

APPENDIX A

PROTOTYPE BUILDING DESIGN

The design of test specimens was based on a five story prototype flat-plate building shown in figure .A-1. It has four bays of 20 ft. each in the long direction direction and three bays in the short direction. The story height is typically 10 ft. except for the first story which is 21.5 ft. high. The design of this building is based on Building Code Requirements for Reinforced Concrete (ACI 318-47) and it represents a typical gavity load design for flat-plate buildings constructed during forties and fifties in the Eastern United States.

MATERIAL PROPERTIES

Concrete compressive strength, f_c' : 3000 psi

Reinforcing Steel, Grade 40, f_y : 40,000 psi

DESIGN LOAD

Assume a slab thickness 9 inches and the unit weight of concrete as 150 lbs/ft³

Self weight of slab: 112.5 lbs/ft²

Partitions: 20.0 lbs/ft²

1" granolithic finish: 13.0 lbs/ft²

Total dead load: 145.5 lbs/ft²

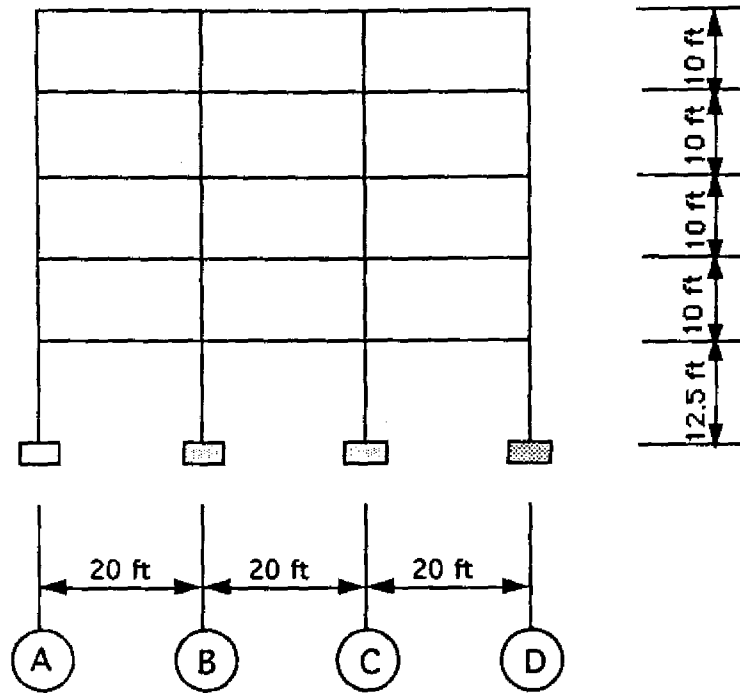
Live load: 50.0 lbs/ft²

Total dead load + live load: 195.5 lbs/ft²

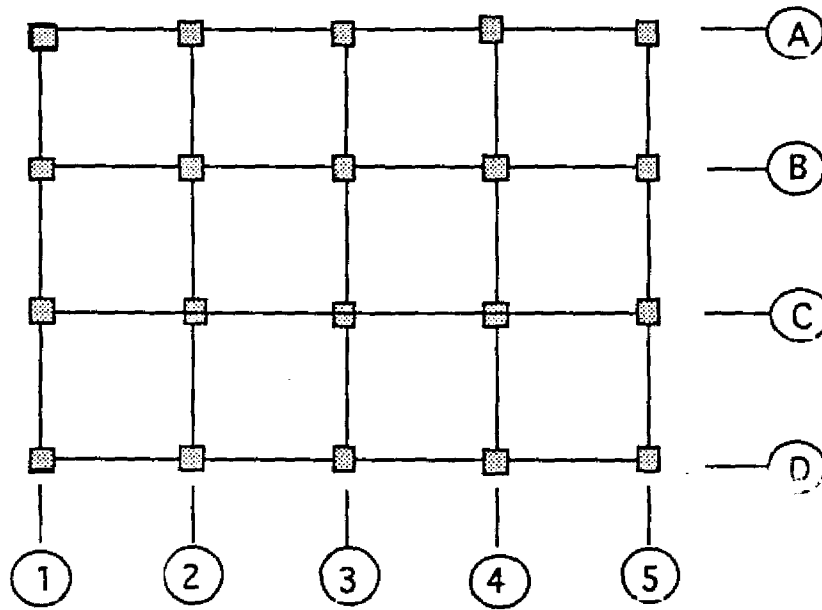
MINIMUM SLAB THICKNESS

$$\text{Slab thickness, } h \geq \frac{L}{36}$$

For a span of 20 ft., the minimum slab thickness requirement is 6.67 inches. Hence, the assumed slab thickness of 9 inches is acceptable.



Elevation of Prototype Building



Plan of Prototype Building

FIGURE A-1 Prototype Building

SHEAR STRENGTH CALCULATIONS

The critical section for punching at interior and exterior connections is located at slab thickness minus 1.5 inches away from the column face as shown in figure A-2.

The shear stress at the critical section is calculated by

$$v_c = \frac{V}{b_o j d}$$

where V is the total shear at the critical section, b_o is the critical section perimeter, j is assumed as 0.88, and d is the slab effective depth taken as $h - 1.5''$.

The calculated shear stress should not exceed: (1) $0.03f_c'$ when at least 50 percent of the total negative reinforcement in the column strip passes directly over the column capital, (2) $0.025f_c'$ when 25 percent or less of the total negative reinforcement in the column strip passes directly over the column capital. Since there are no column capitals in this case, the column capital region is assumed equal to the width of the column plus twice the slab thickness. Furthermore, the allowable shear stress of $0.03f_c'$ is assumed to apply.

For interior connections:

$$\begin{aligned} V &= 20 \times 20 \times 195.5 / 1000 = 78.2 \text{ kips} \\ b_o &= 4(20 + 2 \times 7.5) = 140 \text{ in.} \\ \text{where } d &= 9.0 - 1.5 = 7.5 \text{ in.} \\ \text{Shear stress} &= 84.6 \text{ psi} < 0.03f_c' \quad \text{O.K.} \end{aligned}$$

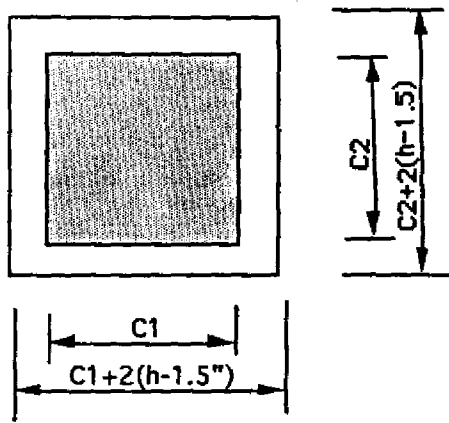
For exterior connections:

$$\begin{aligned} V &= 20 \times 10 \times 195.5 / 1000 = 19.55 \text{ kips} \\ b_o &= 3 \times 20 + 4 \times 7.5 = 90 \text{ in.} \\ \text{Shear stress} &= 65.8 < 0.03f_c' \quad \text{O.K.} \end{aligned}$$

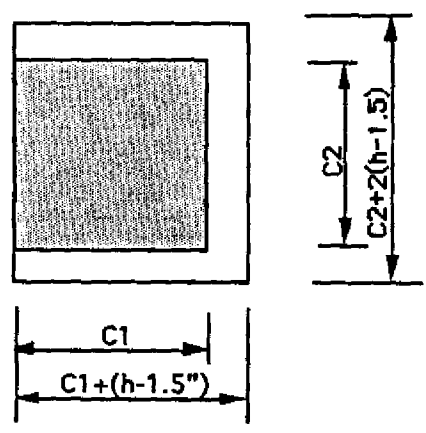
For corner connections:

$$\begin{aligned} V &= 10 \times 10 \times 195.5 / 1000 = 19.55 \text{ kips} \\ b_o &= 2 \times 20 + 2 \times 7.5 = 55 \text{ in.} \\ \text{Shear stress} &= 53.85 \text{ psi} < 0.03f_c' \quad \text{O.K.} \end{aligned}$$

The slab thickness of 9 inches is therefore appropriate for shear resistance.



Critical Section at a Interior Connection



Critical Section at a Exterior Connection

FIGURE A-2 Critical Section at Interior and Exterior Connections

FLEXURAL REINFORCEMENT

Subject to the following constraints, the code allowed the use of moment coefficients.

- (1) Slab must be rectangular and monolithic with columns,
- (2) there should be three or more rows of panels in each direction,
- (3) maximum ratio of length to width of panel=1.33,
- (4) dimensions of adjacent panels should not vary by more than 20% of shorter span, and
- (5) slabs may be solid or ribbed.

The prototype building satisfied these criteria and hence the moment based on the code suggested coefficients is calculated as

$$M_o = 0.09WL \left[1 - \frac{2C}{3L} \right]^2$$

where M_o is sum of positive and average negative bending moment, L is span length center to center of column in direction being considered, C effective column diameter, and W is the total uniform load on the panel.

$$\begin{aligned} M_o &= 0.09 \times 195.5 \times 20^2 \times 20 \left[1 - \frac{2 \times 38}{3 \times 240} \right]^2 \\ &= 112.6 \text{ kip-ft.} \end{aligned}$$

For interior panel:

$$\text{Column strip negative moment} = 0.46M_o = 51.8 \text{ kip-ft.}$$

$$\text{Column strip positive moment} = 0.22M_o = 24.8 \text{ kip-ft.}$$

$$\text{Middle strip negative moment} = 0.16M_o = 18.0 \text{ kip-ft.}$$

$$\text{Middle strip positive moment} = 0.16M_o = 18.0 \text{ kip-ft.}$$

For exterior panel:

$$\text{Column strip exterior negative moment} = 0.41M_o = 46.2 \text{ kip-ft.}$$

$$\text{Column strip positive moment} = 0.28M_o = 31.5 \text{ kip-ft.}$$

$$\text{Column strip interior negative moment} = 0.5M_o = 56.3 \text{ kip-ft.}$$

$$\text{Middle strip exterior negative moment} = 0.10M_o = 11.3 \text{ kip-ft.}$$

Middle strip positive moment = $0.2M_o = 22.5$ kip-ft.

Middle strip interior negative moment = $0.176M_o = 19.8$ kip-ft.

Using the Working Stress Design Method, the slab flexural reinforcement is determined by

$$A_s = \frac{12M_o}{f_s d}$$

where the allowable steel stress f_s is 20 ksi and the effective slab depth $d = 75$ inches. The required areas of steel for column and middle strips in exterior and interior panels are given in table A-1.

TABLE A-1 SLAB REINFORCEMENT AREAS

Panel		Exterior Panel			Interior Panel		
Position		Edge	Mid-span	Edge	Edge	Mid-span	Edge
Column Strip	Moment K-ft.	46.6	31.5	56.3	52.0	24.8	52.0
	A_s (in. ²)	4.2	2.9	5.1	4.7	2.3	4.7
	Straight Bent-Up	12-#5 8-#4	10-#4 8-#4	12-#4 14-#4	6-#7 14-#4	8-#4 6-#4	12-#4 12-#4
Middle Strip	Moment K-ft.	11.3	22.6	20.0	18.4	18.4	18.4
	A_s (in. ²)	1.0	2.1	1.9	1.7	1.7	1.7
	Straight Bent-Up	1-#3 6-#4	7-#4 6-#4	1-#4 11-#4	1-#4 11-#4	5-#4 5-#4	1-#4 10-#4

The arrangement of the slab reinforcement is shown in figure A-3. For the allowable shear stress of $0.03f'_c$, 50% of the total negative reinforcement in column strip must be placed within the column capital region, and the maximum spacing for slab rebars is limited to three times the slab thickness. The positive slab reinforcement extends six inches into the exterior support and three inches to a maximum of $0.125L$ at the interior support. Differences in the distribution of bent-up and straight reinforcement can be seen in figure A-3.

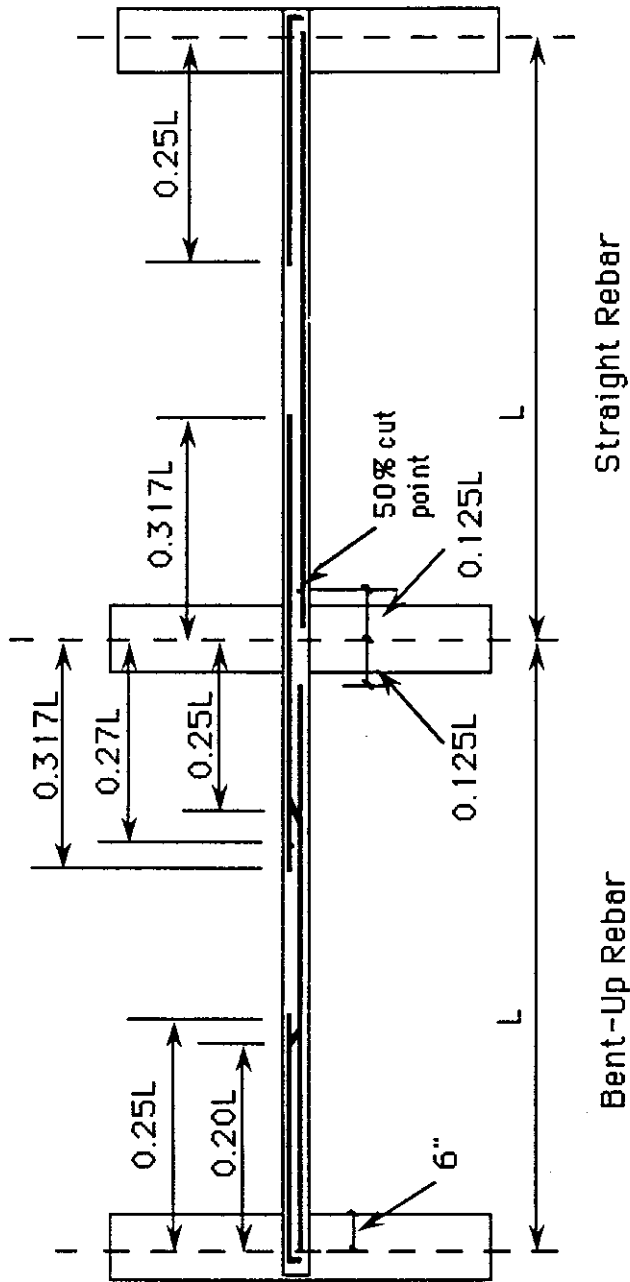


FIGURE A-3 Straight and Bent-up Reinforcing Detail

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