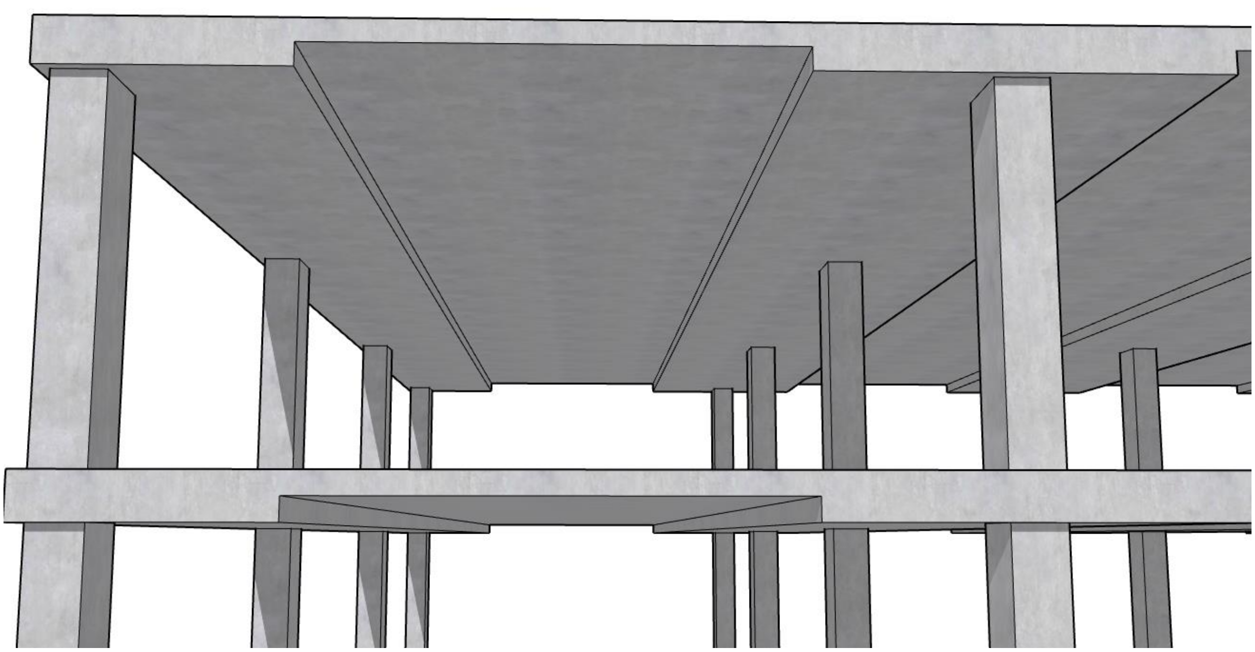
Two-Way Concrete Floor with Slab Bands – Longitudinal Bands Analysis & Design (CAC Design Handbook)





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Slab bands are thickened portions extended along columns centerlines in one direction of the slab to increase the nominal strength of the concrete floor at the critical section around the columns. This system is considered more economical compared to slabs with drop panels due to the savings in the formwork and labor cost. Slab bands are sometimes viewed as continuous extension of drop panels between supports or a support and another slab band. In U.S. standards like ACI-318, slab bands are modeled as a system of wide and shallow beams in one direction.

The concrete floor system with slab bands shown below is for an intermediate floor to be designed considering loads described in design data below. The lateral loads are independently resisted by shear walls. The use of flat plate system will be checked first. If the use of flat plate is not adequate, the use of a slab system with slab bands will be investigated. The analysis procedure "Elastic Frame Method (EFM)" prescribed in ***CSA A23.3-14*** is illustrated in detail in this example (Example #3 from the CAC Design Handbook). The EFM hand solution is also used for a comprehensive comparison with results from the Reference using the Direct Design Method (DDM). The EFM hand solution results are further compared with the output from the engineering software program spSlab. Explanation of the EFM is available in StructurePoint Video Tutorials page. A table comparing the three two-way slab analysis methods is provided at the end of this document.

This example will examine floor design strips with slab bands parallel to the direction of analysis (Longitudinal Bands). Floor design strips with slab bands perpendicular to the direction of analysis (Transverse Bands) are covered in detail in (Two-Way Concrete Floor with Slab Bands – Transverse Bands Analysis & Design (CAC Design Handbook)) design example.

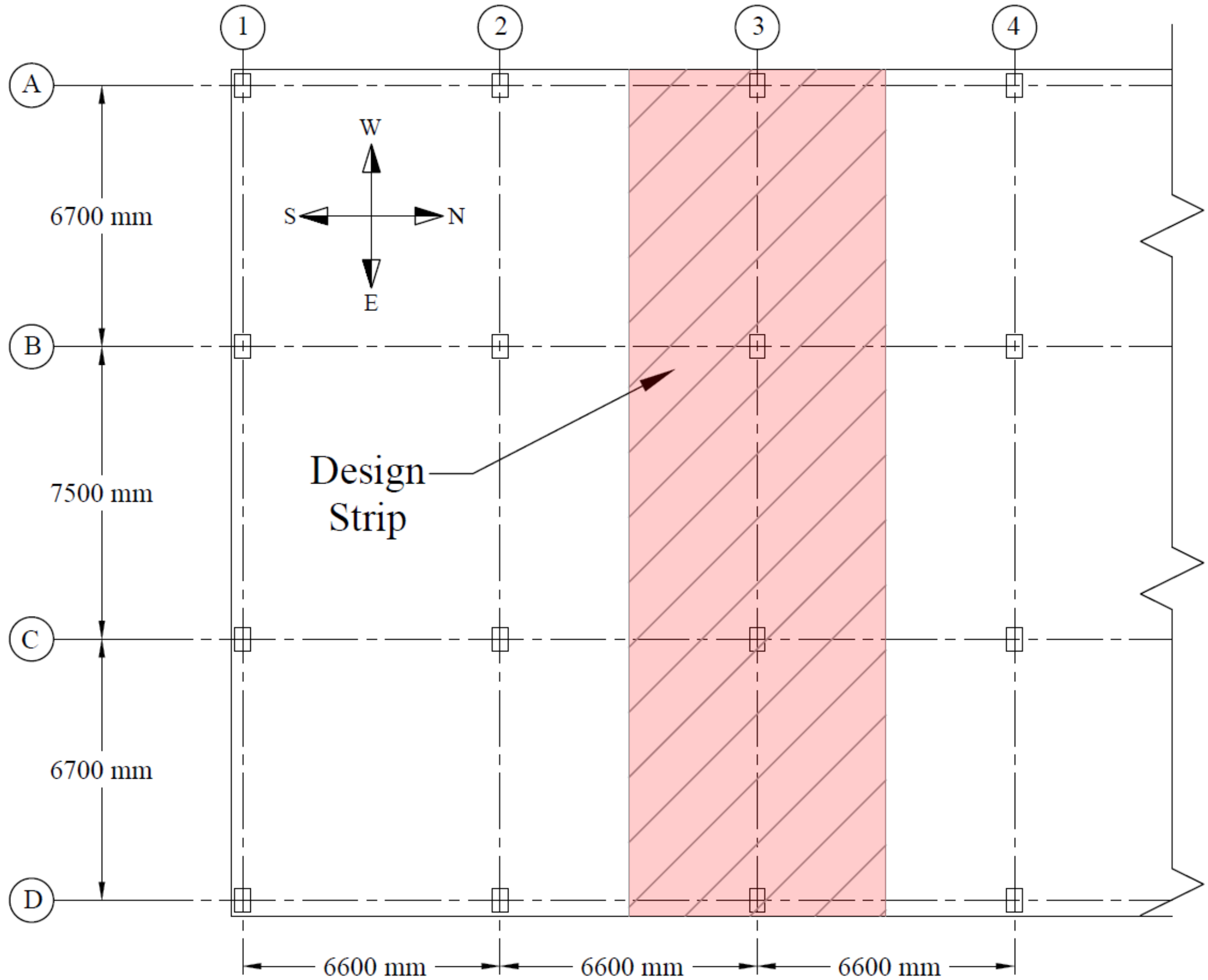


Figure 1 - Two-Way Flat Plate Concrete Floor System

Contents

Code

Design of Concrete Structures (CSA A23.3-14)

Reference

CAC Concrete Design Handbook, 4th Edition, Cement Association of Canada

Notes on ACI 318-11 Building Code Requirements for Structural Concrete, Twelfth Edition, 2013 Portland Cement Association

Design Data

Floor-to-Floor Height = 3 m (provided by architectural drawings)

Superimposed Dead Load, SDL $= 1 \text{ kN/m}^2$ for framed partitions, wood studs plaster 2 sides

$= 1 \text{ kN/m}^2$ for mechanical services

Live Load, $LL = 3.6 \text{ kN/m}^2$ for Residential floors

$f'_c = 25$ MPa (for slabs and columns)

$f'_y = 400$ MPa

Column Dimensions = 400 mm x 600 mm

Solution

1. Preliminary Member Sizing

1.1. Preliminary Member Sizing For Slabs Without Slab Bands

1.1.1. Slab minimum thickness - Deflection

CSA A23.3-14 (13.2)

Minimum member thickness and depths from CSA A23.3-14 will be used for preliminary sizing.

Using CSA A23.3-14 minimum slab thickness for two-way construction without interior beams in ***Section 13.2.3.***

Exterior Panels (N-S Direction Governs):

$$h_{s,\min} = 1.1 \times \frac{l_n\left(0.6 + f_y / 1000\right)}{30} = 1.1 \times \frac{6200\left(0.6 + 400/1000\right)}{30} = 227 \text{ mm}$$ ***CSA A23.3-14 (13.2.3)***

But not less than 120 mm. ***CSA A23.3-14 (13.2.1)***

Where l_n = length of clear span in the long direction = 6600 – 400 = 6200 mm

Interior Panels (E-W Direction Governs):

$$h_{s,\min} = \frac{l_n\left(0.6 + f_y / 1000\right)}{30} = \frac{6900\left(0.6 + 400/1000\right)}{30} = 230 \text{ mm}$$ ***CSA A23.3-14 (13.2.3)***

But not less than 120 mm. ***CSA A23.3-14 (13.2.1)***

Where l_n = length of clear span in the long direction = 7500 – 600 = 6900 mm

Try 250 mm slab for all panels (self-weight = 5.89 kN/m^2)

1.1.2. Slab one way shear strength

Evaluate the average effective depth (Figure 2):

$$d_t = t_{slab} - c_{clear} - d_b - \frac{d_b}{2} = 250 - 25 - 16 - \frac{16}{2} = 201 \text{ mm}$$

$$d_l = t_{slab} - c_{clear} - \frac{d_b}{2} = 250 - 25 - \frac{16}{2} = 217 \text{ mm}$$

$$d_{avg} = \frac{d_l + d_t}{2} = \frac{201 + 217}{2} = 209 \text{ mm}$$

Where:

c_{clear} = 20 mm for 15M steel bar ***CSA A23.3-14 (Annex A. Table 17)***

Note that the reference used 25 mm as clear cover, in this example the clear cover used is 25 mm to be consistent with reference.

d_b = 16 mm for 15M steel bar

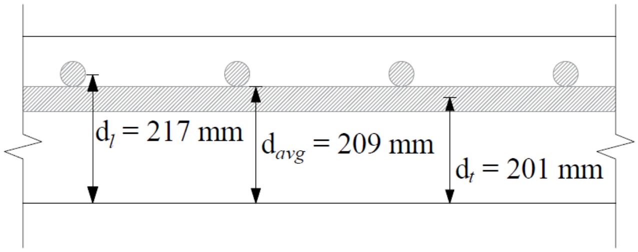


Figure 2 - Two-Way Flat Concrete Floor System

Load Combination 1:

Factored dead load, $w_{df} = 1.4 \times (5.89 + 1 + 1) = 11.05 \text{ kN/m}^2$ ***CSA A23.3-14 (Annex C. Table C.1 a)***

Total factored load $w_f = 11.05 \text{ kN/m}^2$

Load Combination 2:

Factored dead load, $w_{df} = 1.25 \times (5.89 + 1 + 1) = 9.86 \text{ kN/m}^2$

Factored live load, $w_{lf} = 1.5 \times 3.6 = 5.40 \text{ kN/m}^2$ ***CSA A23.3-14 (Annex C. Table C.1 a)***

Total factored load $w_f = w_{df} + w_{lf} = 15.26 \text{ kN/m}^2$ (Controls)

Check the adequacy of slab thickness for beam action (one-way shear) ***CSA A23.3-14 (13.3.6)***

At an interior column:

The critical section for one-way shear is extending in a plane across the entire width and located at a distance, d_v from the face of support or concentrated load (see Figure 3). ***CSA A23.3-14 (13.3.6.1)***

Consider a 1 m. wide strip.

Tributary area for one-way shear is $A_{Tributary} = \left(\frac{\left[\left(\frac{7500}{2} \right) - \left(\frac{600}{2} \right) - 188 \right] \times (1000)}{1000^2} \right) = 3.26 \text{ m}^2$

$V_f = w_f \times A_{Tributary} = 15.26 \times 3.26 = 49.75 \text{ kN}$

$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$ ***CSA A23.3-14 (Eq. 11.6)***

Where:

$\lambda = 1$ for normal weight concrete ***CSA A23.3-14 (8.6.5)***

$\beta = 0.21$ for slabs with overall thickness not greater than 350 mm ***CSA A23.3-14 (11.3.6.2)***

$d_v = \text{Max}\ (0.9d_{avg}, 0.72h) = \text{Max}\ (0.9 \times 209, 0.72 \times 250) = \text{Max}\ (188,180) = 188\ \text{mm}$ **_CSA A23.3-14 (3.2)_**

$\sqrt{f_c^{'}} = 5\ \text{MPa} < 8\ \text{MPa}$ **_CSA A23.3-14 (11.3.4)_**

$$V_c = 0.65 \times 1 \times 0.21 \times \sqrt{25} \times 1000 \times \frac{188}{1000} = 128.3\ \text{kN} > V_f$$

Slab thickness of 250 mm is adequate for one-way shear.

1.1.3. Slab two-way shear strength

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

Shear perimeter: $b_0 = 2 \times (600 + 400 + 2 \times 209) = 2836\ \text{mm}$ **_CSA A23.3-14 (13.3.3)_**

Tributary area for two-way shear is

$$A_{Tributary} = \left(\frac{7.5 + 6.7}{2} \times 6.6\right) - \left(\frac{600 + 209}{1,000} \times \frac{400 + 209}{1,000}\right) = 46.86 - 0.49 = 46.37\ \text{m}^2$$

The factored resisting shear stress, V_r shall be the smallest of: **_CSA A23.3-14 (13.3.4.1)_**

a) $v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c}$ **_CSA A23.3-14 (Eq. 13.5)_**

$$v_r = \left(1 + \frac{2}{1.5}\right) \times 0.19 \times 0.65 \times \sqrt{25} = 1.44\ \text{MPa}$$

Where $\beta_c = \frac{600}{400} = 1.5$ (ratio of long side to short side of the column) **_CSA A23.3-14 (13.3.4.1)_**

b) $v_r = v_c = \left(\frac{\alpha_s d}{b_o} + 0.19\right) \lambda \phi_c \sqrt{f'_c}$ **_CSA A23.3-14 (Eq. 13.6)_**

$$v_r = \left(\frac{4 \times 209}{2836} + 0.19\right) \times 1 \times 0.65 \times \sqrt{25} = 1.58\ \text{MPa}$$

c) $v_r = v_c = 0.38 \lambda \phi_c \sqrt{f'_c} = 0.38 \times 1 \times 0.65 \times \sqrt{25} = 1.24\ \text{MPa}$ (Governs) **_CSA A23.3-14 (Eq. 13.7)_**

Determine the shear stress due to the factored direct shear:

$$\left(v_f\right)_{avr} = \frac{V_f}{b_o d} = \frac{15.26 \times \left(\frac{7.5 + 6.7}{2} \times 6.6\right)}{2836 \times 209} \times 1,000 = 1.206\ \text{MPa}$$

For an interior column, multiply this value with 1.20 in order to account for the effect of unbalanced moment.

$1.20 \times \left(v_f\right)_{avr} = 1.20 \times 1.206 = 1.45\ \text{MPa} > v_r = 1.24\ \text{MPa}$ (No Good)

CAC Concrete Design Handbook 4th Edition (5.2.3)

Slab thickness of 250 mm is **NOT** adequate for two-way shear.

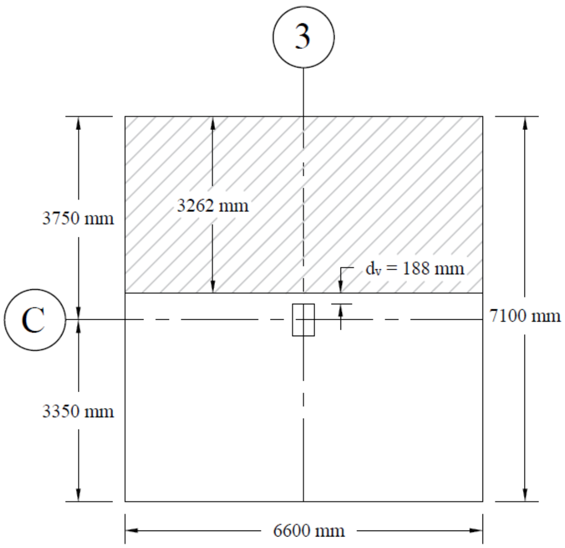


Figure 3 – Critical Section for One-Way Shear

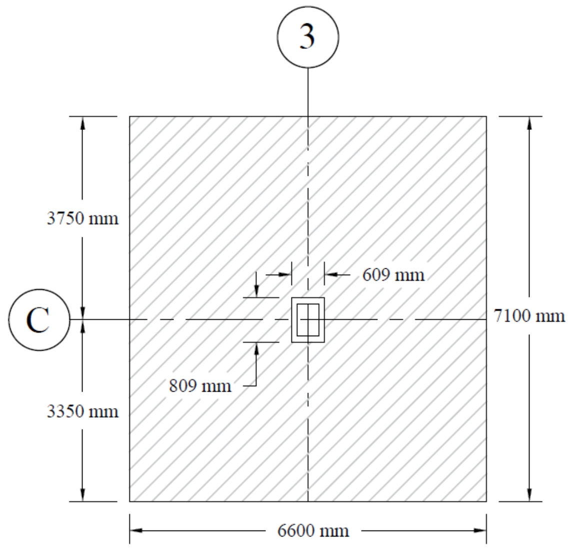


Figure 4 – Critical Section for Two-Way Shear

In this case, four options could be used: 1) to increase the slab thickness, 2) to increase column's cross sectional dimensions or cut the spacing between columns (reducing span lengths), however, this option is assumed to be not permissible in this example due to architectural limitations, 3) to use headed shear reinforcement, or 4) to use drop panels or slab bands. In this example, slab bands will be used to achieve an economical design.

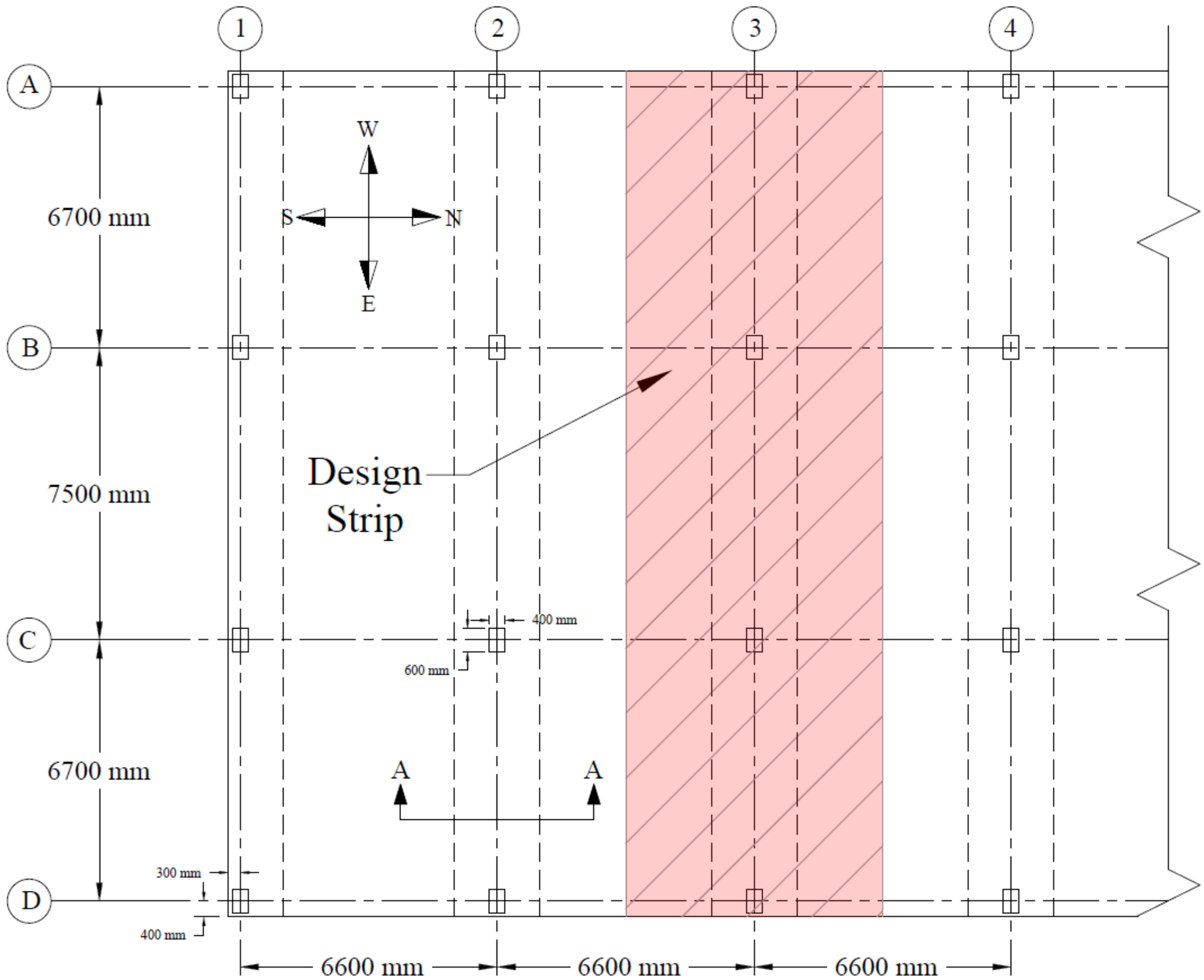


Figure 5 – Two-Way Slab with Slab Bands

1.2. Preliminary Member Sizing For Slab With Slab Bands

For slabs with changes in thickness and subjected to bending in two directions, it is necessary to check shear at multiple sections as defined in the ***CSA A23.3-14***. The critical sections for two-way action shall be located with respect to:

1) Perimeter of the concentrated load or reaction area. ***CSA A.23.3-14 (13.3.3.1)***

2) Changes in slab thickness, such as edges of slab bands. ***CSA A.23.3-14 (13.3.3.2)***

1.2.1. Slab band minimum thickness (E-W direction) – Deflection

Minimum member thickness and depths from CSA A23.3-14 will be used for preliminary sizing.

Determine the slab band thickness by using CSA A23.3-14 minimum slab thickness for slab bands per ***Clause 13.2.6.***

End span (Governs):

$$h_{band,\min} = \frac{l_n}{18} = \frac{6100}{18} = 339 \text{ mm}$$ **CSA A23.3-14 (13.2.3)**

But not less than 120 mm. **CSA A23.3-14 (13.2.1)**

Where l_n = length of clear span in the long direction = 6700 – 600 = 6100 mm

Interior span:

$$h_{band,\min} = \frac{l_n}{21} = \frac{6900}{21} = 329 \text{ mm}$$ **CSA A23.3-14 (13.2.3)**

But not less than 120 mm. **CSA A23.3-14 (13.2.1)**

Where l_n = length of clear span in the long direction = 7500 – 600 = 6900 mm

Try h_{band} = 350 mm slab bands for all panels

1.2.2. Slab minimum thickness (E-W direction) – Deflection

Determine the slab thickness by using CSA A23.3-14 minimum slab thickness for slabs with drop panels.
CSA A23.3-14 (13.2.4)

By definition a slab band is an extended drop panel. However, as a drop panel, the slab band is very deep. The difference between the band thickness and the slab thickness, Δ_h, is likely to exceed the slab thickness. Since, for the purposes of **Equation 13.2** in **CSA A23.3-14**, Δ_h cannot be taken larger than the slab thickness, a preliminary estimate of slab thickness is based on **Equation 13.2** with Δ_h equal to h_s. In the spanning direction of the slab band the term x_d/l_n would take its maximum value of 0.25.

Interior Panel (E-W Direction):

$$h_{s,\min} = \frac{(0.6 + f_y/1{,}000)}{1 + \dfrac{2x_d}{l_n}} \times \left(\frac{l_n}{30}\right)$$ **CSA A23.3-14 (13.2.4)**

$$h_{s,\min} = \frac{(0.6 + 0.4)}{1 + 2 \times 0.25} \times \left(\frac{7500 - 600}{30}\right) = 153.3 \text{ mm}$$

But not less than 120 mm. **CSA A23.3-14 (13.2.1)**

The N-S direction shall be checked in order to determine slab thickness.

Try h_s = 160 mm slab for all panels.

Self-weight for slab section without slab bands = 24 kN/m^3 × 0.160 m = 3.84 kN/m^2

Self-weight for slab section with slab bands = 24 kN/m^3 × 0.350 m = 8.40 kN/m^2

1.2.3. Slab Band Width

The slab band width is assumed to extend in each direction from the centerline of support one-sixth the span length measured from center-to-center of supports in that direction.

$$l_{sb} = \frac{6.6}{6} + \frac{6.6}{6} = 2.2 \text{ m}$$

1.2.4. Slab shear strength – one way shear

<u>For critical section at distance d_v from the edge of the column (slab section with slab band):</u>

Evaluate the average effective depth:

$$d_t = h_{band} - c_{clear} - d_b - \frac{d_b}{2} = 350 - 25 - 16 - \frac{16}{2} = 301 \text{ mm}$$

$$d_l = h_{band} - c_{clear} - \frac{d_b}{2} = 350 - 25 - \frac{16}{2} = 317 \text{ mm}$$

$$d_{avg} = \frac{d_t + d_l}{2} = \frac{301 + 317}{2} = 309 \text{ mm}$$

Where:

c_{clear} = 20 mm **<u>*CSA A23.3-14 (Annex A. Table 17)*</u>**

Note that the reference used 25 mm as clear cover, in this example the clear cover used is 25 mm to be consistent with reference.

d_b = 16 mm for 15M steel bar

Factored dead load $\rightarrow$ $w_{df} = 1.25 \times \left(\left[8.40 \times \frac{2.2}{6.6} + 3.84 \times \frac{6.6 - 2.2}{6.6} \right] + 1 + 1 \right) = 9.20 \text{ kN/m}^2$

Factored live load $\rightarrow$ $w_{lf} = 1.5 \times 3.6 = 5.40 \text{ kN/m}^2$ **<u>*CSA A23.3-14 (Annex C. Table C.1 a)*</u>**

Total factored load $\rightarrow$ $w_f = 9.20 + 5.40 = 14.60 \text{ kN/m}^2$

Check the adequacy of slab thickness for beam action (one-way shear) from the edge of the interior column **<u>*CSA A23.3-14 (13.3.6)*</u>**

Consider a 1 m wide strip. The critical section for one-way shear is located at a distance d_v, from the edge of the column (see Figure 6)

Tributary area for one-way shear is $A_{Tributary} = \left(\frac{\left[\left(\frac{7500}{2} \right) - \left(\frac{600}{2} \right) - 278 \right] \times (1000)}{1000^2} \right) = 3.17 \text{ m}^2$

Where:

$$d_v = Max \begin{Bmatrix} 0.9d \\ 0.72h \end{Bmatrix} = Max \begin{Bmatrix} 0.9(309) \\ 0.72(350) \end{Bmatrix} = Max \begin{Bmatrix} 278 \\ 252 \end{Bmatrix} = 278 \text{ mm}$$

<u>*CSA A23.3-14 (3.2)*</u>

$$V_f = w_f \times A_{Tributary} = 14.60 \times 3.17 = 46.30 \text{ kN}$$

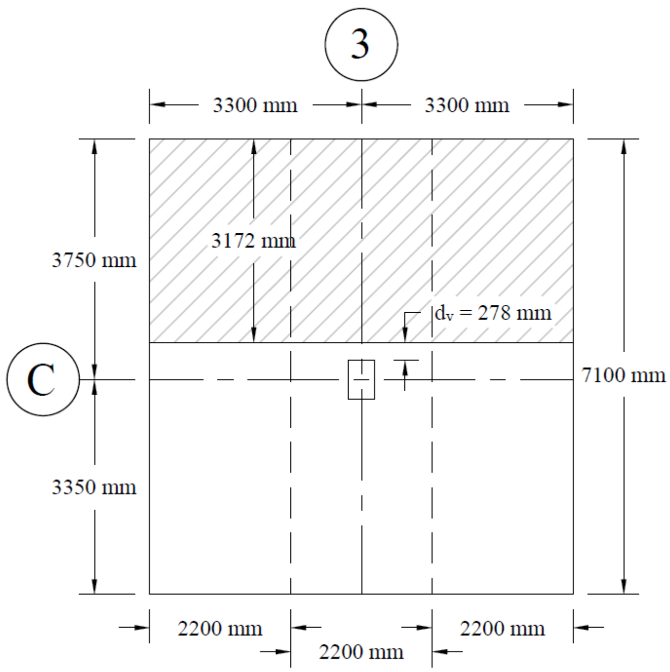
$$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v$$ **<u>CSA A23.3-14 (Eq. 11.6)</u>**

Where $\lambda = 1$ for normal weight concrete

$\beta = 0.21$ for slabs with overall thickness not greater than 350 mm **<u>CSA A23.3-14 (11.3.6.2)</u>**

$$V_c = 0.65 \times 1 \times 0.21 \times \sqrt{25} \times 1000 \times \frac{278}{1000} = 189.8 \text{ kN} > V_f$$

Slab band thickness of 350 mm is adequate for one-way shear for the first critical section (from the edge of the column).



<u>Figure 6 – Critical Sections for One-Way Shear</u>

1.2.5. Slab shear strength – two-way shear

<u>For critical section at distance *d*/2 from the edge of the column (slab section with slab band):</u>

Check the adequacy of slab thickness for punching shear (two-way shear) at an interior column (Figure 7):

Tributary area for two-way shear is $A_{Tributary} = (7.5/2 + 6.7/2) \times (6.6) - (0.6 + 0.309) \times (0.4 + 0.309)$

$$= 46.22 \text{ m}^2$$

$$V_f = w_f \times A_{Tributary} = 14.60 \times 46.22 = 675 \text{ kN}$$

$$b_o = 2 \times (600 + 309) + 2 \times (400 + 309) = 3236 \text{ mm}$$ **<u>CSA A23.3-14 (13.3.3)</u>**

$$v_f = \frac{V_f}{b_o d} = \frac{675 \times 1000}{3236 \times 309} = 0.675 \text{ MPa}$$

The factored resisting shear stress, v_r shall be the smallest of : **_CSA A23.3-14 (13.3.4.1)_**

a) $v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c}$ **_CSA A23.3-14 (Eq. 13.5)_**

$$v_r = \left(1 + \frac{2}{1.5}\right) \times 0.19 \times 0.65 \times \sqrt{25} = 1.44 \text{ MPa}$$

Where $\beta_c = \frac{600}{400} = 1.5$ (ratio of long side to short side of the column) **_CSA A23.3-14 (13.3.4.1)_**

b) $v_r = v_c = \left(\frac{\alpha_s d}{b_o} + 0.19\right) \lambda \phi_c \sqrt{f'_c}$ **_CSA A23.3-14 (Eq. 13.6)_**

$$v_r = \left(\frac{4 \times 309}{3236} + 0.19\right) \times 1 \times 0.65 \times \sqrt{25} = 1.86 \text{ MPa}$$

c) $v_r = v_c = 0.38 \lambda \phi_c \sqrt{f'_c} = 0.38 \times 1 \times 0.65 \times \sqrt{25} = 1.24 \text{ MPa}$ (Governs) **_CSA A23.3-14 (Eq. 13.7)_**

$\frac{v_r}{(v_f)_{avr}} = \frac{1.240}{0.675} = 1.84 > 1.20$ **_CAC Concrete Design Handbook 4th Edition (5.2.3)_**

Slab band thickness of 350 mm is adequate for two-way shear for the critical section (from the edge of the column).

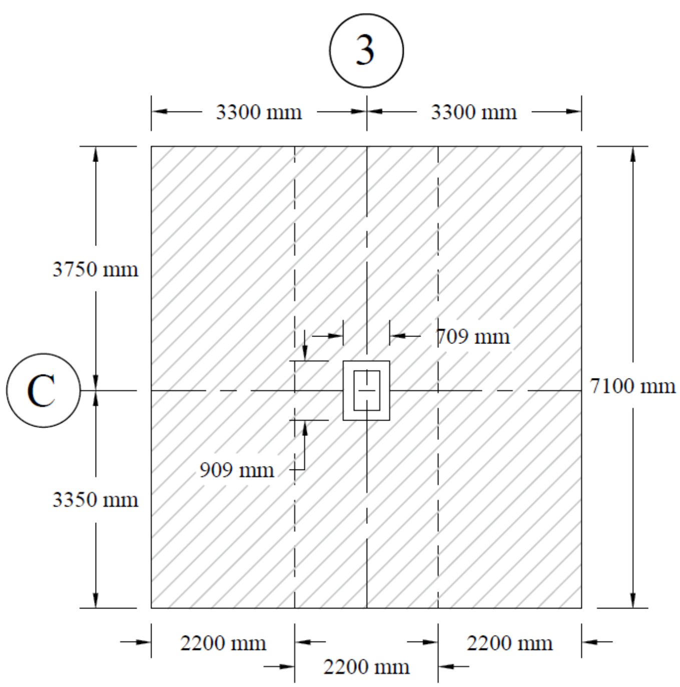


Figure 7 – Critical Section for Two-Way Shear

While the preliminary sizes determined above and summarized in the Figure 8 leads to a more optimal design, we will proceed with the dimensions provided in the reference example (Example #3 of CAC Design Handbook) for comparison purposes (see Figure 9 below).

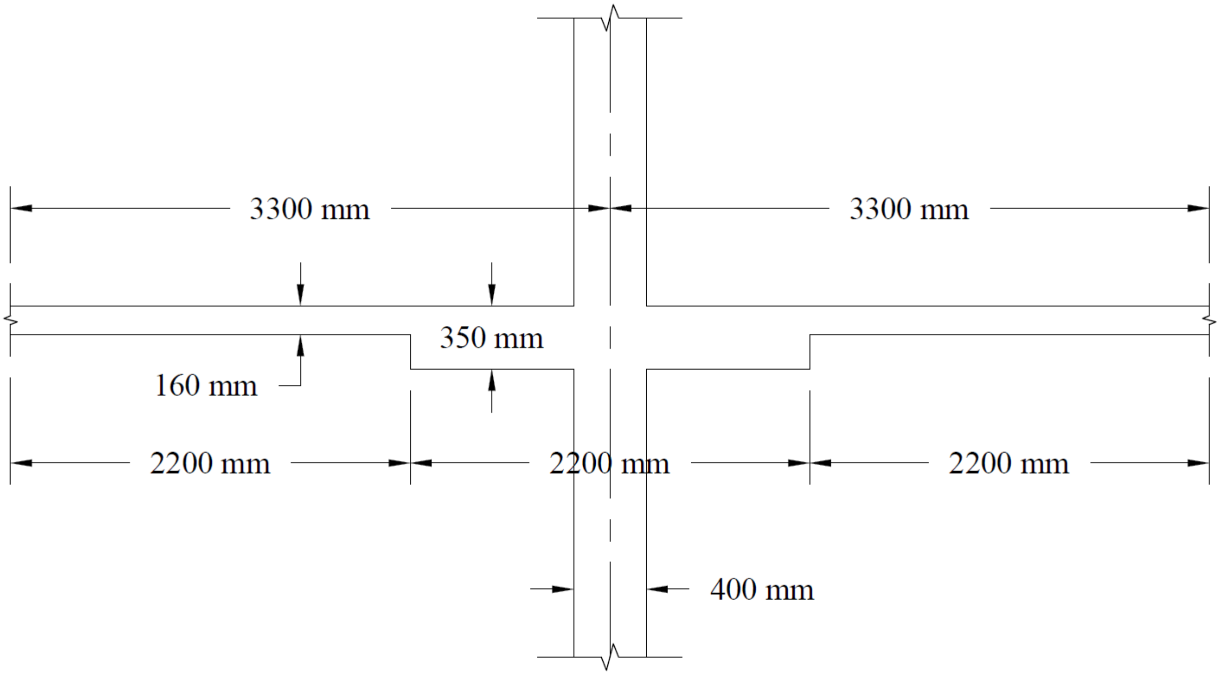


Figure 8 – Section A-A (As Calculated Above)

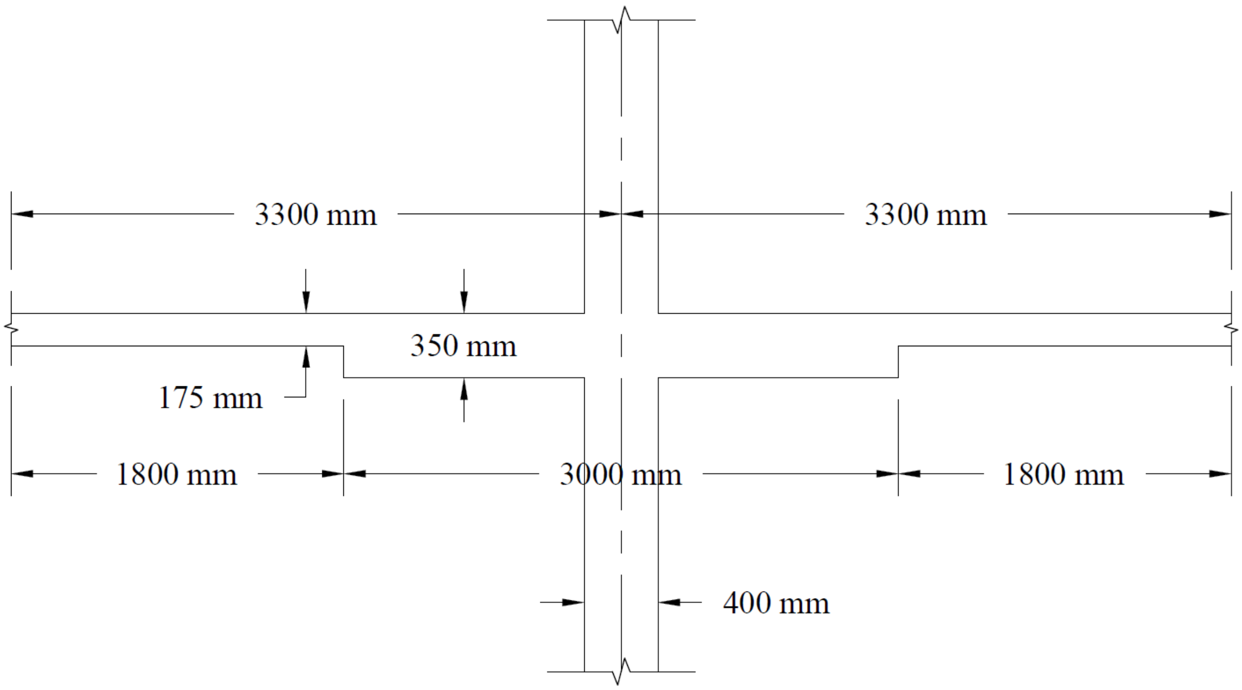


Figure 9 – Section A-A (As Calculated by the Reference)

1.3. Preliminary Member Sizing For Columns

Check the adequacy of column dimensions for axial load:

Tributary area for interior column for live load, superimposed dead load, and self-weight of the slab is

$$A_{Tributary} = \frac{7.5+6.7}{2} \times 6.6 = 46.86 \text{ m}^2$$

Tributary area for interior column for self-weight of additional slab thickness due to the presence of the slab band is

$$A_{Tributary} = \frac{7.5+6.7}{2} \times 3 = 21.3 \text{ m}^2$$

Assuming five story building

$$P_f = n \times w_f \times A_{Tributary} = 5 \times (13.05 \times 46.86 + 5.15 \times 21.3) = 3606 \text{ kN}$$

Assume 600 mm x 400 mm column with 12 – 30M vertical bars with design axial strength, $P_{r,max}$ of

$$P_{r,\max} = (0.2 + 0.002h)P_{ro} \le 0.80 P_{ro} \text{ (For tied column along full length)}$$ **_CSA A23.3-14 (Eq. 10.9)_**

$$P_{ro} = \alpha_1 \ \phi_c \ f'_c (A_g - A_{st} - A_t - A_p) + \phi_s \ \text{f}_y \ \text{A}_{st} + \phi_\alpha F_y A_t - f_{pr} A_p$$ **_CSA A23.3-14 (Eq. 10.11)_**

$$P_{ro} = 0.81 \times 0.65 \times 25 \times (600 \times 400 - 12 \times 700) + 0.85 \times 400 \times (12 \times 700) + 0 = 5904 \text{ kN}$$

$$P_{r,\max} = (0.2 + 0.002 \times 600) \times 5904 \le 0.80 \times 5904$$

$$= 8266 \le 4723$$

$$= 4723 \text{ kN} > P_f = 3606 \text{ kN}$$

Where:

$$\alpha_1 = 0.85 - 0.0015 f'_c = 0.85 - 0.0015 \times 25 = 0.81 > 0.67$$ **_CSA A23.3-14 (Eq. 10.1)_**

Column dimensions of 600 mm × 400 mm are adequate for axial load.

2. Two-Way Slab Analysis and Design

CSA A23.3 states that a regular slab system may be designed using any procedure satisfying conditions of equilibrium and compatibility with the supports, provided that it is shown that the factored resistance at every section is at least equal to the effects of the factored loads and that all serviceability conditions, including specified limits on deflections, are met. ***CSA A23.3-14 (13.5.1)***

CSA A23.3 permits the use of Plastic Plate Theory Method (PPTM), Theorems of Plasticity Method (TPM), Direct Design Method (DDM) and Elastic Frame Method (EFM); known as Equivalent Frame Method in the ACI; for the gravity load analysis of orthogonal frames. The following sections outline a brief description of DDM, a detailed hand solution using EFM and an automated solution using spSlab software respectively.

2.1. Direct Design Method (DDM)

Two-way slabs satisfying the limits in ***CSA A23.3-14 (13.9)*** are permitted to be designed in accordance with the DDM.

2.1.1. Direct design method limitations

There shall be a minimum of three continuous spans in each direction (3 spans) ***CSA A23.3-14 (13.9.1.2)***

Successive span lengths centre-to-centre of supports in each direction shall not differ by more than one- third of the longer span ((7500-6700)/6700 = 0.12 < 0.33) ***CSA A23.3-14 (13.9.1.3)***

All loads shall be due to gravity only and uniformly distributed over an entire panel (Loads are uniformly distributed over the entire panel) ***CSA A23.3-14 (13.9.1.4)***

The factored live load shall not exceed twice the factored dead load (factored live-to-dead load ratio of 0.71 < 2.0) ***CSA A23.3-14 (13.9.1.4)***

Since all the criteria are met, Direct Design Method is utilized in the CAC Design Handbook.

Even though this system meets all the limitations of the DDM, based on engineering judgment, DDM is not recommended to be used with floor systems with slab bands since the generic moment distribution factors used in DDM might, in some cases as this example, underestimate the negative moment values since these factors were derived based on a two-way slab systems without beams between interior supports (Flat Plate). The stiffer the supports (due to the precence of drop panels and slab bands) the more moments the supports will carry. The EFM takes into consideration detailed geometry of the cross section and the slab-beam distribution factors are calculated exactly. This calculation can be tedious and complicated to be done by hand for slab systems with different thicknesses but computer aids such as spSlab or spMats can be utilized. There are design aids tables that can be utilized for simplifying hand calculation. Howerver, the available tables are only applicable for flat plates and some special cases of slabs with drop panels. There are no design aid tables for two-way slabs with slab bands, slabs with beams between all supports, or two-way joist (waffle) slabs. For these systems, using the available design aid tables might in some cases also underestimate the moment values at the supports.

Detailed illustration of analysis and design of a two-way flat plate concrete slab system using DDM can be found in "Two-Way Flat Plate Concrete Slab Floor Analysis and Design (CSA A23.3-14)" example available in the design examples page in StructurePoint website. This example focuses on the analysis of two-way slabs with slab bands using EFM.

2.1.2. Design moments

a. Calculate the total factored static moment:

$$M_o = \frac{w_f \ell_{2a} \ell_n^{\,2}}{8}$$ ***CSA A23.3-14 (13.9.1.4)***

Distribute the total factored moment, M_o, in an interior and end span: ***CSA A23.3-14 (13.9.3.1 &13.9.3.2)***

Table 1 - Distribution of M_o along the span

Location		**Total Design Strip Moment, M_{DES} (kN.m)**
Exterior Span	Exterior Negative	$0.26 \times M_o = 34.8$
	Positive	$0.52 \times M_o = 69.6$
	Interior Negative	$0.70 \times M_o = 93.68$
Interior Span	Positive	$0.35 \times M_o = 46.8$

b. Calculate the column strip moments. ***CSA A23.3-14 (13.11.2)***

That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips. ***CSA A23.3-14 (13.11.3.1)***

Table 2 - Lateral Distribution of the Total Design Strip Moment, M_{DES}

Location		**Total Design Strip Moment, (kN.m)**	**Column Strip Moment, (kN.m)**	**Moment in Two Half Middle Strips, (kN.m)**
Exterior Span	Exterior Negative*	34.8	$1.00 \times M_{DES} = 34.8$	$0.00 \times M_{DES} = 0.0$
	Positive	69.6	$0.6 \times M_{DES} = 41.8$	$0.4 \times M_{DES} = 27.8$
	Interior Negative*	93.68	$0.8 \times M_{DES} = 74.94$	$0.2 \times M_{DES} = 18.7$
Interior Span	Positive	46.8	$0.6 \times M_{DES} = 28.1$	$0.4 \times M_{DES} = 18.7$

* All negative moments are at face of support.

Figure 10 – Sample Calculations Using DDM from "Two-Way Flat Plate Concrete Slab Floor Analysis and Design" Design Example

2.2. Elastic Frame Method (EFM)

EFM (also known as Equivalent Frame Method in the ACI 318) is the most comprehensive and detailed procedure provided by the CSA A23.3 for the analysis and design of two-way slab systems where these systems may, for purposes of analysis, be considered a series of plane frames acting longitudinally and transversely through the building. Each frame shall be composed of equivalent line members intersecting at member centrelines, shall follow a column line, and shall include the portion of slab bounded laterally by the centreline of the panel on each side. ***CSA A23.3-14 (13.8.1.1)***

Probably the most frequently used method to determine design moments in regular two-way slab systems is to consider the slab as a series of two-dimensonal frames that are analyzed elastically. When using this analogy, it is essential that stiffness properties of the elements of the frame be selected to properly represent the behavior of the three-dimensional slab system.

In a typical frame analysis it is assumed that at a beam-column cconnection all members meeting at the joint undergo the same rotaion. For uniform gravity loading this reduced restraint is accounted for by reducing the effective stiffness of the column by either Clause 13.8.2 or Clause 13.8.3. ***CSA A23.3-14 (N.13.8)***

Each floor and roof slab with attached columns may be analyzed separately, with the far ends of the columns considered fixed. ***CSA A23.3-14 (13.8.1.2)***

The moment of inertia of column and slab-beam elements at any cross-section outside of joints or column capitals shall be based on the gross area of concrete at that section. ***CSA A23.3-14 (13.8.2.5)***

An equivalent column shall be assumed to consist of the actual columns above and below the slab-beam plus an attached torsional member transverse to the direction of the span for which moments are being determined. ***CSA A23.3-14 (13.8.2.5)***

2.2.1. Elastic frame method limitations

In EFM, live load shall be arranged in accordance with 13.8.4 which requires:

- slab systems to be analyzed and designed for the most demanding set of forces established by investigating the effects of live load placed in various critical patterns. ***CSA A23.3-14 (13.8.4)***
- Complete analysis must include representative interior and exterior equivalent elastic frames in both the longitudinal and transverse directions of the floor. ***CSA A23.3-14 (13.8.1.1)***
- Panels shall be rectangular, with a ratio of longer to shorter panel dimensions, measured center-to-center of supports, not to exceed 2. ***CSA A23.3-14 (3.1a)***
- For slab systems with beams between sypports, the relative effective stiffness of beams in the two directions is not less than 0.2 or greater than 5.0. ***CSA A23.3-14 (3.1b)***
- Column offsets are not greater than 20% of the span (in the direction of offset) from either axis between centerlines of successive columns. ***CSA A23.3-14 (3.1c)***

The reinforcement is placed in an orthogonal grid. ***CSA A23.3-14 (3.1d)***

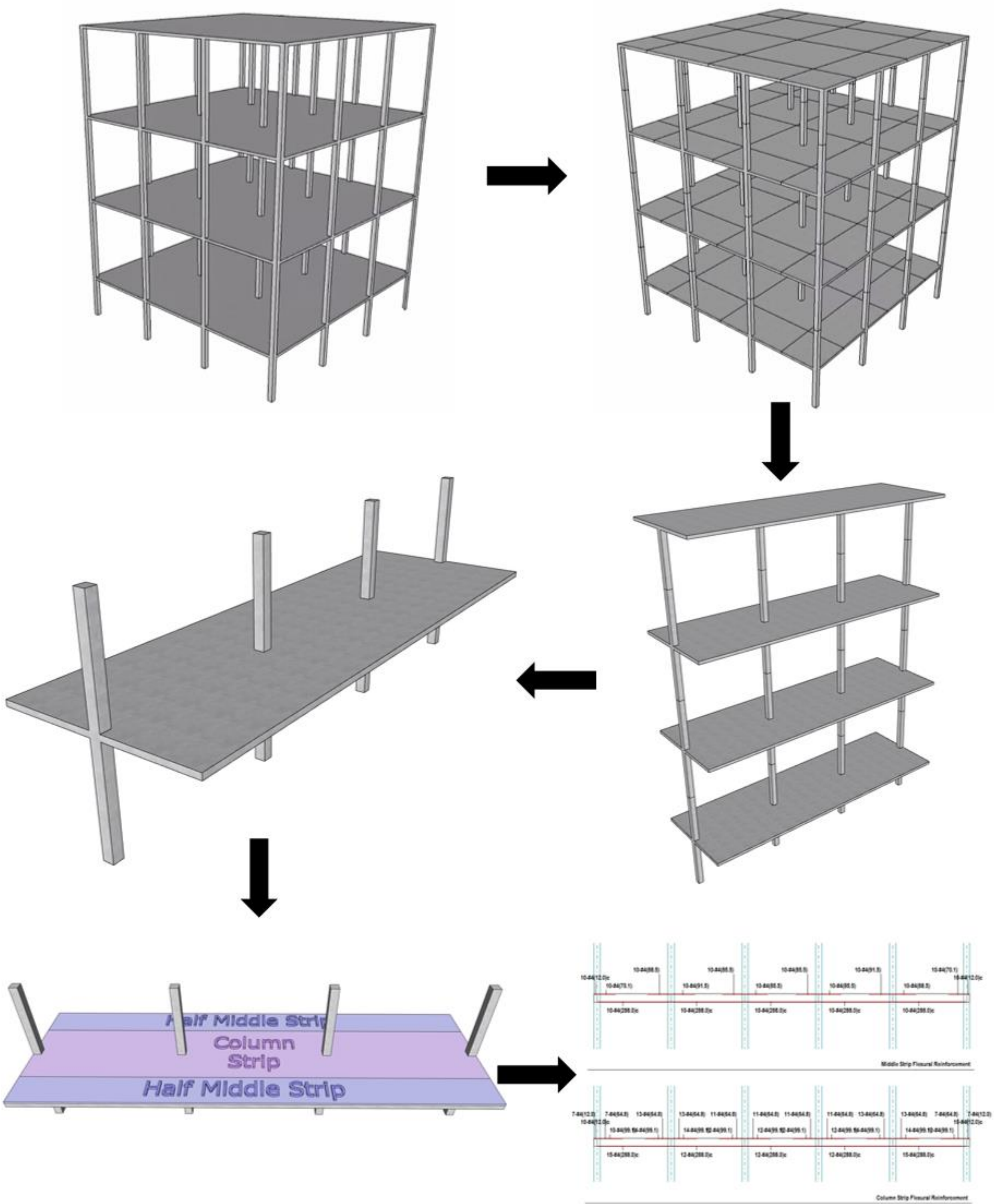


Figure 11 – Elastic (Equivalent) Frame Methodology

2.2.2. Frame members of elastic frame

Determine moment distribution factors and fixed-end moments for the elastic frame members. The moment distribution procedure will be used to analyze the equivalent frame. Stiffness factors k, carry over factors COF, and fixed-end moment factors FEM for the slab-beams and column members are determined using the design aids tables at ***Appendix 20A of PCA Notes on ACI 318-11***. Note that the available tables are limited to flat plate and slab with drop panels systems, litreture showed that these tables can be used for other systems for simplicity to an extent. This point will be discussed later in this document. These calculations are shown below.

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

Table A1 in the PCA Notes handbook will be used to calculate the flexural stiffness of slab-beams since the slab has a constant cross-section along the span length. This table has been adopted in this example and is deemed to represent the most comparable system for the analysis of two-way slab with longitudinal slab bands.

For Interior Span:

$$\frac{c_{N1}}{\ell_1} = \frac{600}{7500} = 0.080 \ , \ \frac{c_{N2}}{\ell_2} = \frac{600}{6600} = 0.061$$

For $c_{F1} = c_{N2}$, stiffness factors, $k_{NF} = k_{FN} = 4.09$ ***PCA Notes on ACI 318-11 (Table A1)***

Thus, $K_{sb,int} = k_{NF}\dfrac{E_{cs}I_s}{\ell_1} = 4.09\dfrac{E_{cs}I_s}{\ell_1}$ ***PCA Notes on ACI 318-11 (Table A1)***

Where I_s is the moment of inertia of slab-beam section shown in the following figure and can be computed as follows:

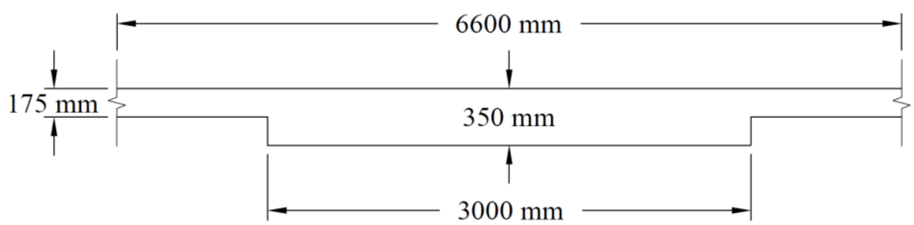


Figure 12 – Cross-Section of Slab-Beam

$$C_t = 1 + (A-1)B^3 + \frac{3(1-B)^2 B(A-1)}{1+B(A-1)} = 1.43$$ ***PCA Notes on ACI 318-11 (Figure 20-21)***

Where A = b/b_w = 6600 / 3000 = 2.2 and B = h_s/h = 175 / 350 = 0.5

$$I_s = C_t\left(\frac{b_w h^3}{12}\right) = 1.43\left(\frac{3000\times 350^3}{12}\right) = 15.34\times 10^9 \text{ mm}^4$$ ***PCA Notes on ACI 318-11 (Figure 20-21)***

$$E_{cs} = \left(3300\sqrt{f_c^{'}} + 6900\right)\left(\frac{\gamma_c}{2300}\right)^{1.5}$$ ***CSA A23.3-14(8.6.2.2)***

$$E_{cs} = (3300\sqrt{25} + 6900)\left(\frac{2402.8}{2300}\right)^{1.5} = 24,986 \text{ MPa}$$

$$K_{sb,int} = 4.09 \times 24,986 \times \frac{15.34 \times 10^9}{7500} \times 10^{-3} = 208.9 \times 10^6 \text{ N.m}$$

Carry-over factor COF = 0.505 ***PCA Notes on ACI 318-11 (Table A1)***

Fixed-end moment $\text{FEM} = 0.0840 w_u \ell_2 \ell_1^{\,2}$ ***PCA Notes on ACI 318-11 (Table A1)***

For Exterior Span:

$$\frac{c_{N1}}{\ell_1} = \frac{600}{6700} = 0.090 \;,\quad \frac{c_{N2}}{\ell_2} = \frac{400}{6600} = 0.061$$

For $c_{F1} = c_{N2}$, stiffness factors, $k_{NF} = k_{FN} = 4.10$ ***PCA Notes on ACI 318-11 (Table A1)***

Thus, $K_{sb,ext} = k_{NF}\dfrac{E_{cs} I_s}{\ell_1} = 4.10\dfrac{E_{cs} I_s}{\ell_1}$ ***PCA Notes on ACI 318-11 (Table A1)***

$$K_{sb,ext} = 4.10 \times 24,986 \times \frac{15.34 \times 10^9}{6700} \times 10^{-3} = 234.5 \times 10^6 \text{ N.m}$$

Carry-over factor COF = 0.505 ***PCA Notes on ACI 318-11 (Table A1)***

Fixed-end moment $\text{FEM} = 0.0841 w_u \ell_2 \ell_1^{\,2}$ ***PCA Notes on ACI 318-11 (Table A1)***

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

Referring to ***Table A7, Appendix 20A***,

For the Top Column (Above):

$$t_a = 350 - \frac{175}{2} = 262.5 \text{ mm} \text{ , } t_b = \frac{175}{2} = 87.5 \text{ mm}$$

$$\frac{t_a}{t_b} = \frac{262.5}{87.5} = 3$$

$$H = 3 \text{ m} = 3000 \text{ mm} \text{ , } H_c = 3000 \text{ mm} - 350 = 2650 \text{ mm}$$

$$\frac{H}{H_c} = \frac{3000}{2650} = 1.132$$

Thus, $k_{AB} = 6.02$ and $C_{AB} = 0.536$ by interpolation.

$$K_{c,top} = \frac{6.02 E_{cc} I_c}{\ell_c}$$ ***PCA Notes on ACI 318-11 (Table A7)***

$$K_{c,top} = 6.02 \times 24{,}986 \times \frac{7.20 \times 10^9}{3000 \times 1000} = 360.8 \times 10^6 \text{ N.m}$$

Where $I_c = \frac{b \times h^3}{12} = \frac{400(600)^3}{12} = 7.20 \times 10^9 \text{ mm}^4$

$$E_{cc} = (3300\sqrt{f_c^{'}} + 6900)\left(\frac{\gamma_c}{2300}\right)^{1.5}$$ ***CSA A23.3-14(8.6.2.2)***

$$E_{cc} = (3300\sqrt{25} + 6900)\left(\frac{2402.8}{2300}\right)^{1.5} = 24{,}986 \text{ MPa}$$

$$\ell_c = 3.00 \text{ m} = 3000 \text{ mm}$$

For the Bottom Column (Below):

$$\frac{t_b}{t_a} = \frac{87.5}{262.5} = 0.33$$

$$\frac{H}{H_c} = \frac{3000}{2650} = 1.132$$

Thus, $k_{BA} = 4.99$ and $C_{BA} = 0.641$ by interpolation.

$$K_c = \frac{4.99 E_{cc} I_c}{\ell_c}$$ ***PCA Notes on ACI 318-11 (Table A7)***

$$K_{c,bottom} = 4.99 \times 24{,}986 \times \frac{7.20 \times 10^9}{3000 \times 1000} = 299.1 \times 10^6 \text{ N.m}$$

c. Torsional stiffness of torsional members, K_t

$$K_t = \frac{9E_{cs}C}{\ell_t\left(1-\frac{c_2}{\ell_t}\right)^3}$$ ***CSA A23.3-14 (13.8.2.8)***

$$K_t = \frac{9\times 24{,}986\times 874.9\times 10^6}{6600\times\left(1-\frac{400}{6600}\right)^3}\times 10^{-3} = 35.96\times 10^6 \text{ N.m}$$

Where $C = \sum\left(1-0.63\frac{x}{y}\right)\left(\frac{x^3y}{3}\right)$ ***CSA A23.3-14(13.8.2.9)***

$$C = \left(1-0.63\times\frac{175}{600}\right)\left(\frac{175^3\times 600}{3}\right) = 874.9\times 10^6 \text{ mm}^4$$

c_2 = 400 mm, and l_t = 6.6 m = 6600 mm

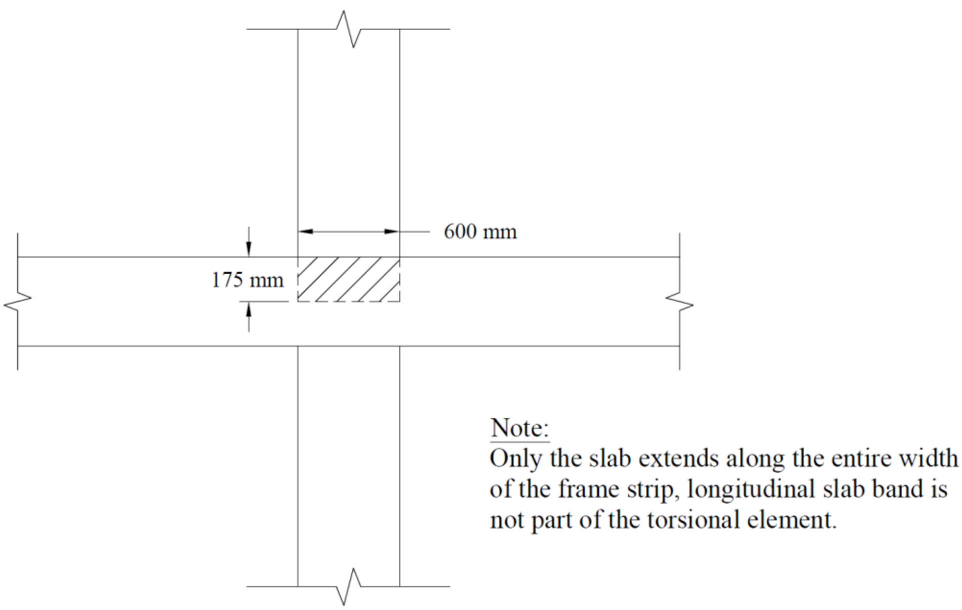


Figure 13 – Torsional Member

d. Increased torsional stiffness due to parallel beams, K_{ta}.

For Interior Columns:

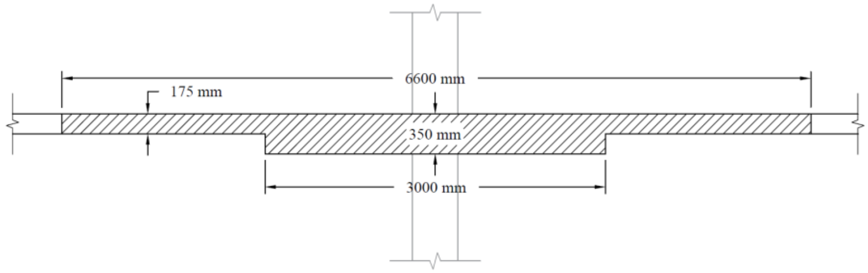


Figure 14 – Slab-Beam in the Direction of Analysis

$$K_{ta\_int} = \frac{K_{t\_int} I_{sb}}{I_s} = \left(35.96\times10^6\right)\times\frac{15.34\times10^9}{2.95\times10^9} = 187.2\times10^6 \text{ N.m}$$

Where:

$$I_s = \frac{l_2 \times h^3}{12} = \frac{6600\times175^3}{12} = 2.95\times10^9 \text{ mm}^4$$

For Exterior Columns:

$$K_{ta\_ext} = \frac{K_{t\_ext} I_{sb}}{I_s} = \left(35.96\times10^6\right)\times\frac{15.34\times10^9}{2.95\times10^9} = 187.2\times10^6 \text{ N.m}$$

e. Equivalent column stiffness, K_{ec}

Where $\sum K_t$ is for two torsional members one on each side of the column, and $\sum K_c$ is for the upper and lower columns at the slab-beam joint of an intermediate floor.

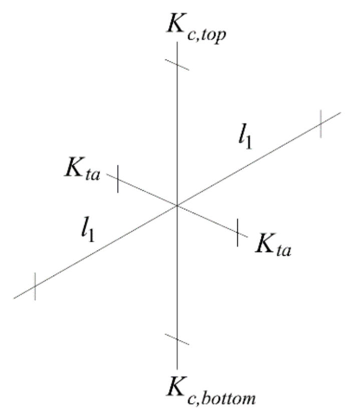


Figure 15 – Equivalent Column Stiffness

For Interior Columns:

$$K_{ec\_int}=\frac{\left(360.8\times10^{6}+299.1\times10^{6}\right)\times\left(2\times187.2\times10^{6}\right)}{\left(360.8\times10^{6}+299.1\times10^{6}\right)+\left(2\times187.2\times10^{6}\right)}=238.8\times10^{6}\text{ N.m}$$

For Exterior Columns:

$$K_{ec\_ext}=\frac{\left(360.8\times10^{6}+299.1\times10^{6}\right)\times\left(2\times187.2\times10^{6}\right)}{\left(360.8\times10^{6}+299.1\times10^{6}\right)+\left(2\times187.2\times10^{6}\right)}=238.8\times10^{6}\text{ N.m}$$

f. Slab-beam joint distribution factors, *DF*

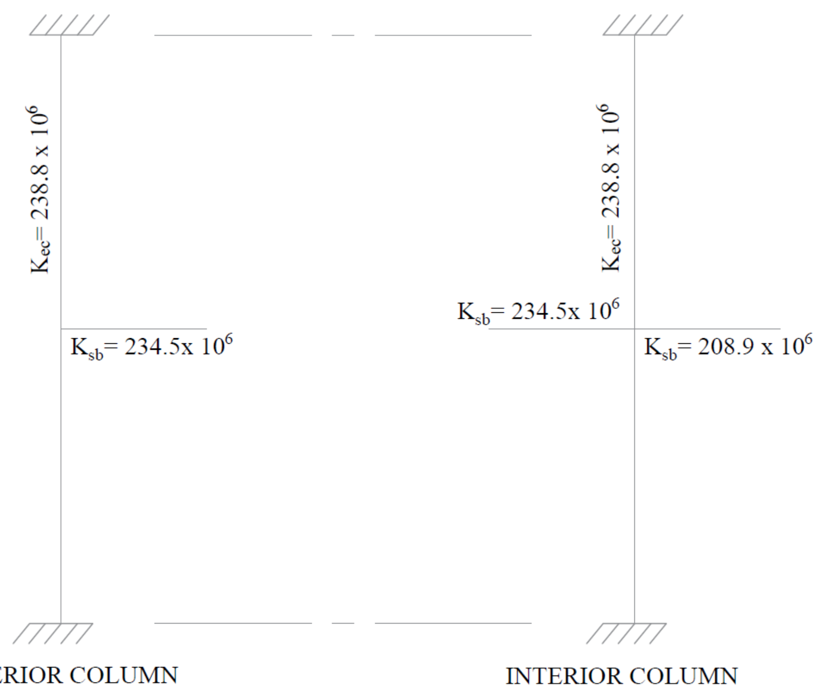


Figure 16 – Slab and Column Siffness

At exterior joint:

$$DF=\frac{234.5\times10^{6}}{\left(234.5\times10^{6}+238.8\times10^{6}\right)}=0.495$$

At interior joint:

$$DF_{Ext}=\frac{234.5\times10^{6}}{\left(234.5\times10^{6}+208.9\times10^{6}+238.8\times10^{6}\right)}=0.344$$

$$DF_{Int}=\frac{208.9\times10^{6}}{\left(234.5\times10^{6}+208.9\times10^{6}+238.8\times10^{6}\right)}=0.306$$

COF for slab-beam = 0.505 for Interior and Exterior Spans

2.2.3. Elastic frame analysis

Determine negative and positive moments for the slab-beams using the moment distribution method. Since the unfactored live load does not exceed three-quarters of the unfactored dead load, design moments are assumed to occur at all critical sections with full factored live on all spans. ***CSA A23.3-14 (13.8.4.2)***

$$\frac{L}{D}=\frac{3.6}{\left(175\times2400+(350-175)\times2400\times\frac{3}{6.6}+2\right)}=\frac{3.6}{(4.2+1.87+2)}=0.45<\frac{3}{4}$$

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

Factored dead load, $w_{df}=1.25\times\left(175\times2400+(350-175)\times2400\times\frac{3}{6.6}+2\right)$

$$w_{df}=1.25\times(4.2+1.87+2)=10\text{ kN/m}^2$$

Factored live load, $w_{lf}=1.5\times3.6=5.4\text{ kN/m}^2$

Total factored load $q_u=w_f=w_{df}+w_{lf}=15.4\text{ kN/m}^2$

FEM's for slab-beams $=m_{NF}q_u\ell_2\ell_1^2$ ***PCA Notes on ACI 318-11 (Table A1)***

$$=0.0840\times15.4\times6.6\times7.5^2=480.2\text{ kN.m (For interior span)}$$

$$=0.0841\times15.4\times6.6\times6.7^2=383.7\text{ kN.m (For exterior span)}$$

b. Moment distribution. Computations are shown in Table 1. Counterclockwise rotational moments acting on the member ends are taken as positive. Positive span moments are determined from the following equation:

$$M_u\text{ (midspan)}=M_o-\frac{M_{uL}+M_{uR}}{2}$$

Where M_o is the moment at the midspan for a simple beam.

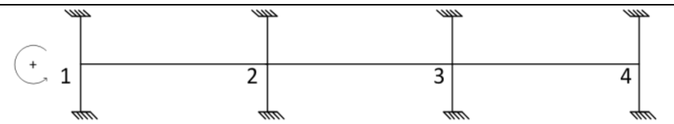
When the end moments are not equal, the maximum moment in the span does not occur at the midspan, the maximum positive moment for a uniformly distributed load and variable end moments can be calculated using any design aid as follows (For positive moment in span 1-2):

$$M_u^+=\frac{(15.4\times6.6)\times6.7^2}{8}-\frac{(193.7+479.8)}{2}+-\frac{(193.7-479.8)^2}{2\times(15.4\times6.6)\times6.7^2}=242.5\text{ kN.m}$$

<u>For positive moment span 2-3:</u>

$$M_u^+=\frac{(15.4\times6.6)\times7.5^2}{8}-\frac{(480.1+480.1)}{2}=234.6\text{ kN.m}$$

Table 1 – Moment Distribution for Elastic Frame



Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.495	0.344	0.306	0.306	0.344	0.495
COF	0.505	0.505	0.505	0.505	0.505	0.505
FEM	383.70	-383.70	480.20	-480.20	383.70	-383.70
Dist	-189.93	-33.20	-29.53	29.53	33.20	189.93
CO	-16.77	-95.91	14.91	-14.91	95.91	16.77
Dist	8.30	27.86	24.79	-24.79	-27.86	-8.30
CO	14.07	4.19	-12.52	12.52	-4.19	-14.07
Dist	-6.96	2.86	2.55	-2.55	-2.86	6.96
CO	1.44	-3.51	-1.29	1.29	3.51	-1.44
Dist	-0.71	1.65	1.47	-1.47	-1.65	0.71
CO	0.83	-0.36	-0.74	0.74	0.36	-0.83
Dist	-0.41	0.38	0.34	-0.34	-0.38	0.41
CO	0.19	-0.21	-0.17	0.17	0.21	-0.19
Dist	-0.09	0.13	0.12	-0.12	-0.13	0.09
CO	0.07	-0.05	-0.06	0.06	0.05	-0.07
Dist	-0.03	0.04	0.03	-0.03	-0.04	0.03
CO	0.02	-0.02	-0.02	0.02	0.02	-0.02
Dist	-0.01	0.01	0.01	-0.01	-0.01	0.01
CO	0.01	-0.01	-0.01	0.01	0.01	-0.01
Dist	0.00	0.00	0.00	0.00	0.00	0.00
M, kN.m	193.7	-479.8	480.1	-480.1	479.8	-193.7
Midspan M, kN.m	242.5		234.6		242.5	

2.2.4. Design moments

Positive and negative factored moments for the slab system in the direction of analysis are plotted in Figure 14. The negative moments used for design are taken at the faces of supports (rectangle section or equivalent rectangle for circular or polygon sections) but not at distances greater than $0.175\ell_1$ from the centers of supports. ***CSA A23.3-14 (13.8.5.1)***

$$\frac{600}{2} = 300 \text{ mm} < 0.175 \times 6700 = 1172.5 \text{ mm (use face of supporting location)}$$

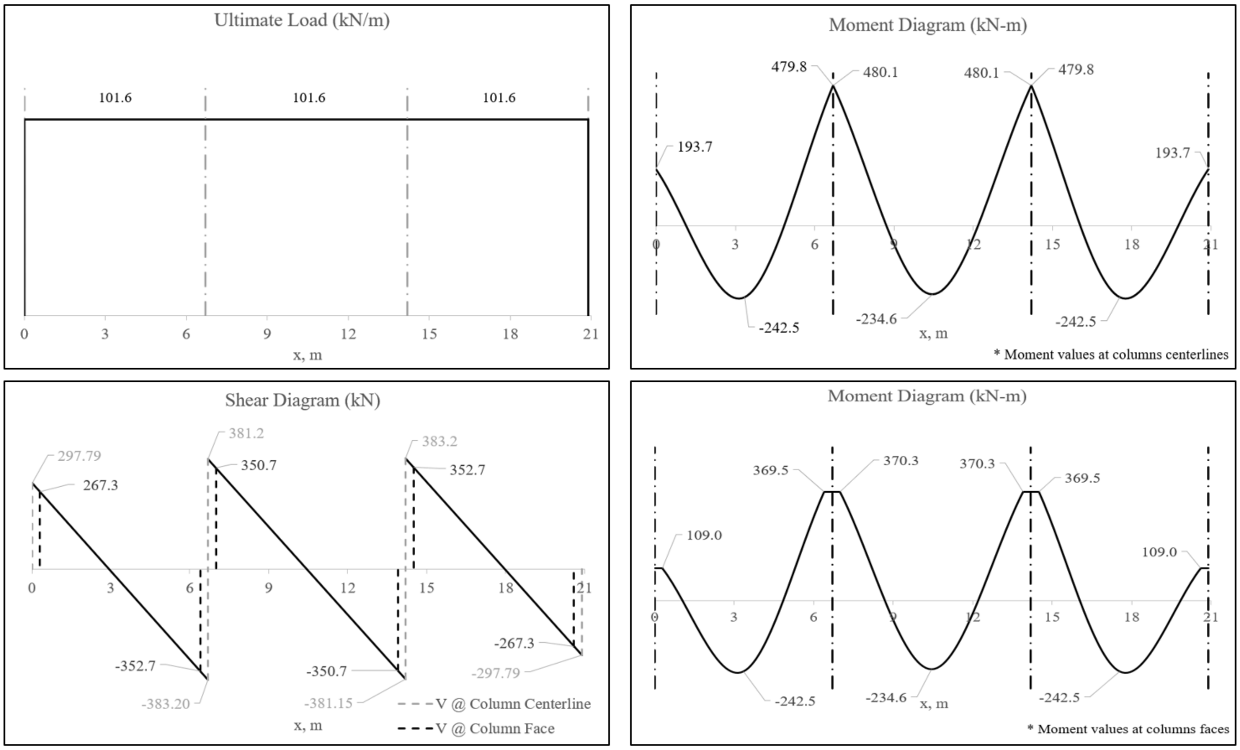


Figure 17 - Positive and Negative Design Moments for Slab-Beam (All Spans Loaded with Full Factored Live Load)

2.2.5. Distribution of design moments

After the negative and positive moments have been determined for the slab-beam strip, the CSA code permits the distribution of the moments at critical sections to the column strips, beams (if any), and middle strips in accordance with the DDM. ***CSA A23.3-14 (13.11.2.5)***

- For negative moment at an interior column, the column strip should resist 80% to 100% of the total frame strip moment.
- For negative moment at an exterior column, the column strip should resist 100% of the total frame strip moment.
- For positive moment at all spans, the column strip should resist 80% to 100% of the total frame strip moment.

Distribution of factored moments at critical sections is summarized in Table 2.

<table>
<tr><th colspan="7">Table 2 - Distribution of factored moments</th></tr>
<tr><td colspan="2" rowspan="2"></td><td>Slab-beam Strip</td><td colspan="2">Column Strip</td><td colspan="2">Middle Strip</td></tr>
<tr><td>Moment (kN.m)</td><td>Percent</td><td>Moment (kN.m)</td><td>Percent</td><td>Moment (kN.m)</td></tr>
<tr><td rowspan="3">End Span</td><td>Exterior Negative</td><td>109.0</td><td>100.0</td><td>109.0</td><td>0.0</td><td>0.0</td></tr>
<tr><td>Positive</td><td>242.5</td><td>90.0</td><td>218.3</td><td>10.0</td><td>24.2</td></tr>
<tr><td>Interior Negative</td><td>369.5</td><td>90.0</td><td>332.5</td><td>10.0</td><td>37.0</td></tr>
<tr><td rowspan="2">Interior Span</td><td>Negative</td><td>370.3</td><td>90.0</td><td>333.3</td><td>10.0</td><td>37.0</td></tr>
<tr><td>Positive</td><td>234.6</td><td>90.0</td><td>211.1</td><td>10.0</td><td>23.5</td></tr>
</table>

2.2.6. Flexural reinforcement requirements

a. Determine the flexural reinforcement required for strip moments

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

Reinforcement for the total factored negative moment transferred to the exterior columns shall be placed within a band width b_b. Temperature and shrinkage reinforcement determined as specified in clause 7.8.1 shall be provided in that section of the slab outside of the band region defined by b_b or as required by clause 13.10.9. ***CSA A23.3-14 (13.10.3)***

$M_r = 109 \text{ kN.m}$

band strip width, $b = 3000 \text{ mm}$

Middle strip width, $b = 6600 - 3000 = 3600 \text{ mm}$

Use d = 350 – (25+0.5(16)) = 317 mm

In this example, jd will be assumed to be taken equal to 0.986d. The assumptions will be verified once the area of steel in finalized.

Assume $jd = 0.986 \times d = 312.56 \text{ mm}$

$$A_s = \frac{M_f}{\phi_s f_y jd} = \frac{109}{0.85 \times 400 \times 0.987 \times 317} = 1025.6 \text{ mm}^2$$

$\alpha_1 = 0.85 - 0.0015 f_c^{'} = 0.81 > 0.67$ ***CSA A23.3-14 (10.1.7)***

Recalculate 'a' for the actual $A_s = 1025.6 \text{ mm}^2 \rightarrow a = \frac{\phi_s A_s f_y}{\phi_c \alpha_1 f'_c b} = \frac{0.85 \times 1025.6 \times 400}{0.65 \times 0.81 \times 25 \times 3000} = 8.80 \text{ mm}$

$$jd = d - \frac{a}{2} = 0.986d$$

Therefore, the assumption that jd equals to $0.986d$ is valid.

$\therefore A_{s,req} = 1025.6 \text{ mm}^2$

Reinforcement for the total factored negative moment transferred to the exterior columns shall be placed within a band width b_b. ***CSA A23.3-14 (13.10.3)***

For negative reinforcement in the band defined by b_b:

$$b_b = c_2 + 2\times(1.5\times h_{sb}) = 400 + 2\times(1.5\times 350) = 1450 \text{ mm}$$

$$A_{s,\min} = \frac{0.2\times\sqrt{f_c'}}{f_y}\times b_b\times h_{sb}$$ ***CSA A23.3-14 (Eq. 10.4)***

$$A_{s,\min} = \frac{0.2\times\sqrt{25}}{400}\times 1450\times 350 = 1268.75 \text{ mm}^2 > A_{s,req}$$

$\therefore A_{s,min}$ governs

Provide 7 - 15M bars (1400 mm^2 > 1268.75 mm^2)

Maximum spacing for negative reinforcement in the band defined by b_b:

$$s_{\max} = 1.5h_{sb} = 525 \text{ mm} \le 250 \text{ mm}$$

s_{max} = 250 mm > $s_{provided}$ = 1450/7 = 207.14 mm (**O.K.**) ***CSA A23.3-14 (13.10.4)***

Temperature and shrinkage reinforcement shall be provided in that section of the slab outside of the band region defined by b_b (including middle strip and the remaining part of the band strip outside the band region). ***CSA A23.3-14 (13.10.3)***

For the remaining reinforcement of the band strip outside b_b:

$$A_{s,\min} = \frac{0.2\times\sqrt{f_c'}}{f_y}\times A_g = \frac{0.2\times\sqrt{25}}{400}\times 350\times(3000-1450) = 1356.3 \text{ mm}^2$$ ***CSA A23.3-14 (Eq. 10.4)***

Provide 7 - 15M bars (1400 mm^2 > 1356.3 mm^2)

Maximum spacing for the remaining reinforcement of the band strip outside b_b:

$$s_{\max} = 3h_{sb} = 1050 \text{ mm} \le 500 \text{ mm}$$

s_{max} = 500 mm > $s_{provided}$ = (3000-1450)/7 = 221.4 mm (**O.K.**) ***CSA A23.3-14 (13.10.4)***

Total reinforcement in the band Strip:

(7 – 15M) + (7 – 15M) = (14 – 15M)

$$(A_{s,\min})_{\text{Band Strip}} = (A_{s,\min})_{\text{Within bb}} + (A_{s,\min})_{\text{Remaining}} = 1268.75 + 1356.3 = 2625 \text{ mm}^2$$

For middle strip:

$$A_{s,\min} = 0.002A_g = 0.002\times 175\times 3600 = 1260 \text{ mm}^2$$ ***CSA A23.3-14 (7.8.1)***

Provide 8 - 15M bars (1600 mm^2 > 1260 mm^2)

8 bars are used instead of 7 bars to meet the maximum spacing requirement as shown below.

Maximum spacing for negative moment reinforcement in middle:

$$s_{max} = 3h_s = 525 \text{ mm} \leq 500 \text{ mm}$$

s_{max} = 500 mm > $s_{provided}$ = 3600/8 = 450 mm (**O.K.**) ***CSA A23.3-14 (13.10.4)***

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

Table 3 - Required Slab Reinforcement for Flexure

Span Location		M_r (kN.m)	b (m)	d (mm)	A_s Req'd for flexure (mm²)	Min A_s (mm²)	Reinforcement Provided	A_s Prov. for flexure (mm²)
End Span								
Band Strip	Exterior Negative	109.0	3000	317	1025.6	2625	14 - 15M*	2800
	Positive	218.3	3000	317	2084.2	2625	14 - 15M	2800
	Interior Negative	332.5	3000	317	3225.9	2625	17 - 15M	3400
Middle Strip	Exterior Negative	0.0	3600	142	0.0	1260	8 - 15M*†	1600
	Positive	24.2	3600	142	508.3	1260	8 - 15M†	1600
	Interior Negative	37.0	3600	142	781.8	1260	8 - 15M†	1600
Interior Span								
Band Strip	Negative	333.3	3000	317	3234.0	2625	17 - 15M	3400
	Positive	211.1	3000	317	2013.5	2625	14 - 15M	2800
Middle Strip	Negative	37.0	3600	142	781.8	1260	8 - 15M†	1600
	Positive	23.5	3600	142	492.9	1260	8 - 15M†	1600

* The reinforcement is selected to meet CSA A23.3-14 provision 13.10.3.
† Number of bars governed by maximum allowable spacing.

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

When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of unbalanced moment given by γ_f shall be transferred by flexural reinforcement placed within a width b_b. ***CSA A23.3-14 (13.10.2)***

Portion of the unbalanced moment transferred by flexure is $\gamma_f \times M_r$

$$\gamma_f = \frac{1}{1+(2/3)\times\sqrt{b_1/b_2}}$$ ***CSA A23.3-14 (13.10.2)***

Where

b_1 = Width width of the critical section for shear measured in the direction of the span for which moments are determined according to CSA A23.3-14, clause 13 (see Figure 18).

b_2 = Width of the critical section for shear measured in the direction perpendicular to b_1 according to CSA A23.3-14, clause 13 (see Figure 18).

b_b = Effective slab width = $c_2 + 3\times h_s$ ***CSA A23.3-14 (3.2)***

$$b_b = 400 + 3 \times 350 = 1450 \text{ mm}$$

For Exterior Column

$$b_1 = 100 + 600 + \frac{317}{2} = 858.5 \text{ mm}$$

$$b_2 = 400 + 317 = 717 \text{ mm}$$

$$\gamma_f = \frac{1}{1 + \left(\frac{2}{3}\right) \times \sqrt{\frac{858.5}{717}}} = 0.578$$

For Interior Column

$$b_1 = 600 + 317 = 917 \text{ mm}$$

$$b_2 = 400 + 317 = 717 \text{ mm}$$

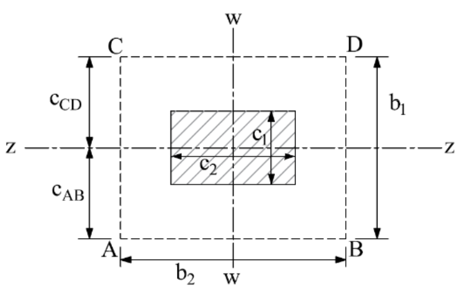
$$\gamma_f = \frac{1}{1 + \left(\frac{2}{3}\right) \times \sqrt{\frac{917}{717}}} = 0.570$$

Repeat the same procedure in section 2.2.6.a to calculate the additional reinforcement required for the unbalanced moment as shown in the following table:

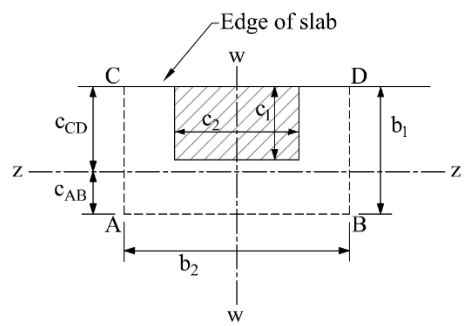
Table 4 - Additional Slab Reinforcement required for moment transfer between slab and column

Span Location		M_u* (kN.m)	γ_f	$\gamma_f M_u$ (kN.m)	Effective slab width, b_b (mm)	d (mm)	A_s req'd within b_b (mm^2)	A_s prov. For flexure within b_b (mm^2)	Add'l Reinf.
End Span									
Column Strip	Exterior Negative	193.7	0.578	112.0	1450	317	1071.3	1400	-
	Interior Negative	0.3	0.570	0.17	1450	317	1.6	1600	-

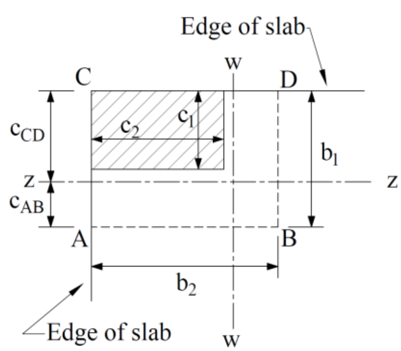
*M_u is taken at the centerline of the support in Elastic Frame Method solution.



Critical shear perimeter for interior column



Critical shear perimeter for exterior column



Critical shear perimeter for corner column

Figure 18 - Critical Shear Perimeters for Columns

2.2.7. Factored moments in columns

The unbalanced moment from the slab-beams at the supports of the equivalent frame are distributed to the support columns above and below the slab-beam in proportion to the relative stiffness of the support columns. Detailed calculations regarding this topic (including column design for axial load and biaxial moments) can be found in "Two-Way Flat Slab (Drop Panels) Concrete Floor Analysis and Design (CSA A23.3-14)" example available in the design examples page in StructurePoint website.

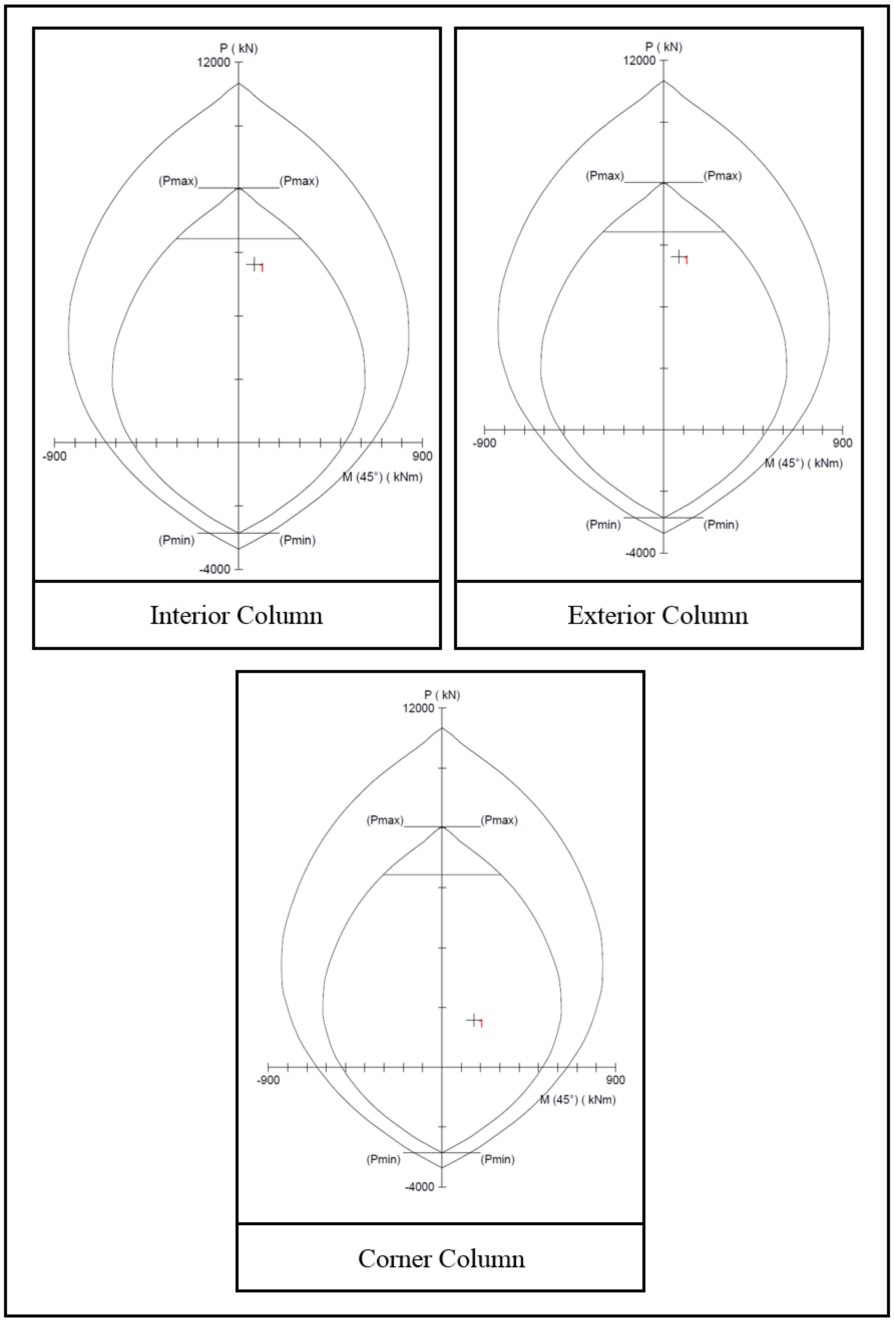


Figure 19 - Sample Calculations of Column Design from "Two-Way Flat Slab (Drop Panels) Concrete Floor Analysis and Design" Design Example

3. Two-Way Slab Shear Strength

Shear strength of the slab in the vicinity of columns/supports includes an evaluation of one-way shear (beam action) and two-way shear (punching) in accordance with CSA A23.3-14 clause 13.

3.1. One-Way (Beam action) Shear Strength

CSA A23.3-14 (13.3.6)

One-way shear is critical at a distance d_v from the face of the column and slab band. Figure 20 shows the factored shear forces (V_f) at the critical sections. In members without shear reinforcement, the design shear capacity of the section equals to the design shear capacity of the concrete:

$$V_r = V_c + V_s + V_p = V_c \quad , \quad (V_s = V_p = 0)$$ **CSA A23.3-14 (Eq. 11.4)**

Where:

$$V_c = \phi_c \lambda \beta \sqrt{f_c^{'}} b d_v$$ **CSA A23.3-14 (Eq. 11.5)**

3.1.1. Shear capacity of the entire frame strip

It is assumed that the shear is resisted by the slab band:

$h_{sb} = 350$ mm

d = 350 – (25-0.5(16)) = 317 mm

$$d_v = Max\begin{Bmatrix} 0.9 \times d \\ 0.72 \times h \end{Bmatrix} = Max\begin{Bmatrix} 0.9 \times 317 \\ 0.72 \times 350 \end{Bmatrix} = Max\begin{Bmatrix} 285.3 \\ 252.0 \end{Bmatrix} = 285.3 \text{ mm}$$ **CSA A23.3-14 (3.2)**

$\lambda = 1$ for normal weight concrete

$\beta = 0.21$ for slabs with overall thickness not greater than 350 mm **CSA A23.3-14 (11.3.6.2)**

$$V_c = 0.65 \times 1.0 \times 0.21 \times \sqrt{25} \times 3.0 \times \frac{285.3}{1000} = 584.15 \text{ kN} > V_f$$

Because $V_c \geq V_f$ at all the critical sections, the slab has adequate one-way shear strength.

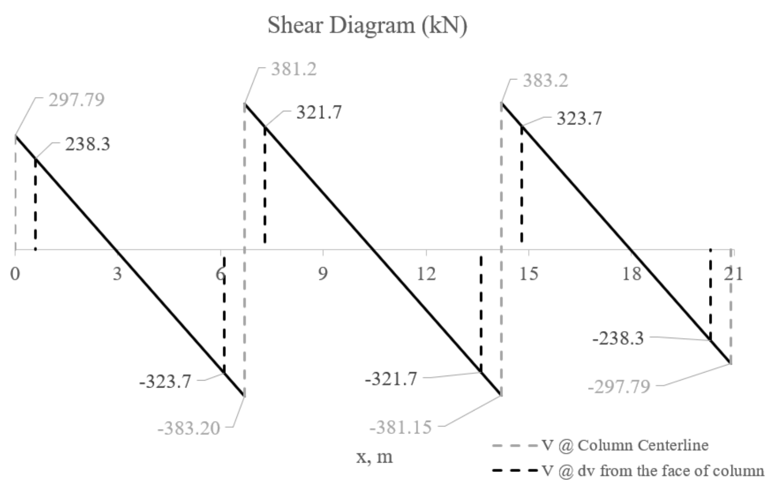


Figure 20 – One-way shear at critical sections (at distance d_v from the face of the supporting column)

3.2. Two-Way (Punching) Shear Strength

CSA A23.3-14 (13.3.2)

Two-way shear is critical on a rectangular section located at d/2 away from the face of the column as shown in Figure 18.

a. Exterior column:

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

$$V_f = 297.8 - 15.4\left(0.8585 \times 0.717 - 0.600 \times 0.400\right) = 292.01\,\text{kN}$$

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

$$\mathrm{M_{unb}} = \mathrm{M_u} - \mathrm{M}_f\left(\frac{b_1 - c_{AB} - c_1/2 - 100\text{ mm}}{1000\text{ mm}}\right)$$

$$\mathrm{M_{unb}} = 193.7 - 292.01\left(\frac{858.5 - 302.8 - 600/2 - 100}{1000}\right) = 148.24\text{ kN.m}$$

For the exterior column in Figure 18, the location of the centroidal axis z-z is:

$$\mathrm{c_{AB}} = e = \frac{\text{moment of area of the sides about AB}}{\text{area of the sides}} = \frac{2 \times (858.5 \times 317 \times 858.5/2)}{2 \times 858.5 \times 317 + 717 \times 317} = 302.8\text{ mm}$$

The polar moment J_c of the shear perimeter is:

$$\mathrm{J}_c = 2\left(\frac{b_1 d^3}{12} + \frac{d b_1^3}{12} + \left(b_1 d\right)\left(\frac{b_1}{2} - c_{AB}\right)^2\right) + b_2 d c_{AB}^2$$

$$\mathrm{J}_c = 2\left(\frac{858.5 \times 317^3}{12} + \frac{317 \times 858.5^3}{12} + \left(858.5 \times 317\right)\left(\frac{858.5}{2} - 302.8\right)^2\right) + 717 \times 317 \times (302.8)^2$$

$$\mathrm{J}_c = 67.53 \times 10^9\text{ mm}^4$$

$$\gamma_v = 1 - \gamma_f = 1 - 0.578 = 0.422$$ ***CSA A23.3-14 (Eq. 13.8)***

The length of the critical perimeter for the exterior column:

$$\mathrm{b_o} = 2 \times 858.5 + 717 = 2434\,\text{mm}$$

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

$$v_f = \frac{V_f}{b_o \times d} + \frac{\gamma_v M_{unb} e}{J}$$ ***CSA A23.3-14 (Eq.13.9)***

$$v_f = \frac{292.01 \times 1000}{2434 \times 317} + \frac{0.422 \times (148.8 \times 10^6) \times 302.8}{67.53 \times 10^9}$$

$$v_f = 0.378 + 0.281 = 659 \text{ MPa}$$

The factored resisiting shear stress, v_r shall be the smallest of : ***CSA A23.3-14 (13.3.4.1)***

a) $v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c} = \left(1 + \frac{2}{1.5}\right) 0.19 \times 0.65 \times \sqrt{25} = 1.441 \text{ MPa}$

Where $\beta_c = c_1/c_2 = 600/400 = 1.5$

b) $v_r = v_c = \left(\frac{\alpha_s d}{b_o} + 0.19\right) \lambda \phi_c \sqrt{f'_c} = \left(\frac{3 \times 317}{2434} + 0.19\right) \times 1 \times 0.65 \times \sqrt{25} = 1.887 \text{ MPa}$

Where $\alpha_s = 3$ for edge columns

c) $v_r = v_c = 0.38 \lambda \phi_c \sqrt{f'_c} = 0.38 \times 1 \times 0.65 \times \sqrt{25} = 1.235 \text{ MPa}$

$$v_r = v_c = \min \begin{Bmatrix} 1.441 \\ 1.887 \\ 1.235 \end{Bmatrix} = 1.235 \text{ MPa}$$

CSA A23.3 requires multiplying the value of v_c by 1300/(1000+d) if the effective depth used in the two-way shear calculations exceeds 300 mm. ***CSA A23.3-14 (13.3.4.3)***

$$v_c = \left(\frac{1300}{1000 + 317}\right) \times 1.235 = 1.219 \text{ MPa}$$

Since ($v_r = 1.219 \text{ MPa} \geq v_f = 0.659 \text{ MPa}$) at the critical section, the slab with slab band has adequate two-way shear strength at this joint.

b. Interior column:

$$V_f = (381.2 + 383.2) - 15.4 \times (0.917 \times 0.717 - 0.600 \times 0.400) = 757.97 \text{ kN}$$

$$M_{unb} = (480.1 - 479.8) - 757.97 \times 0 = 0.3 \text{ kN.m}$$

For the interior column in Figure 18, the location of the centroidal axis z-z is:

$$c_{AB} = \frac{b_1}{2} = \frac{917}{2} = 458.5 \text{ mm}$$

The polar moment J_c of the shear perimeter is:

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

$$\text{J}_c = 2\left(\frac{917 \times 317^3}{12} + \frac{317 \times 917^3}{12} + (917 \times 317)\left(\frac{917}{2} - 458.5\right)^2\right) + 2 \times 717 \times 317 \times (458.5)^2$$

$$\text{J}_c = 141.17 \times 10^9 \text{ mm}^4$$

$$\gamma_v = 1 - \gamma_f = 1 - 0.570 = 0.430$$ **CSA A23.3-14 (Eq. 13.8)**

The length of the critical perimeter for the interior column:

$$\text{b}_o = 2 \times (917 + 717) = 3268 \text{ mm}$$

$$v_f = \frac{V_f}{b_o \times d} + \frac{\gamma_v M_{unb} e}{J}$$ **CSA A23.3-14 (Eq.13.9)**

$$v_f = \frac{757.97 \times 1000}{3268 \times 317} + \frac{0.430 \times (0.13 \times 10^6) \times 458.5}{141.17 \times 10^9}$$

$$v_f = 0.732 + 0.000 = 0.732 \text{ MPa}$$

The factored resisiting shear stress, V_r shall be the smallest of : **CSA A23.3-14 (13.3.4.1)**

a) $v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c} = \left(1 + \frac{2}{1.5}\right) 0.19 \times 0.65 \times \sqrt{25} = 1.441 \text{ MPa}$

b) $v_r = v_c = \left(\frac{\alpha_s d}{b_o} + 0.19\right) \lambda \phi_c \sqrt{f'_c} = \left(\frac{4 \times 317}{3268} + 0.19\right) \times 1 \times 0.65 \times \sqrt{25} = 1.879 \text{ MPa}$

c) $v_r = v_c = 0.38 \lambda \phi_c \sqrt{f'_c} = 0.38 \times 1 \times 0.65 \times \sqrt{25} = 1.235 \text{ MPa}$

$$v_r = v_c = \min \begin{Bmatrix} 1.441 \\ 1.879 \\ 1.235 \end{Bmatrix} = 1.235 \text{ MPa}$$

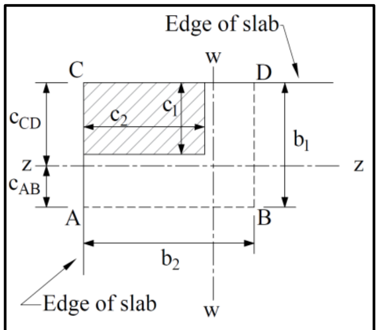
CSA A23.3 requires multiplying the value of v_c by 1300/(1000+d) if the effective depth used in the two-way shear calculations exceeds 300 mm. ***CSA A23.3-14 (13.3.4.3)***

$$v_c = \left(\frac{1300}{1000+317}\right)\times 1.235 = 1.219 \text{ MPa}$$

Since ($v_r = 1.219 \text{ MPa} \geq v_f = 0.728 \text{ MPa}$) at the critical section, the slab with slab band has adequate two-way shear strength at this joint.

c. Corner column:

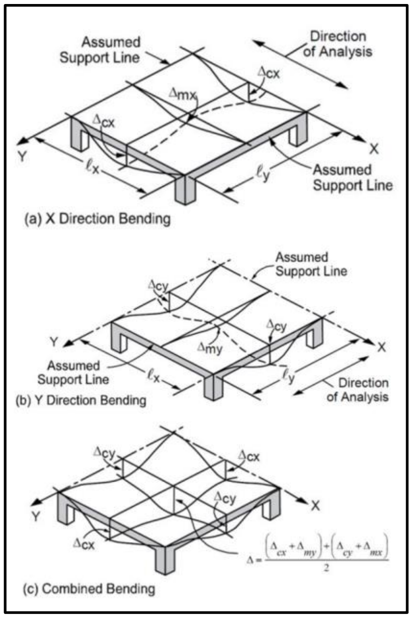
In this example, interior equivalent elastic frame strip was selected where it only have exterior and interior supports (no corner supports are included in this strip). Detailed calculations for two-way (punching) shear check around corner supports can be found in "Two-Way Flat Plate Concrete Slab Floor Analysis and Design (CSA A23.3-14)" example available in the design examples page in StructurePoint website.



Critical shear perimeter for corner column

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

Since the slab thickness was selected based on the minimum slab thickness equations in CSA A23.3-14, the deflection calculations are not required. Detailed calculations of immediate and time-dependent deflections can be found in "Two-Way Flat Plate Concrete Slab Floor Analysis and Design (CSA A23.3-14)" example available in the design examples page in StructurePoint website.

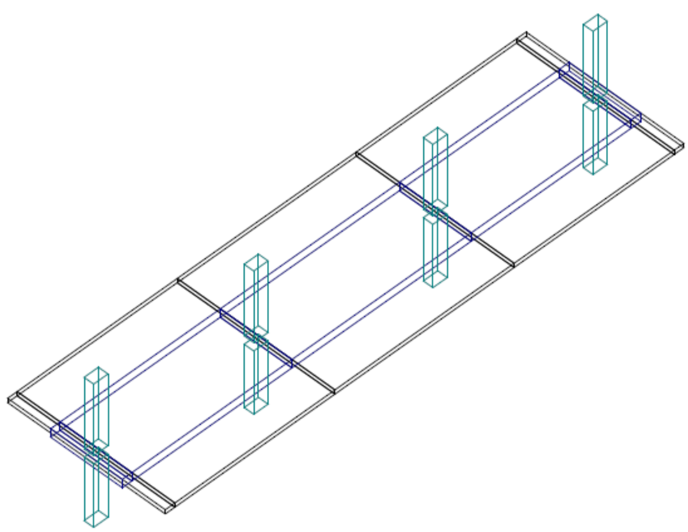


5. spSlab Software Solution

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

spSlab Program models the equivalent elastic frame as a design strip. The design strip is, then, separated by spSlab into column and middle strips. The program calculates the internal forces (Shear Force & Bending Moment), moment and shear capacity vs. demand diagrams for column and middle strips, instantaneous and long-term deflection results, and required flexural reinforcement for column and middle strips. The graphical and text results are provided below for both input and output of the spSlab model.

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



Contents

1. Input Echo

1.1. General Information

File Name	C:\CSA A...\Slab with Longitudinal Slab Bands.slb
Project	CAC 4th Edition - Section 5.7 - Example 3
Frame	Interior
Engineer	SP
Code	CSA A23.3-14
Reinforcement Database	CSA G30.18
Mode	Design
Number of supports =	4 + Left cantilever + Right cantilever
Floor System	Two-Way
Slab Bands	Longitudinal

1.2. Solve Options

Live load pattern ratio = 0%
Minimum free edge distance for punching shear = 5 times slab effective depth.
Circular critical section around circular supports used (if possible).
Deflections are based on cracked section properties.
In negative moment regions, Ig and Mcr DO NOT include flange/slab contribution (if available)
Long-term deflections are calculated for load duration of 60 months.
0% of live load is sustained.
Compression reinforcement calculations NOT selected.
Default incremental rebar design selected.
Combined M-V-T reinforcement design NOT selected.
User-defined slab strip widths NOT selected.
User-defined distribution factors NOT selected.
One-way shear in drop panel NOT selected.
Distribution of shear to strips selected.
Beam T-section design NOT selected.
Longitudinal beam contribution in negative reinforcement design over support NOT selected.
Transverse beam contribution in negative reinforcement design over support NOT selected.

1.3. Material Properties

1.3.1. Concrete: Slabs / Bands

w_c	2402.8 kg/m³
f'_c	25 MPa
E_c	24986 MPa
f_r	1.5 MPa
Precast concrete	No

1.3.2. Concrete: Columns

w_c	2402.8 kg/m³
f'_c	25 MPa
E_c	24986 MPa
f_r	1.5 MPa
Precast concrete	No

1.3.3. Reinforcing Steel

f_y	400 MPa
f_{yt}	400 MPa
E_s	200000 MPa
Epoxy coated bars	No

1.4. Reinforcement Database

Size	Db	Ab	Wb	Size	Db	Ab	Wb
	mm	mm²	kg/m		mm	mm²	kg/m
#10	11	100	1	#15	16	200	2
#20	20	300	2	#25	25	500	4
#30	30	700	5	#35	36	1000	8
#45	44	1500	12	#55	56	2500	20

1.5. Span Data

1.5.1. Slabs

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

Span	Loc	L1	t	wL	wR	L2L	L2R	H_{min}
		m	mm	m	m	m	m	mm
1	Int	0.400	175	3.300	3.300	6.600	6.600	--- LC *i
2	Int	6.700	175	3.300	3.300	6.600	6.600	133
3	Int	7.500	175	3.300	3.300	6.600	6.600	133
4	Int	6.700	175	3.300	3.300	6.600	6.600	133
5	Int	0.400	175	3.300	3.300	6.600	6.600	--- RC *i

1.5.2. Longitudinal Slab Bands

Span	b	h	Offset	H_{min}
	mm	mm	mm	mm
1	3000	350	0	13
2	3000	350	0	339
3	3000	350	0	329
4	3000	350	0	339
5	3000	350	0	13

1.6. Support Data

1.6.1. Columns

Support	c1a	c2a	Ha	c1b	c2b	Hb	Red %
	mm	mm	m	mm	mm	m	
1	600	400	3.000	600	400	3.000	100
2	600	400	3.000	600	400	3.000	100
3	600	400	3.000	600	400	3.000	100
4	600	400	3.000	600	400	3.000	100

1.6.2. Boundary Conditions

Support	Spring		Far End	
	K_z	K_{ry}	Above	Below
	kN/mm	kN-mm/rad		
1	0	0	Fixed	Fixed
2	0	0	Fixed	Fixed
3	0	0	Fixed	Fixed

Support	Spring		Far End	
	K_z	K_{ry}	Above	Below
	kN/mm	kN-mm/rad		
4	0	0	Fixed	Fixed

1.7. Load Data

1.7.1. Load Cases and Combinations

Case	Dead	Live
Type	DEAD	LIVE
U1	1.250	1.500

1.7.2. Area Loads

Case/Patt	Span	Wa
		kN/m^2
Dead	2	8.00
	3	8.00
	4	8.00
Live	2	3.60
	3	3.60
	4	3.60

1.8. Reinforcement Criteria

1.8.1. Slabs and Ribs

	Units	Top Bars		Bottom Bars	
		Min.	Max.	Min.	Max.
Bar Size		#15	#15	#15	#15
Bar spacing	mm	25	500	25	500
Reinf ratio	%	0.14	5.00	0.14	5.00
Clear Cover	mm	25		25	

There is NOT more than 300 mm of concrete below top bars.

1.8.2. Longitudinal Slab Bands

	Units	Top Bars		Bottom Bars	
		Min.	Max.	Min.	Max.
Bar Size		#15	#15	#15	#15
Bar spacing	mm	25	500	25	500
Reinf ratio	%	0.14	5.00	0.14	5.00
Clear Cover	mm	25		25	
Layer dist.	mm	25		25	

There is NOT more than 300 mm of concrete below top bars.

2. Design Results*

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

2.1. Strip Widths and Distribution Factors

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

		Width			Moment Factor			Shear Factor		
Span	Strip	Left **	Right **	Bottom *	Left **	Right **	Bottom *	Left	Center	Right
		m	m	m	m	m	m	m	m	m
1	Middle	3.60	3.60	3.60	0.000	0.000	0.100	0.000	0.000	0.000
	Band	3.00	3.00	3.00	1.000	1.000	0.900	1.000	1.000	1.000
2	Middle	3.60	3.60	3.60	0.000	0.100	0.100	0.000	0.000	0.000
	Band	3.00	3.00	3.00	1.000	0.900	0.900	1.000	1.000	1.000
3	Middle	3.60	3.60	3.60	0.100	0.100	0.100	0.000	0.000	0.000
	Band	3.00	3.00	3.00	0.900	0.900	0.900	1.000	1.000	1.000
4	Middle	3.60	3.60	3.60	0.100	0.000	0.100	0.000	0.000	0.000
	Band	3.00	3.00	3.00	0.900	1.000	0.900	1.000	1.000	1.000
5	Middle	3.60	3.60	3.60	0.000	0.000	0.100	0.000	0.000	0.000
	Band	3.00	3.00	3.00	1.000	1.000	0.900	1.000	1.000	1.000

2.2. Top Reinforcement

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

Span	Strip	Zone	Width	M_{max}	X_{max}	$A_{s,min}$	$A_{s,max}$	$A_{s,req}$	Sp_{Prov}	Bars
			m	kNm	m	mm²	mm²	mm²	mm	
1	Middle	Left	3.60	0.00	0.000	1260	11464	0	450	8-#15 *3 *5
		Midspan	3.60	0.00	0.165	1260	11464	0	450	8-#15 *3 *5
		Right	3.60	0.00	0.330	1260	11464	0	450	8-#15 *3 *5
	Band	Left	3.00	0.00	0.000	2625	21327	0	218	14-#15 *3
		Midspan	3.00	0.00	0.165	2625	21327	0	218	14-#15 *3
		Right	3.00	0.00	0.330	2625	21327	0	218	14-#15 *3
2	Middle	Left	3.60	0.19	0.523	1260	11464	4	450	8-#15 *3 *5
		Midspan	3.60	0.00	3.350	0	11464	0	0	---
		Right	3.60	36.98	6.400	1260	11464	781	450	8-#15 *3 *5
	Band	Left	3.00	108.94	0.300	2625	21327	1025	218	14-#15 *3
		Midspan	3.00	0.00	3.350	0	21327	0	0	---
		Right	3.00	332.82	6.400	2625	21327	3229	177	17-#15
3	Middle	Left	3.60	37.06	0.300	1260	11464	783	450	8-#15 *3 *5
		Midspan	3.60	0.00	3.750	0	11464	0	0	---
		Right	3.60	37.06	7.200	1260	11464	783	450	8-#15 *3 *5
	Band	Left	3.00	333.53	0.300	2625	21327	3236	177	17-#15
		Midspan	3.00	0.00	3.750	0	21327	0	0	---
		Right	3.00	333.53	7.200	2625	21327	3236	177	17-#15
4	Middle	Left	3.60	36.98	0.300	1260	11464	781	450	8-#15 *3 *5

Span	Strip	Zone	Width	M_{max}	X_{max}	$A_{s,min}$	$A_{s,max}$	$A_{s,req}$	Sp_{Prov}	Bars
			m	kNm	m	mm²	mm²	mm²	mm	
		Midspan	3.60	0.00	3.350	0	11464	0	0	---
		Right	3.60	0.19	6.177	1260	11464	4	450	8-#15 *3 *5
	Band	Left	3.00	332.82	0.300	2625	21327	3229	177	17-#15
		Midspan	3.00	0.00	3.350	0	21327	0	0	---
		Right	3.00	108.94	6.400	2625	21327	1025	218	14-#15 *3
5	Middle	Left	3.60	0.00	0.070	1260	11464	0	450	8-#15 *3 *5
		Midspan	3.60	0.00	0.235	1260	11464	0	450	8-#15 *3 *5
		Right	3.60	0.00	0.400	1260	11464	0	450	8-#15 *3 *5
	Band	Left	3.00	0.00	0.070	2625	21327	0	218	14-#15 *3
		Midspan	3.00	0.00	0.235	2625	21327	0	218	14-#15 *3
		Right	3.00	0.00	0.400	2625	21327	0	218	14-#15 *3

2.3. Top Bar Details

NOTES:
* - Bar cut-off location shall be manually checked for compliance with CSA A23.3, 11.2.13.

		Left				Continuous		Right			
Span	Strip	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
			m		m		m		m		m
1	Middle	---		---		8-#15	0.40	---		---	
	Band	---		---		14-#15	0.40	---		---	
2	Middle	8-#15 *	0.68	---		---		8-#15	2.02	---	
	Band	14-#15	1.13	---		---		14-#15	2.02	3-#15 *	0.76
3	Middle	8-#15	2.07	---		---		8-#15	2.07	---	
	Band	14-#15	2.07	3-#15 *	0.76	---		14-#15	2.07	3-#15 *	0.76
4	Middle	8-#15	2.02	---		---		8-#15 *	0.68	---	
	Band	14-#15	2.02	3-#15 *	0.76	---		14-#15	1.13	---	
5	Middle	---		---		8-#15	0.40	---		---	
	Band	---		---		14-#15	0.40	---		---	

2.4. Top Bar Development Lengths

		Left				Continuous		Right			
Span	Strip	Bars	DevLen	Bars	DevLen	Bars	DevLen	Bars	DevLen	Bars	DevLen
			mm		mm		mm		mm		mm
1	Middle	---		---		8-#15	300.00	---		---	
	Band	---		---		14-#15	300.00	---		---	
2	Middle	8-#15	300.00	---		---		8-#15	300.00	---	
	Band	14-#15	300.00	---		---		14-#15	425.89	3-#15	425.89
3	Middle	8-#15	300.00	---		---		8-#15	300.00	---	
	Band	14-#15	426.85	3-#15	426.85	---		14-#15	426.85	3-#15	426.85
4	Middle	8-#15	300.00	---		---		8-#15	300.00	---	
	Band	14-#15	425.89	3-#15	425.89	---		14-#15	300.00	---	
5	Middle	---		---		8-#15	300.00	---		---	

Span	Strip	Left Bars	Left DevLen (mm)	Left Bars	Left DevLen (mm)	Continuous Bars	Continuous DevLen (mm)	Right Bars	Right DevLen (mm)	Right Bars	Right DevLen (mm)
	Band	---		---		14-#15	300.00	---		---	

2.5. Band Reinforcement at Supports

NOTES:
<C> Total Strip, <B> Banded Strip, <S> Remaining Strip

Support	Width <C> (mm)	Width <B> (mm)	Width <S> (mm)	A_s <C> (mm²)	A_s <B> (mm²)	A_s <S> (mm²)	Bars <C>	Bars <B>	Bars <S>
1	3000	1450	1550	2800	1400	1400	14-#15	7-#15	7-#15
2	3000	1450	1550	3400	1600	1800	17-#15	8-#15	9-#15
3	3000	1450	1550	3400	1600	1800	17-#15	8-#15	9-#15
4	3000	1450	1550	2800	1400	1400	14-#15	7-#15	7-#15

2.6. Bottom Reinforcement

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

Span	Strip	Width (m)	M_{max} (kNm)	X_{max} (m)	$A_{s,min}$ (mm²)	$A_{s,max}$ (mm²)	$A_{s,req}$ (mm²)	Sp_{Prov} (mm)	Bars
1	Middle	3.60	0.00	0.165	0	11464	0	0	---
	Band	3.00	0.00	0.165	0	21327	0	0	---
2	Middle	3.60	24.23	2.904	1260	11464	508	450	8-#15 *3 *5
	Band	3.00	218.11	2.904	2625	21327	2082	218	14-#15 *3
3	Middle	3.60	23.42	3.713	1260	11464	491	450	8-#15 *3 *5
	Band	3.00	210.80	3.713	2625	21327	2011	218	14-#15 *3
4	Middle	3.60	24.23	3.796	1260	11464	508	450	8-#15 *3 *5
	Band	3.00	218.11	3.796	2625	21327	2082	218	14-#15 *3
5	Middle	3.60	0.00	0.235	0	11464	0	0	---
	Band	3.00	0.00	0.235	0	21327	0	0	---

2.7. Bottom Bar Details

Span	Strip	Long Bars: Bars	Long Bars: Start (m)	Long Bars: Length (m)	Short Bars: Bars	Short Bars: Start (m)	Short Bars: Length (m)
1	Middle	---			---		
	Band	---			---		
2	Middle	7-#15	0.00	6.70	1-#15	0.00	5.70
	Band	14-#15	0.00	6.70	---		
3	Middle	7-#15	0.00	7.50	1-#15	1.13	5.25
	Band	14-#15	0.00	7.50	---		
4	Middle	7-#15	0.00	6.70	1-#15	1.00	5.70
	Band	14-#15	0.00	6.70	---		
5	Middle	---			---		
	Band	---			---		

2.8. Bottom Bar Development Lengths

Span	Strip	Long Bars		Short Bars	
		Bars	DevLen	Bars	DevLen
			mm		mm
1	Middle	---		---	
	Band	---		---	
2	Middle	7-#15	300.00	1-#15	300.00
	Band	14-#15	333.50	---	
3	Middle	7-#15	300.00	1-#15	300.00
	Band	14-#15	322.00	---	
4	Middle	7-#15	300.00	1-#15	300.00
	Band	14-#15	333.50	---	
5	Middle	---		---	
	Band	---		---	

2.9. Flexural Capacity

Span	Strip	x	Top					Bottom				
			$A_{s,top}$	ΦM_n-	M_u-	Comb Pat	Status	$A_{s,bot}$	ΦM_n+	M_u+	Comb Pat	Status
		m	mm²	kNm	kNm			mm²	kNm	kNm		
1	Middle	0.000	1600	-74.13	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.115	1600	-74.13	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.200	1600	-74.13	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.215	1600	-74.13	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.330	1600	-74.13	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.400	1600	-74.13	0.00	U1 All	---	0	0.00	0.00	U1 All	---
	Band	0.000	2800	-290.34	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.115	2800	-290.34	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.200	2800	-290.34	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.215	2800	-290.34	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.330	2800	-290.34	0.00	U1 All	OK	0	0.00	0.00	U1 All	OK
		0.400	2800	-290.34	0.00	U1 All	---	0	0.00	0.00	U1 All	---
2	Middle	0.000	1600	-74.13	0.95	U1 All	---	1600	74.13	0.00	U1 All	---
		0.300	1600	-74.13	0.00	U1 All	OK	1600	74.13	0.00	U1 All	OK
		0.381	1600	-74.13	-0.12	U1 All	OK	1600	74.13	0.00	U1 All	OK
		0.523	843	-39.84	-0.19	U1 All	OK	1600	74.13	0.00	U1 All	OK
		0.681	0	0.00	-0.09	U1 All	*EXCEEDED	1600	74.13	0.00	U1 All	OK
		2.435	0	0.00	0.00	U1 All	OK	1600	74.13	22.99	U1 All	OK
		2.904	0	0.00	0.00	U1 All	OK	1600	74.13	24.23	U1 All	OK
		3.350	0	0.00	0.00	U1 All	OK	1600	74.13	23.34	U1 All	OK
		4.265	0	0.00	0.00	U1 All	OK	1600	74.13	15.16	U1 All	OK
		4.680	0	0.00	0.00	U1 All	OK	1600	74.13	8.66	U1 All	OK
		4.980	1600	-74.13	0.00	U1 All	OK	1600	74.13	2.87	U1 All	OK
		5.395	1600	-74.13	-5.57	U1 All	OK	1600	74.13	0.00	U1 All	OK
		5.695	1600	-74.13	-12.95	U1 All	OK	1400	65.21	0.00	U1 All	OK
		6.400	1600	-74.13	-36.98	U1 All	OK	1400	65.21	0.00	U1 All	OK
		6.700	1600	-74.13	-50.38	U1 All	---	1400	65.21	0.00	U1 All	---
	Band	0.000	2800	-290.34	-194.64	U1 All	---	2800	290.34	0.00	U1 All	---
		0.300	2800	-290.34	-108.94	U1 All	OK	2800	290.34	0.00	U1 All	OK
		0.828	2800	-290.34	0.00	U1 All	OK	2800	290.34	16.10	U1 All	OK
		1.128	0	0.00	0.00	U1 All	OK	2800	290.34	69.65	U1 All	OK
		2.435	0	0.00	0.00	U1 All	OK	2800	290.34	206.92	U1 All	OK

Span	Strip	Top							Bottom					
		x	$A_{s,top}$	ΦM_n-	M_u-	Comb	Pat	Status	$A_{s,bot}$	ΦM_n+	M_u+	Comb	Pat	Status
		m	mm²	kNm	kNm				mm²	kNm	kNm			
		2.904	0	0.00	0.00	U1	All	OK	2800	290.34	218.11	U1	All	OK
		3.350	0	0.00	0.00	U1	All	OK	2800	290.34	210.05	U1	All	OK
		4.265	0	0.00	0.00	U1	All	OK	2800	290.34	136.48	U1	All	OK
		4.680	0	0.00	0.00	U1	All	OK	2800	290.34	77.97	U1	All	OK
		5.106	2800	-290.34	0.00	U1	All	OK	2800	290.34	1.44	U1	All	OK
		5.944	2800	-290.34	-199.24	U1	All	OK	2800	290.34	0.00	U1	All	OK
		6.370	3400	-349.58	-323.55	U1	All	OK	2800	290.34	0.00	U1	All	OK
		6.400	3400	-349.58	-332.82	U1	All	OK	2800	290.34	0.00	U1	All	OK
		6.475	3400	-349.58	-356.40	U1	All	---	2800	290.34	0.00	U1	All	---
		6.700	3400	-349.58	-429.82	U1	All	---	2800	290.34	0.00	U1	All	---
3	Middle	0.000	1600	-74.13	-48.04	U1	All	---	1400	65.21	0.00	U1	All	---
		0.300	1600	-74.13	-37.06	U1	All	OK	1400	65.21	0.00	U1	All	OK
		1.125	1600	-74.13	-11.59	U1	All	OK	1400	65.21	0.00	U1	All	OK
		1.425	1600	-74.13	-4.05	U1	All	OK	1600	74.13	0.00	U1	All	OK
		1.767	1600	-74.13	0.00	U1	All	OK	1600	74.13	3.44	U1	All	OK
		2.067	0	0.00	0.00	U1	All	OK	1600	74.13	9.03	U1	All	OK
		2.715	0	0.00	0.00	U1	All	OK	1600	74.13	17.98	U1	All	OK
		3.713	0	0.00	0.00	U1	All	OK	1600	74.13	23.42	U1	All	OK
		3.750	0	0.00	0.00	U1	All	OK	1600	74.13	23.42	U1	All	OK
		4.785	0	0.00	0.00	U1	All	OK	1600	74.13	17.98	U1	All	OK
		5.433	0	0.00	0.00	U1	All	OK	1600	74.13	9.03	U1	All	OK
		5.733	1600	-74.13	0.00	U1	All	OK	1600	74.13	3.44	U1	All	OK
		6.075	1600	-74.13	-4.05	U1	All	OK	1600	74.13	0.00	U1	All	OK
		6.375	1600	-74.13	-11.59	U1	All	OK	1400	65.21	0.00	U1	All	OK
		7.200	1600	-74.13	-37.06	U1	All	OK	1400	65.21	0.00	U1	All	OK
		7.500	1600	-74.13	-48.04	U1	All	---	1400	65.21	0.00	U1	All	---
	Band	0.000	3400	-349.58	-432.33	U1	All	---	2800	290.34	0.00	U1	All	---
		0.240	3400	-349.58	-352.63	U1	All	---	2800	290.34	0.00	U1	All	---
		0.300	3400	-349.58	-333.53	U1	All	OK	2800	290.34	0.00	U1	All	OK
		0.330	3400	-349.58	-324.19	U1	All	OK	2800	290.34	0.00	U1	All	OK
		0.757	2800	-290.34	-198.96	U1	All	OK	2800	290.34	0.00	U1	All	OK
		1.640	2800	-290.34	0.00	U1	All	OK	2800	290.34	7.20	U1	All	OK
		2.067	0	0.00	0.00	U1	All	OK	2800	290.34	81.23	U1	All	OK
		2.715	0	0.00	0.00	U1	All	OK	2800	290.34	161.81	U1	All	OK
		3.713	0	0.00	0.00	U1	All	OK	2800	290.34	210.80	U1	All	OK
		3.750	0	0.00	0.00	U1	All	OK	2800	290.34	210.80	U1	All	OK
		4.785	0	0.00	0.00	U1	All	OK	2800	290.34	161.81	U1	All	OK
		5.433	0	0.00	0.00	U1	All	OK	2800	290.34	81.23	U1	All	OK
		5.860	2800	-290.34	0.00	U1	All	OK	2800	290.34	7.20	U1	All	OK
		6.743	2800	-290.34	-198.96	U1	All	OK	2800	290.34	0.00	U1	All	OK
		7.170	3400	-349.58	-324.19	U1	All	OK	2800	290.34	0.00	U1	All	OK
		7.200	3400	-349.58	-333.53	U1	All	OK	2800	290.34	0.00	U1	All	OK
		7.275	3400	-349.58	-357.46	U1	All	---	2800	290.34	0.00	U1	All	---
		7.500	3400	-349.58	-432.33	U1	All	---	2800	290.34	0.00	U1	All	---
4	Middle	0.000	1600	-74.13	-50.38	U1	All	---	1400	65.21	0.00	U1	All	---
		0.300	1600	-74.13	-36.98	U1	All	OK	1400	65.21	0.00	U1	All	OK
		1.005	1600	-74.13	-12.95	U1	All	OK	1400	65.21	0.00	U1	All	OK
		1.305	1600	-74.13	-5.57	U1	All	OK	1600	74.13	0.00	U1	All	OK
		1.720	1600	-74.13	0.00	U1	All	OK	1600	74.13	2.87	U1	All	OK
		2.020	0	0.00	0.00	U1	All	OK	1600	74.13	8.66	U1	All	OK
		2.435	0	0.00	0.00	U1	All	OK	1600	74.13	15.16	U1	All	OK
		3.350	0	0.00	0.00	U1	All	OK	1600	74.13	23.34	U1	All	OK

Span	Strip	x	$A_{s,top}$	ΦM_n-	M_u-	Comb	Pat	Status	$A_{s,bot}$	ΦM_n+	M_u+	Comb	Pat	Status
		Top							Bottom					
		m	mm²	kNm	kNm				mm²	kNm	kNm			
		3.796	0	0.00	0.00	U1	All	OK	1600	74.13	24.23	U1	All	OK
		4.265	0	0.00	0.00	U1	All	OK	1600	74.13	22.99	U1	All	OK
		6.019	0	0.00	-0.09	U1	All	*EXCEEDED	1600	74.13	0.00	U1	All	OK
		6.177	843	-39.84	-0.19	U1	All	OK	1600	74.13	0.00	U1	All	OK
		6.319	1600	-74.13	-0.12	U1	All	OK	1600	74.13	0.00	U1	All	OK
		6.400	1600	-74.13	0.00	U1	All	OK	1600	74.13	0.00	U1	All	OK
		6.700	1600	-74.13	0.95	U1	All	---	1600	74.13	0.00	U1	All	---
	Band	0.000	3400	-349.58	-429.82	U1	All	---	2800	290.34	0.00	U1	All	---
		0.240	3400	-349.58	-351.65	U1	All	---	2800	290.34	0.00	U1	All	---
		0.300	3400	-349.58	-332.82	U1	All	OK	2800	290.34	0.00	U1	All	OK
		0.330	3400	-349.58	-323.55	U1	All	OK	2800	290.34	0.00	U1	All	OK
		0.756	2800	-290.34	-199.24	U1	All	OK	2800	290.34	0.00	U1	All	OK
		1.594	2800	-290.34	0.00	U1	All	OK	2800	290.34	1.44	U1	All	OK
		2.020	0	0.00	0.00	U1	All	OK	2800	290.34	77.97	U1	All	OK
		2.435	0	0.00	0.00	U1	All	OK	2800	290.34	136.48	U1	All	OK
		3.350	0	0.00	0.00	U1	All	OK	2800	290.34	210.05	U1	All	OK
		3.796	0	0.00	0.00	U1	All	OK	2800	290.34	218.11	U1	All	OK
		4.265	0	0.00	0.00	U1	All	OK	2800	290.34	206.92	U1	All	OK
		5.572	0	0.00	0.00	U1	All	OK	2800	290.34	69.65	U1	All	OK
		5.872	2800	-290.34	0.00	U1	All	OK	2800	290.34	16.10	U1	All	OK
		6.400	2800	-290.34	-108.94	U1	All	OK	2800	290.34	0.00	U1	All	OK
		6.700	2800	-290.34	-194.64	U1	All	---	2800	290.34	0.00	U1	All	---
5	Middle	0.000	1600	-74.13	0.00	U1	All	---	0	0.00	0.00	U1	All	---
		0.070	1600	-74.13	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.186	1600	-74.13	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.200	1600	-74.13	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.285	1600	-74.13	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.400	1600	-74.13	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
	Band	0.000	2800	-290.34	0.00	U1	All	---	0	0.00	0.00	U1	All	---
		0.070	2800	-290.34	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.186	2800	-290.34	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.200	2800	-290.34	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.285	2800	-290.34	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK
		0.400	2800	-290.34	0.00	U1	All	OK	0	0.00	0.00	U1	All	OK

2.10. Longitudinal Slab Band Shear Capacity

Span	b	d_v	β	V_{ratio}	ΦV_c	V_u	X_u
	mm	mm			kN	kN	m
1	3000	285	0.210	1.000	584.15	0.00	0.00
2	3000	285	0.210	1.000	584.15	323.77	6.11
3	3000	285	0.210	1.000	584.15	321.66	6.91
4	3000	285	0.210	1.000	584.15	323.77	0.59
5	3000	285	0.210	1.000	584.15	0.00	0.40

2.11. Slab Shear Capacity

Span	Strip	b	d_v	β	V_{ratio}	ΦV_c	V_u	X_u
		mm	mm			kN	kN	m
1	Middle	3600	128	0.210	0.000	314.00	0.00	0.00
2	Middle	3600	128	0.210	0.000	314.00	0.00	0.00
3	Middle	3600	128	0.210	0.000	314.00	0.00	0.00
4	Middle	3600	128	0.210	0.000	314.00	0.00	0.00

Span	Strip	b	d_v	β	V_{ratio}	$ΦV_c$	V_u	X_u
		mm	mm			kN	kN	m
5	Middle	3600	128	0.210	0.000	314.00	0.00	0.40

2.12. Flexural Transfer of Negative Unbalanced Moment at Supports

Support	Width	Width-c	d	M_{unb}	Comb	Patt	$γ_f$	$A_{s,req}$	$A_{s,prov}$	Add Bars
	mm	mm	mm	kNm				mm²	mm²	
1	1450	1450	317	193.68	U1	All	0.578	1071	1400	---
2	1450	1450	317	0.16	U1	All	0.570	1	1600	---
3	1450	1450	317	0.16	U1	All	0.570	1	1600	---
4	1450	1450	317	193.68	U1	All	0.578	1071	1400	---

2.13. Punching Shear Around Columns

2.13.1. Critical Section Properties

Support	Type	b_1	b_2	b_0	d_{avg}	CG	$c_{(left)}$	$c_{(right)}$	A_c	J_c
		mm	mm	mm	mm	mm	mm	mm	mm²	mm⁴
1	Rect	858.5	717.0	2434.0	317.0	155.7	555.7	302.8	7.7158e+005	6.753e+010
2	Rect	917.0	717.0	3268.0	317.0	0.0	458.5	458.5	1.036e+006	1.4117e+011
3	Rect	917.0	717.0	3268.0	317.0	0.0	458.5	458.5	1.036e+006	1.4117e+011
4	Rect	858.5	717.0	2434.0	317.0	-155.7	302.8	555.7	7.7158e+005	6.753e+010

2.13.2. Punching Shear Results

Support	V_u	v_u	M_{unb}	Comb	Patt	$γ_v$	v_u	$ΦV_c$
	kN	N/mm²	kNm				N/mm²	N/mm²
1	292.67	0.379	148.12	U1	All	0.422	0.659	1.219
2	754.28	0.728	0.16	U1	All	0.430	0.728	1.219
3	754.28	0.728	-0.16	U1	All	0.430	0.728	1.219
4	292.67	0.379	-148.12	U1	All	0.422	0.659	1.219

2.14. Integrity Reinforcement at Supports

Notes:
The sum of bottom reinforcement crossing the perimeter of the support on all sides shall not be less than the below listed values.

Support	V_{se}	A_{sb}
	kN	mm²
1	299.18	1496
2	783.35	3917
3	783.35	3917
4	299.18	1496

2.15. Material TakeOff

2.15.1. Reinforcement in the Direction of Analysis

Top Bars	390.9 kg	<=>	18.01 kg/m	<=>	2.729 kg/m²
Bottom Bars	715.2 kg	<=>	32.96 kg/m	<=>	4.994 kg/m²
Stirrups	0.0 kg	<=>	0.00 kg/m	<=>	0.000 kg/m²
Total Steel	1106.1 kg	<=>	50.97 kg/m	<=>	7.723 kg/m²
Concrete	36.5 m³	<=>	1.68 m³/m	<=>	0.255 m³/m²

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}
		mm^4	mm^4	kNm	mm^4	mm^4	kNm
1	Left	1.5341e+010	0	110.73	1.0719e+010	2.4845e+009	-91.88
	Midspan	1.5341e+010	0	110.73	1.0719e+010	2.4845e+009	-91.88
	Right	1.5341e+010	0	110.73	1.0719e+010	2.4845e+009	-91.88
2	Left	1.5341e+010	1.7642e+009	110.73	1.0719e+010	2.4845e+009	-91.88
	Midspan	1.5341e+010	1.9633e+009	110.73	1.0719e+010	0	-91.88
	Right	1.5341e+010	1.7642e+009	110.73	1.0719e+010	2.76e+009	-91.88
3	Left	1.5341e+010	1.7642e+009	110.73	1.0719e+010	2.76e+009	-91.88
	Midspan	1.5341e+010	1.9633e+009	110.73	1.0719e+010	0	-91.88
	Right	1.5341e+010	1.7642e+009	110.73	1.0719e+010	2.76e+009	-91.88
4	Left	1.5341e+010	1.7642e+009	110.73	1.0719e+010	2.76e+009	-91.88
	Midspan	1.5341e+010	1.9633e+009	110.73	1.0719e+010	0	-91.88
	Right	1.5341e+010	1.7642e+009	110.73	1.0719e+010	2.4845e+009	-91.88
5	Left	1.5341e+010	0	110.73	1.0719e+010	2.4845e+009	-91.88
	Midspan	1.5341e+010	0	110.73	1.0719e+010	2.4845e+009	-91.88
	Right	1.5341e+010	0	110.73	1.0719e+010	2.4845e+009	-91.88

3.1.2. Frame Effective Section Properties

Span	Zone	Weight	Load Level					
			Dead		Sustained		Dead+Live	
			M_{max}	I_e	M_{max}	I_e	M_{max}	I_e
			kNm	mm^4	kNm	mm^4	kNm	mm^4
1	Right	1.000	0.00	1.5341e+010	0.00	1.5341e+010	0.00	1.5341e+010
	Span Avg	----	----	1.5341e+010	----	1.5341e+010	----	1.5341e+010
2	Left	0.150	118.03	1.2975e+010	118.03	1.2975e+010	171.15	5.4416e+009
	Middle	0.700	125.89	1.1067e+010	125.89	1.1067e+010	182.55	4.9494e+009
	Right	0.150	-249.46	3.1576e+009	-249.46	3.1576e+009	-361.71	2.8904e+009
	Span Avg	----	----	1.0167e+010	----	1.0167e+010	----	4.7144e+009
3	Left	0.150	-249.54	3.1572e+009	-249.54	3.1572e+009	-361.83	2.8903e+009
	Middle	0.700	121.67	1.2047e+010	121.67	1.2047e+010	176.43	5.271e+009
	Right	0.150	-249.54	3.1572e+009	-249.54	3.1572e+009	-361.83	2.8903e+009
	Span Avg	----	----	9.3802e+009	----	9.3802e+009	----	4.5568e+009
4	Left	0.150	-249.46	3.1576e+009	-249.46	3.1576e+009	-361.71	2.8904e+009
	Middle	0.700	125.89	1.1067e+010	125.89	1.1067e+010	182.55	4.9494e+009
	Right	0.150	118.03	1.2975e+010	118.03	1.2975e+010	171.15	5.4416e+009
	Span Avg	----	----	1.0167e+010	----	1.0167e+010	----	4.7144e+009
5	Left	1.000	0.00	1.5341e+010	0.00	1.5341e+010	0.00	1.5341e+010
	Span Avg	----	----	1.5341e+010	----	1.5341e+010	----	1.5341e+010

3.1.3. Strip Section Properties at Midspan

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

Span	Column/Band Strip			Middle Strip		
	I_g mm⁴	LDF	Ratio	I_g mm⁴	LDF	Ratio
1	1.07188e+010	0.950	1.360	1.60781e+009	0.050	0.477
2	1.07188e+010	0.925	1.324	1.60781e+009	0.075	0.716
3	1.07188e+010	0.900	1.288	1.60781e+009	0.100	0.954
4	1.07188e+010	0.925	1.324	1.60781e+009	0.075	0.716
5	1.07188e+010	0.950	1.360	1.60781e+009	0.050	0.477

3.2. Instantaneous Deflections

3.2.1. Extreme Instantaneous Frame Deflections and Corresponding Locations

Span	Direction	Value	Units	Dead	Live			Total	
					Sustained	Unsustained	Total	Sustained	Dead+Live
1	Down	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
	Up	Def	mm	-0.20	---	-0.17	-0.17	-0.20	-0.37
		Loc	m	0.000	---	0.000	0.000	0.000	0.000
2	Down	Def	mm	1.40	---	2.04	2.04	1.40	3.43
		Loc	m	3.052	---	3.127	3.127	3.052	3.127
	Up	Def	mm	---	---	-0.01	-0.01	---	-0.01
		Loc	m	---	---	6.400	6.400	---	6.550
3	Down	Def	mm	1.66	---	2.93	2.93	1.66	4.59
		Loc	m	3.713	---	3.713	3.713	3.713	3.713
	Up	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
4	Down	Def	mm	1.40	---	2.04	2.04	1.40	3.43
		Loc	m	3.648	---	3.573	3.573	3.648	3.573
	Up	Def	mm	0.00	---	-0.01	-0.01	0.00	-0.01
		Loc	m	0.060	---	0.300	0.300	0.060	0.120
5	Down	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
	Up	Def	mm	-0.20	---	-0.17	-0.17	-0.20	-0.37
		Loc	m	0.400	---	0.400	0.400	0.400	0.400

3.2.2. Extreme Instantaneous Column Strip Deflections and Corresponding Locations

Span	Direction	Value	Units	Dead	Live			Total	
					Sustained	Unsustained	Total	Sustained	Dead+Live
1	Down	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
	Up	Def	mm	-0.20	---	-0.17	-0.17	-0.20	-0.37
		Loc	m	0.000	---	0.000	0.000	0.000	0.000
2	Down	Def	mm	1.70	---	2.58	2.58	1.70	4.28
		Loc	m	3.127	---	3.201	3.201	3.127	3.127
	Up	Def	mm	---	---	-0.01	-0.01	---	0.00
		Loc	m	---	---	6.400	6.400	---	6.625
3	Down	Def	mm	2.12	---	3.73	3.73	2.12	5.85
		Loc	m	3.713	---	3.713	3.713	3.713	3.713
	Up	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
4	Down	Def	mm	1.70	---	2.58	2.58	1.70	4.28
		Loc	m	3.573	---	3.499	3.499	3.573	3.573

Span	Direction	Value	Units	Dead	Live Sustained	Live Unsustained	Live Total	Total Sustained	Total Dead+Live
	Up	Def	mm	---	---	-0.01	-0.01	---	0.00
		Loc	m	---	---	0.240	0.240	---	0.120
5	Down	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
	Up	Def	mm	-0.20	---	-0.17	-0.17	-0.20	-0.37
		Loc	m	0.400	---	0.400	0.400	0.400	0.400

3.2.3. Extreme Instantaneous Middle Strip Deflections and Corresponding Locations

Span	Direction	Value	Units	Dead	Live Sustained	Live Unsustained	Live Total	Total Sustained	Total Dead+Live
1	Down	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
	Up	Def	mm	-0.20	---	-0.17	-0.17	-0.20	-0.37
		Loc	m	0.000	---	0.000	0.000	0.000	0.000
2	Down	Def	mm	1.13	---	1.56	1.56	1.13	2.69
		Loc	m	2.904	---	3.052	3.052	2.904	3.052
	Up	Def	mm	0.00	---	-0.02	-0.02	0.00	-0.01
		Loc	m	6.625	---	6.400	6.400	6.625	6.550
3	Down	Def	mm	1.58	---	2.80	2.80	1.58	4.38
		Loc	m	3.713	---	3.713	3.713	3.713	3.713
	Up	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
4	Down	Def	mm	1.13	---	1.56	1.56	1.13	2.69
		Loc	m	3.796	---	3.648	3.648	3.796	3.648
	Up	Def	mm	0.00	---	-0.02	-0.02	0.00	-0.01
		Loc	m	0.060	---	0.300	0.300	0.060	0.180
5	Down	Def	mm	---	---	---	---	---	---
		Loc	m	---	---	---	---	---	---
	Up	Def	mm	-0.20	---	-0.17	-0.17	-0.20	-0.37
		Loc	m	0.400	---	0.400	0.400	0.400	0.400

3.3. Long-term Deflections

3.3.1. Long-term Column Strip Deflection Factors

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

Time dependant factor for sustained loads = 2.000

Span	Zone	M_{+ve} $A_{s,top}$ mm²	M_{+ve} b mm	M_{+ve} d mm	M_{+ve} Rho' %	M_{+ve} Lambda	M_{-ve} $A_{s,bot}$ mm²	M_{-ve} b mm	M_{-ve} d mm	M_{-ve} Rho' %	M_{-ve} Lambda
1	Right	----	----	----	0.000	2.000	----	----	----	0.000	2.000
2	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
3	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
4	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
5	Left	----	----	----	0.000	2.000	----	----	----	0.000	2.000

3.3.2. Long-term Middle Strip Deflection Factors

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

Time dependant factor for sustained loads = 2.000

Span	Zone	M+ve $A_{s,top}$	b	d	Rho'	Lambda	M-ve $A_{s,bot}$	b	d	Rho'	Lambda
		mm^2	mm	mm	%		mm^2	mm	mm	%	
1	Right	----	----	----	0.000	2.000	----	----	----	0.000	2.000
2	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
3	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
4	Midspan	----	----	----	0.000	2.000	----	----	----	0.000	2.000
5	Left	----	----	----	0.000	2.000	----	----	----	0.000	2.000

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

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

Span	Direction	Value	Units	cs	cs+lu	cs+l	Total
1	Down	Def	mm	---	---	---	---
		Loc	m	---	---	---	---
	Up	Def	mm	-0.40	-0.57	-0.57	-0.77
		Loc	m	0.000	0.000	0.000	0.000
2	Down	Def	mm	3.40	5.99	5.99	7.69
		Loc	m	3.127	3.127	3.127	3.127
	Up	Def	mm	---	0.00	0.00	0.00
		Loc	m	---	6.625	6.625	6.625
3	Down	Def	mm	4.25	7.98	7.98	10.10
		Loc	m	3.713	3.713	3.713	3.713
	Up	Def	mm	---	---	---	---
		Loc	m	---	---	---	---
4	Down	Def	mm	3.40	5.99	5.99	7.69
		Loc	m	3.573	3.573	3.573	3.573
	Up	Def	mm	---	0.00	0.00	0.00
		Loc	m	---	0.060	0.060	0.060
5	Down	Def	mm	---	---	---	---
		Loc	m	---	---	---	---
	Up	Def	mm	-0.40	-0.57	-0.57	-0.77
		Loc	m	0.400	0.400	0.400	0.400

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

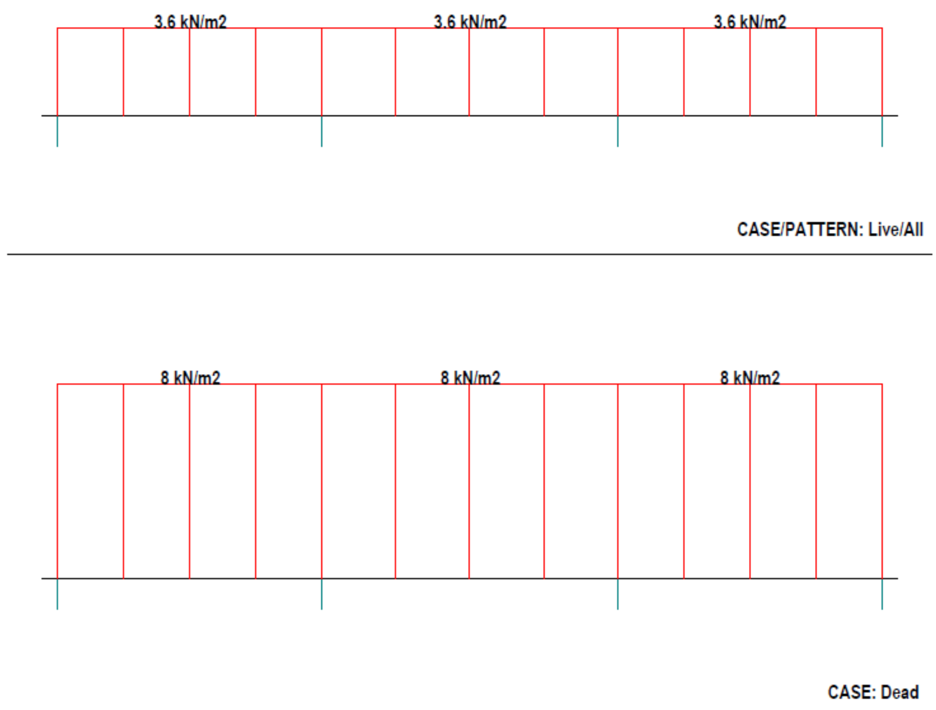
Notes:
Incremental deflections due to creep and shrinkage (cs) based on sustained load level values.
Incremental deflections after partitions are installed can be estimated by deflections due to:
- creep and shrinkage plus unsustained live load (cs+lu), if live load applied before partitions,
- creep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

Span	Direction	Value	Units	cs	cs+lu	cs+l	Total
1	Down	Def	mm	---	---	---	---
		Loc	m	---	---	---	---
	Up	Def	mm	-0.40	-0.57	-0.57	-0.77
		Loc	m	0.000	0.000	0.000	0.000

Span	Direction	Value	Units	cs	cs+lu	cs+l	Total
2	Down	Def	mm	2.26	3.83	3.83	4.96
		Loc	m	2.904	2.978	2.978	2.978
	Up	Def	mm	0.00	-0.01	-0.01	-0.01
		Loc	m	6.625	6.625	6.625	6.625
3	Down	Def	mm	3.16	5.97	5.97	7.55
		Loc	m	3.713	3.713	3.713	3.713
	Up	Def	mm	---	---	---	---
		Loc	m	---	---	---	---
4	Down	Def	mm	2.26	3.83	3.83	4.96
		Loc	m	3.796	3.722	3.722	3.722
	Up	Def	mm	0.00	-0.01	-0.01	0.00
		Loc	m	0.060	0.120	0.120	0.060
5	Down	Def	mm	---	---	---	---
		Loc	m	---	---	---	---
	Up	Def	mm	-0.40	-0.57	-0.57	-0.77
		Loc	m	0.400	0.400	0.400	0.400

4. Diagrams

4.1. Loads



spSlab v5.50. Licensed to: StructurePoint. License ID: 00000-0000000-4-25EF2-2C6B6

File: C:\CSA A23.3-14 Examples\Slab with Longitudinal Slab Bands.slb

Project: CAC 4th Edition - Section 5.7 - Example 3

Frame: Interior

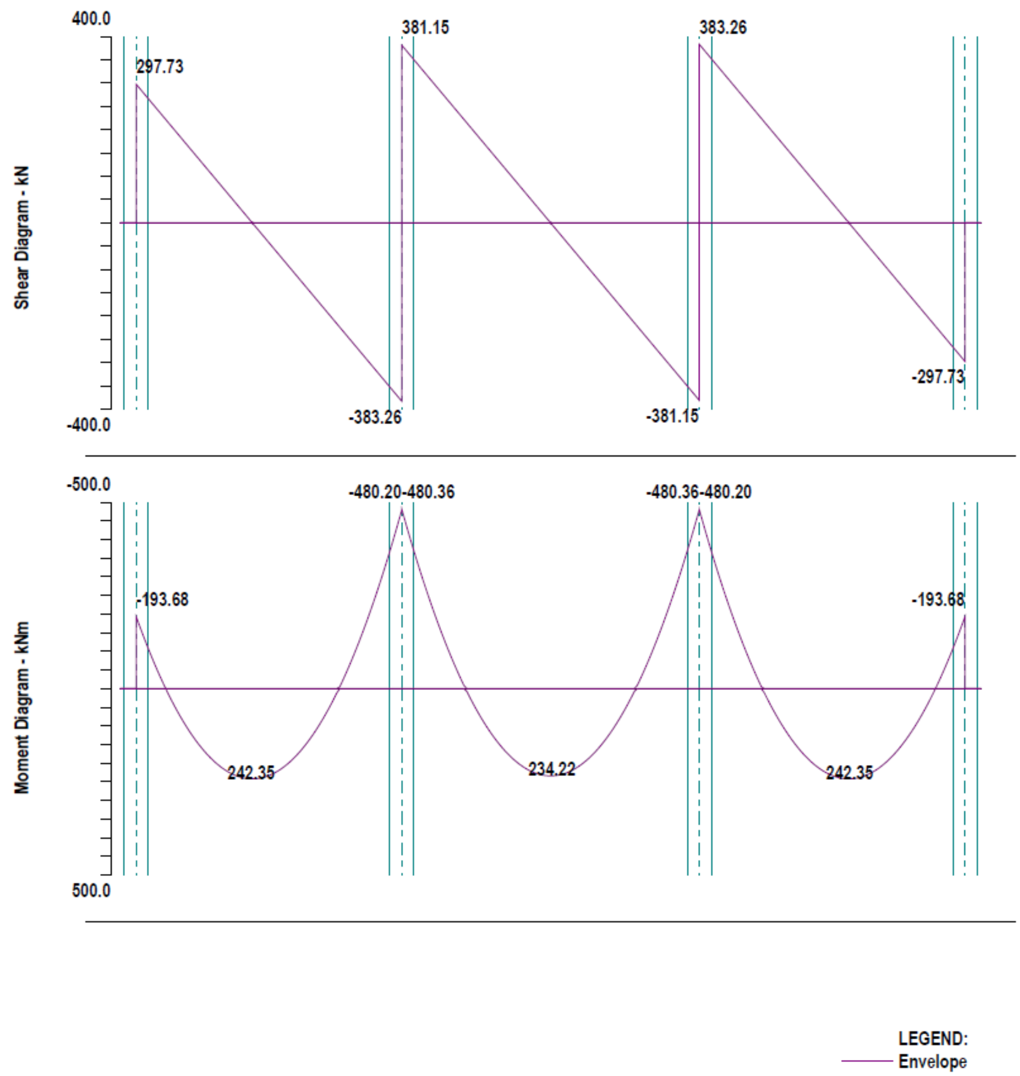
Engineer: SP

Code: CSA A23.3-14

Date: 01/13/20

Time: 08:24:44

4.2. Internal Forces



spSlab v5.50. Licensed to: StructurePoint. License ID: 00000-0000000-4-25EF2-2C6B6

File: C:\CSA A23.3-14 Examples\Slab with Longitudinal Slab Bands.slb

Project: CAC 4th Edition - Section 5.7 - Example 3

Frame: Interior

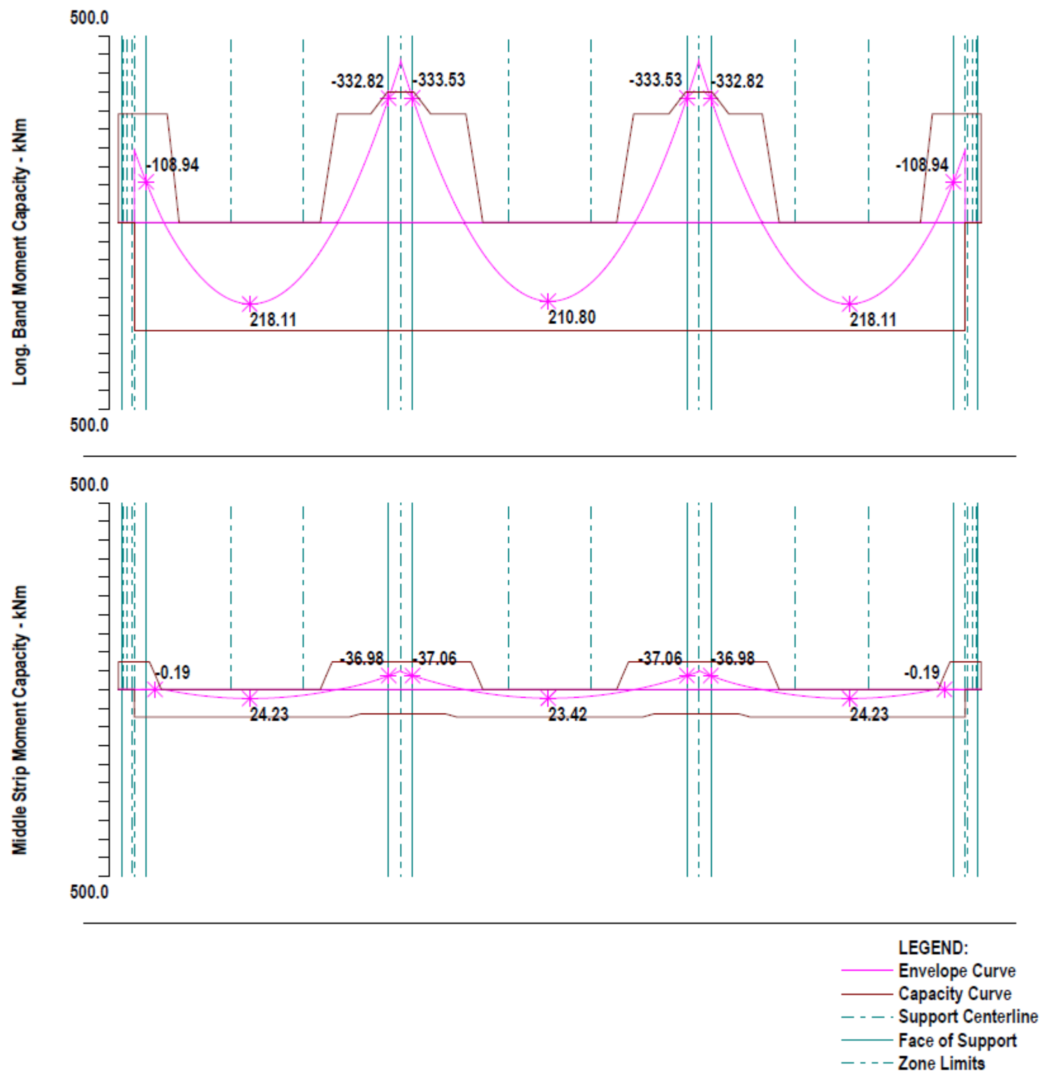
Engineer: SP

Code: CSA A23.3-14

Date: 01/13/20

Time: 08:24:44

4.3. Moment Capacity

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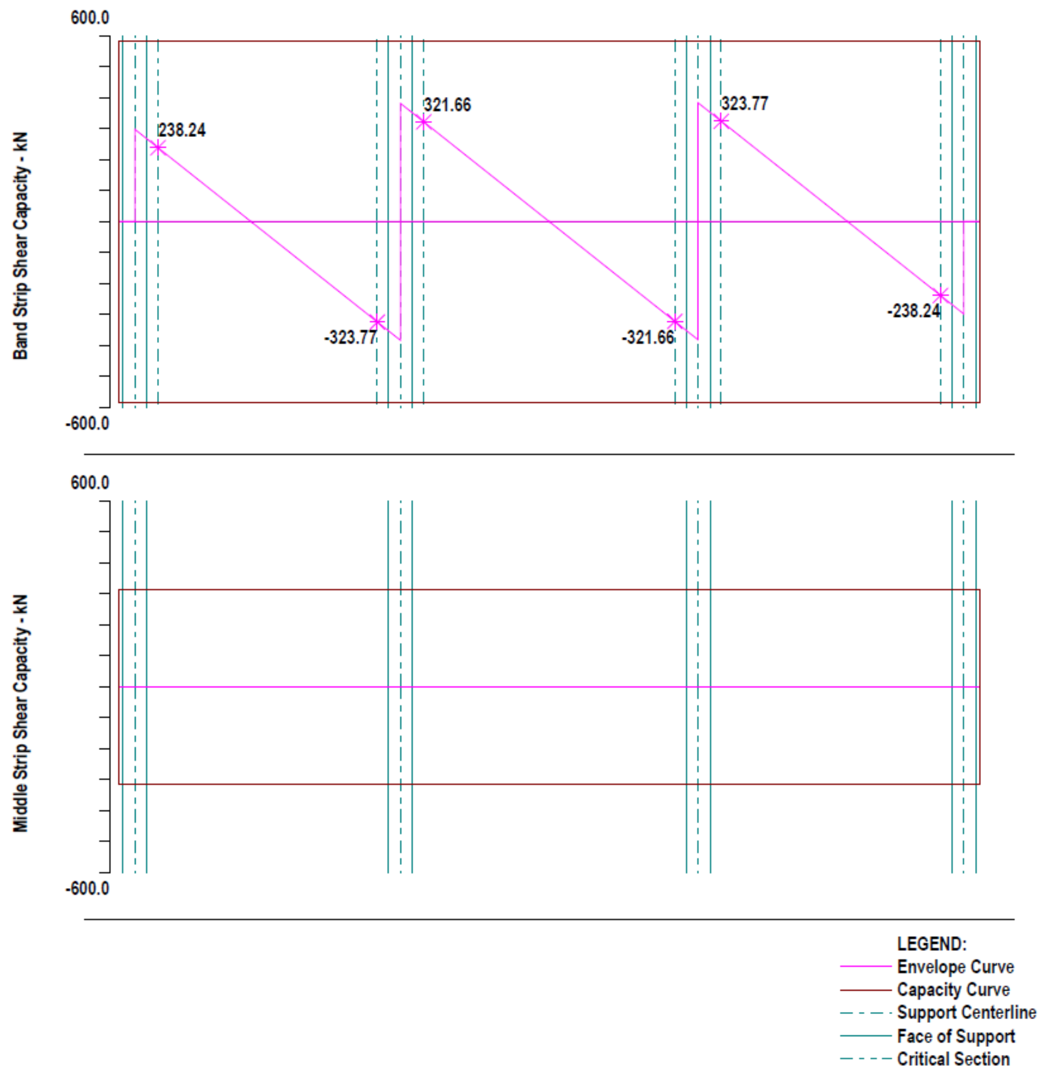
spSlab v5.50. Licensed to: StructurePoint. License ID: 00000-0000000-4-25EF2-2C6B6

File: C:\CSA A23.3-14 Examples\Slab with Longitudinal Slab Bands.slb
Project: CAC 4th Edition - Section 5.7 - Example 3
Frame: Interior
Engineer: SP
Code: CSA A23.3-14
Date: 01/13/20
Time: 08:24:44
```

4.4. Shear Capacity



spSlab v5.50. Licensed to: StructurePoint. License ID: 00000-0000000-4-25EF2-2C6B6

File: C:\CSA A23.3-14 Examples\Slab with Longitudinal Slab Bands.slb

Project: CAC 4th Edition - Section 5.7 - Example 3

Frame: Interior

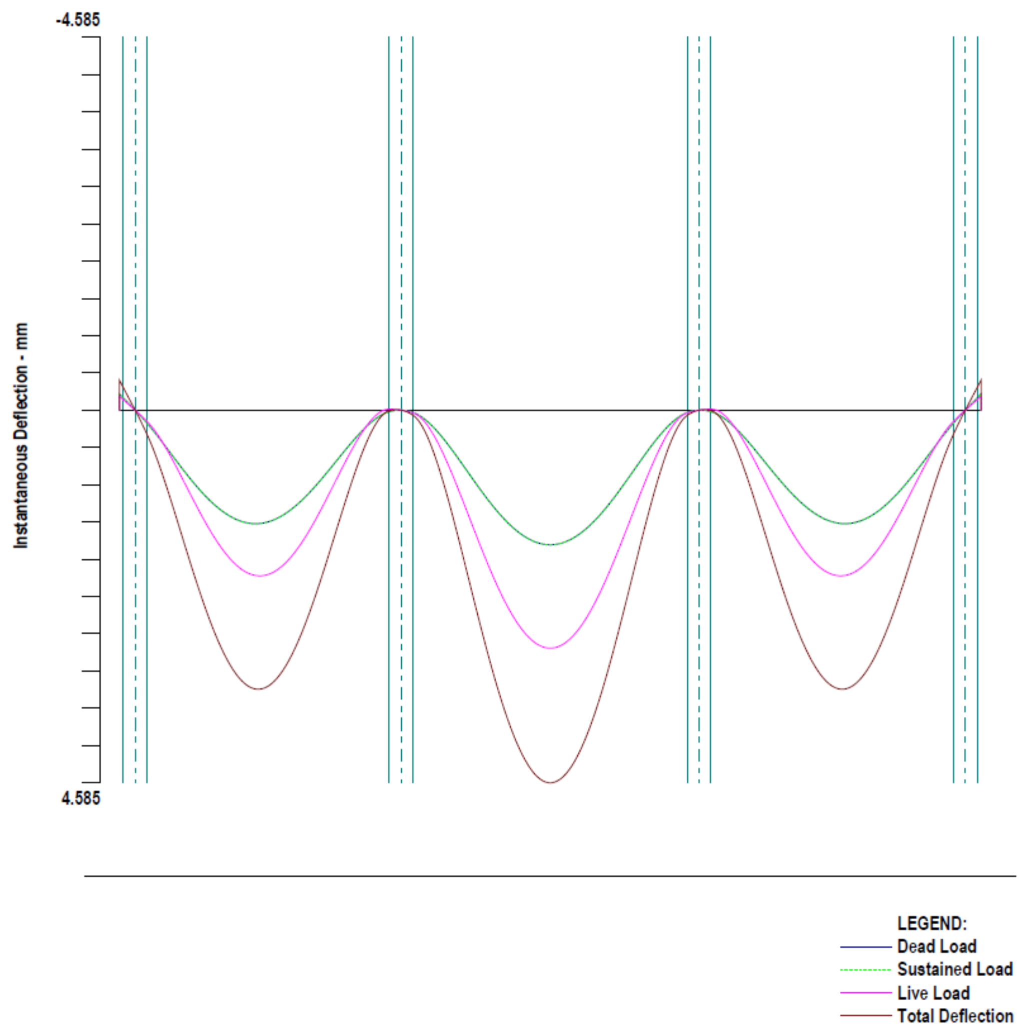
Engineer: SP

Code: CSA A23.3-14

Date: 01/13/20

Time: 08:24:44

4.5. Deflection



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File: C:\CSA A23.3-14 Examples\Slab with Longitudinal Slab Bands.slb

Project: CAC 4th Edition - Section 5.7 - Example 3

Frame: Interior

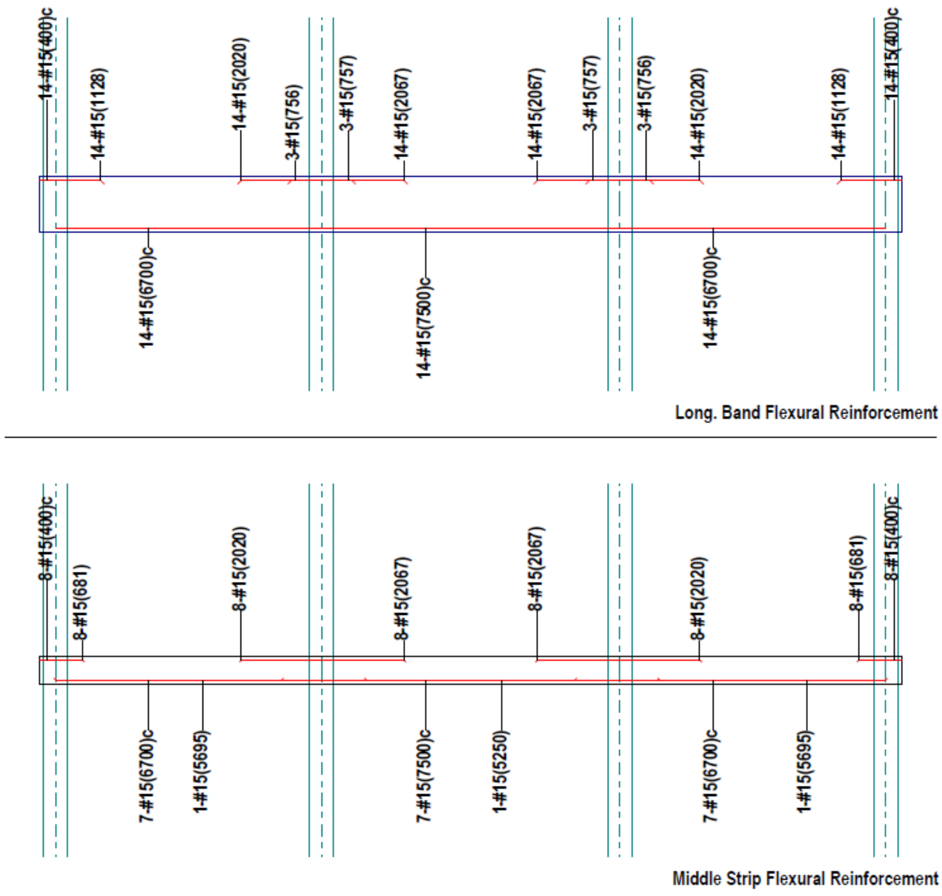
Engineer: SP

Code: CSA A23.3-14

Date: 01/13/20

Time: 08:24:44

4.6. Reinforcement

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File: C:\CSA A23.3-14 Examples\Slab with Longitudinal Slab Bands.slb
Project: CAC 4th Edition - Section 5.7 - Example 3
Frame: Interior
Engineer: SP
Code: CSA A23.3-14
Date: 01/13/20
Time: 08:24:44
```

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

Table 6 - Comparison of Moments obtained from Hand (EFM) and spSlab Solution			
		Hand (EFM)	spSlab
Exterior Span			
Band Strip	Exterior Negative*	109.0	108.9
	Positive	218.3	218.1
	Interior Negative*	332.5	332.8
Middle Strip	Exterior Negative*	0.0	0.0
	Positive	24.2	24.2
	Interior Negative*	37.0	37.0
Interior Span			
Band Strip	Interior Negative*	333.3	333.5
	Positive	211.1	210.8
Middle Strip	Interior Negative*	37.0	37.1
	Positive	23.5	23.4
* negative moments are taken at the faces of supports			

Table 7 - Comparison of Reinforcement Results with Hand and spSlab Solution							
Span Location		Reinforcement Provided for Flexure		Additional Reinforcement Provided for Unbalanced Moment Transfer*		Total Reinforcement Provided	
		Hand	spSlab	Hand	spSlab	Hand	spSlab
Exterior Span							
Band Strip	Exterior Negative	14-15M	14-15M	---	---	14-15M	14-15M
	Positive	12-15M	14-15M	n/a	n/a	14-15M	14-15M
	Interior Negative	17-15M	17-15M	---	---	17-15M	17-15M
Middle Strip	Exterior Negative	8-15M	8-15M	n/a	n/a	8-15M	8-15M
	Positive	8-15M	8-15M	n/a	n/a	8-15M	8-15M
	Interior Negative	8-15M	8-15M	n/a	n/a	8-15M	8-15M
Interior Span							
Band Strip	Negative	17-15M	17-15M	---	---	17-15M	17-15M
	Positive	14-15M	14-15M	n/a	n/a	14-15M	14-15M
Middle Strip	Negative	8-15M	8-15M	---	---	8-15M	8-15M
	Positive	8-15M	8-15M	n/a	n/a	8-15M	8-15M
* In the EFM, the unbalanced moment (M_{sc}, M_{unb}) at the support centerline is used to determine the value of the additional reinforcement as compared with DDM using the moments at the face of support.							

Table 8 - Comparison of One-Way (Beam Action) Shear Check Results Using Hand and spSlab Solution				
Span	**V_u , kN**		**ϕV_c, kN**	
	Hand	**spSlab**	**Hand**	**spSlab**
Exterior	323.7	323.7	584.2	584.2
Interior	321.7	321.7	584.2	584.2

Table 9 - Comparison of Two-Way (Punching) Shear Check Results Using Hand and spSlab Solution

Support	**b_1, mm**		**b_2, mm**		**b_o, mm**		**A_c, mm^2**		**V_u, kN**		**v_u, N/mm^2**	
	Hand	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**
Exterior	858.5	858.5	717.0	717.0	2434	2434	7.72×10^5	7.72×10^5	292.0	292.7	0.378	0.379
Interior	917.0	917.0	717.0	717.0	3268	3268	1.04×10^6	1.04×10^6	758.0	754.3	0.732	0.728

Support	**c_{AB}, mm**		**J_c, x 10^9 mm^4**		**γ_v**		**M_{unb}, kN.m**		**v_u, MPa**		**ϕv_c, MPa**	
	Hand	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**	**Hand**	**spSlab**
Exterior	302.8	302.8	67.53	67.53	0.422	0.422	148.24	148.12	0.659	0.659	1.219	1.219
Interior	458.5	458.5	141.17	141.17	0.430	0.430	0.30	0.16	0.732	0.728	1.219	1.219

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

A slab system can be analyzed and designed by any procedure satisfying equilibrium and geometric compatibility. Three established methods are widely used. The requirements for two of them are described in detail in ***CSA A23.3-14 Clasues (13.8 and 13.9)*** for regular two-way slab systems. ***CSA A23.3-14 (13.5.1)***

Direct Design Method (DDM) is an approximate method and is applicable to flat plate concrete floor systems that meet the stringent requirements of ***CSA A23.3-14 (13.9.1)***. In many projects, however, these requirements limit the usability of the Direct Design Method significantly.

The Elastic Frame Method (EFM) has less stringent limitations compared to DDM. It requires more accurate analysis methods that, depending on the size and geometry can prove to be long, tedious, and time-consuming.

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

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

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

Applicable CSA A23.3-14 Provision	Limitations/Applicability	Concrete Slab Analysis Method		
		DDM (Hand)	EFM (Hand//spSlab)	FEM (spMats)
13.8.1.1 13.9.1.1	Panels shall be rectangular, with ratio of longer to shorter panel dimensions, measured center-to-center supports, not exceed 2.	☑	☑	
13.8.1.1 13.9.1.1	For a panel with beams between supports on all sides, slab-to-beam stiffness ratio shall be satisfied for beams in the two perpendicular directions.	☑	☑	
13.8.1.1 13.9.1.1	Column offset shall not exceed 20% of the span in direction of offset from either axis between centerlines of successive columns	☑	☑	
13.8.1.1 13.9.1.1	The reinforcement is placed in an orthogonal grid.	☑	☑	
13.9.1.2	Minimum of three continuous spans in each direction	☑		
13.9.1.3	Successive span lengths measured center-to-center of supports in each direction shall not differ by more than one-third the longer span	☑		
13.9.1.4	All loads shall be due to gravity only	☑		
13.9.1.4	All loads shall be uniformly distributed over an entire panel (q_f)	☑		
13.9.1.4	Unfactored live load shall not exceed two times the unfactored dead load	☑		
13.10.6	Structural integrity steel detailing	☑	☑	☑
13.10.10	Openings in slab systems	☑	☑	☑
8.2	Concentrated loads	Not permitted	☑	☑
13.8.4.1	Live load arrangement (Load Patterning)	Not required	Required	Engineering judgment required based on modeling technique
13.10.2*	Reinforcement for unbalanced slab moment transfer to column (M_{sc})	Moments @ support face	Moments @ support centerline	Engineering judgment required based on modeling technique
13.8.2	Irregularities (i.e. variable thickness, non-prismatic, partial bands, mixed systems, support arrangement, etc.)	Not permitted	Engineering judgment required	Engineering judgment required
Complexity		Low	Average	Complex to very complex
Design time/costs		Fast	Limited	Unpredictable/Costly
Design Economy		Conservative (see detailed comparison with spSlab output)	Somewhat conservative	Unknown - highly dependent on modeling assumptions: 1. Linear vs. non-linear 2. Isotropic vs non-isotropic 3. Plate element choice 4. Mesh size and aspect ratio 5. Design & detailing features
General (Drawbacks)		Very limited applications	Limited geometry	Limited guidance non-standard application (user dependent). Required significant engineering judgment
General (Advantages)		Very limited analysis is required	Detailed analysis is required or via software (e.g. spSlab)	Unlimited applicability to handle complex situations permissible by the features of the software used (e.g. spMats)

* The unbalanced slab moment transferred to the column M_{sc} (M_{unb}) is the difference in slab moment on either side of a column at a specific joint. In DDM only moments at the face of the support are calculated and are also used to obtain M_{sc} (M_{unb}). In EFM where a frame analysis is used, moments at the column center line are used to obtain M_{sc} (M_{unb}).