$$= 4.48 \text{ ft}$$

$$w_u = 157 + 235/DW$$

$$= 157 + 235/4.48$$

$$= 209 \text{ psf}$$

Find shear is adequate at 4.14 ft and all points further into span. Use 4 - 7/16" dia., 270 ksi, low relaxation strands.

3.4 Continuity

Hollow core slabs are normally designed as part of a simple span system. However, continuity over supports can be achieved by placing reinforcing steel in the grouted keyways, in a composite structural topping, or by concreting bars into cores. Within limits, the result will be better control of superimposed load deflections and a lower requirement for positive moment capacity.

With reinforcing steel in either a composite topping or in cores, elastic moments with allowance for negative moment redistribution determine the amount of reinforcing required. Because of the relative efficiencies of positive prestressing steel and negative mild reinforcing, it is difficult to economically justify a continuous system design.

When reinforcing is required at supports for reasons such as structural integrity ties or diaphragm connections, the reinforcing ratios are generally quite low, and therefore, develop little moment capacity. While this reinforcing may be considered in calculating service load deflections, it is recommended that full simple span positive moment capacity be provided for strength design unless moment-curvature relationships existing at the supports at ultimate loads are known.

One situation that seems reasonable for considering a reduction in the positive moment requirements is where the slabs are required to have a fire rating developed using the rational design procedure. In this case, a limit analysis approach would be reasonable.

The negative moment reinforcing, which is unaffected by fire loads, can develop full yield moment potential and effectively provide a plastic hinge at the support. As a result, the positive moment at midspan may be correspondingly reduced. A detailed discussion of this is presented in Section 6.3.3.

3.5 Cantilevers

Cantilever design in hollow core slabs differs from design with conventional precast members because of the production procedures used for hollow core slabs. Guidelines noted here are conservative and may be exceeded depending on the specific product used.

Because long line beds are used for the production of hollow core slabs, top prestressing strands may be economical only when full bed capacity is utilized. Even then, substantial amounts of prestressing strand may be used inefficiently because of debonding requirements. The economics of using top strand must, therefore, be determined by the local producer.

When top strands are used, the length of the cantilever is usually not sufficient to fully develop a strand. A reduced value for f_{ps} is required and the design procedures given in Section 2.6 should be used. In dry cast systems, the bond of top strands may be less than desired so a further reduction in f_{ps} is required. This reduction may be substantial and each producer should be consulted on top strand bond performance.

When top strands are not economical, non-prestressed reinforcement may be placed in the cores or directly in the unit in the case of a wet cast product. This is generally done while the slab concrete is still plastic so bond of the fill concrete with the slab may be achieved. The reinforcement is selected based on conventional design with due consideration given to bar development length.

With either top strands or reinforcing bars, it may be necessary to debond portions of the bottom prestressing strand in the cantilever zone to help minimize the top tension under service loads. Not all producers have the ability to debond bottom strands which could potentially limit cantilever length or load capacity.

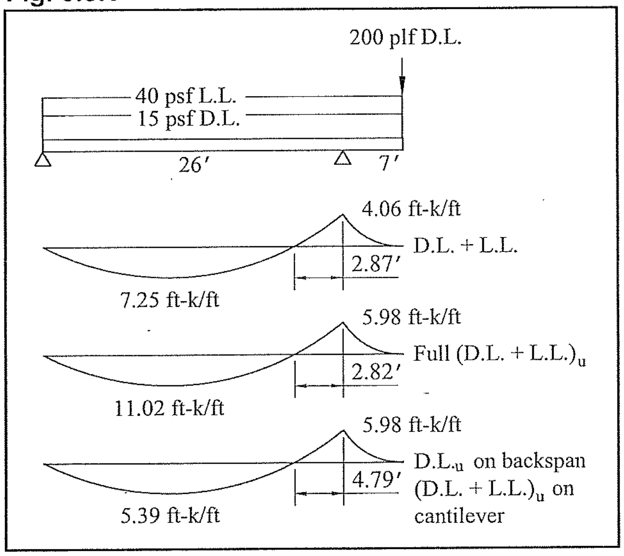
It is desirable to limit service level tensions in cantilevers so that uncracked section properties may be used to more accurately predict deflections. The tensile stress limit may vary for different systems used. For example, the practice with some dry cast systems is to limit tensile stresses to 100 psi (0.7 MPa). In other dry cast systems and in wet cast systems, the limit may be raised to $6\sqrt{f'_c}$. The tension limit will basically be a function of a producer's past experience.

As a rule of thumb, cantilever lengths falling in the range of 6 to 12 times the slab thickness will be workable depending on the superimposed load and individual producer's capabilities.

Example 3.5.1 Cantilever Design

Using the generic hollow core slab section defined in Section 1.7, design for the following conditions shown in Figure 3.5.1.

Fig. 3.5.1



Solution:

From the load table in Figure 1.7.1, select 4 - 3/8" dia., 270 ksi strands as the primary reinforcement. Try 2 - 3/8" dia., 270 ksi, low relaxation strands as cantilever reinforcement. Assume 15% losses and 70% initial stress.

Check stresses at cantilever:

Bottom strands:

$$f_{top} = 0.7(0.85)(4)(23)\left(\frac{1}{154} - \frac{2.89 \times 4.11}{1224.5}\right)$$

$$= -0.176 \text{ ksi (tension)}$$

Top strands:

$$f_{top} = 0.7(0.85)(2)(23)\left(\frac{1}{154} - \frac{3.11 \times 4.11}{1224.5}\right)$$

$$= +0.463 \text{ ksi}$$

Applied moment:

$$f_{top} = -\frac{4.06(3)(12)(4.11)}{1224.5}$$

$$= -0.491 \text{ ksi (tension)}$$

Net tension with fully bonded bottom strands:

$$f_t = -0.176 + 0.463 - 0.491$$

$$= -0.204 \text{ ksi}$$

Allow $6\sqrt{5000} = 0.424$ ksi OK

Note that some of the bottom strands could have been debonded for the length of the cantilever if top tensile stresses had exceeded a desirable level.

Stresses in backspan:

Because the backspan is long in this example, stresses will not be critical in the backspan. When the backspan is short relative to the cantilever length, stresses may require a check in the backspan to determine the length of bonding of the top strands.

Ultimate Strength

At the cantilever, strain compatibility will generally show that the bottom strands may be ignored in determining the nominal moment capacity. When the bottom prestress is very heavy or the bottom strands are high in the slab, a strain compatibility analysis should be performed considering both strand layers.

For this example, assume the bottom strands may be ignored.

$$f_{ps} = 270\left[1 - \left(\frac{0.28}{0.8}\right)\left(\frac{2(0.085)(270)}{(36)(7)(5)}\right)\right]$$

$$= 265 \text{ ksi}$$

$$a = \frac{2(0.085)(265)}{(0.85)(5)(36)} = 0.294 \text{ in}$$

$$\phi M_n = \frac{0.9}{12}(2)(0.085)(265)\left(7 - \frac{0.294}{2}\right)$$

$$= 23.15 \text{ ft-k/slab}$$

$$M_u = 3(5.98) = 17.94 \text{ ft-k/slab}$$

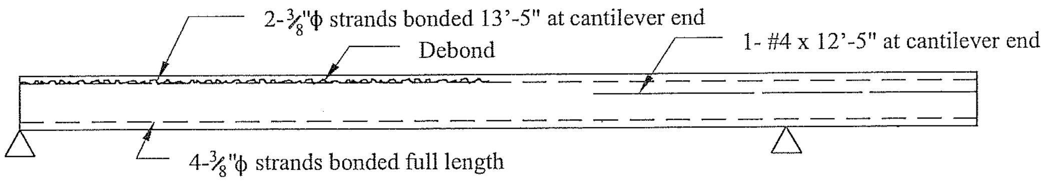
$$M_{cr} = \frac{1224.5}{4.11}\left(0.463 - 0.176 + \frac{7.5\sqrt{5000}}{1000}\right)\frac{1}{12}$$

$$= 20.29 \text{ ft-k/slab}$$

$$\frac{\phi M_n}{M_{cr}} = \frac{23.15}{20.29} = 1.14 < 1.2 \quad \text{NG}$$

Add 1 - #4 bar top per slab.

Design summary



Check length of top strand to be bonded:

$l_{available} = 7(12) = 84$ in

$\ell_d = (f_{ps} - 2/3 f_{se}) d_b$

$= (265 - 2(0.7)(0.85)(270)/3)(0.375)$

$= 59.2 \text{ in} < 84 \text{ in}$

Therefore, the strand is fully effective in the cantilever. The moment capacity would have to be recalculated by the procedures of Section 2.6 if the development length were found to be greater than the length available.

Bond of the top strands in the backspan must be long enough to develop the f_{ps} required in the cantilever design. The top strands should also be bonded for a distance of one transfer length (50 diameters) past the inflection point under the worst load condition. For this example a bonded length of 77 in. would be required.

Alternate Design

Provide mild reinforcement in lieu of top prestressing strands

Try 2 - #5 Gr 60 bars at d = 7″

$$a = \frac{2(0.31)(60)}{(0.85)(5)(36)} = 0.243''$$

$$\phi M_n = \frac{0.9}{12}(2)(0.31)(60)\left(7 - \frac{0.243}{2}\right)$$

$$= 19.19 \text{ ft-k/slab}$$

$$= 6.4 \text{ ft-k/ft} > 5.98 \quad \text{OK}$$

Top stress = -0.176 - 0.491

= -0.667 ksi with fully bonded bottom strands

Note that a cracked section must be considered in calculating cantilever deflections because the top stress exceeds a tension of $6\sqrt{f'_c}$.

3.6 Horizontal Joints

Figure 3.6.1 depicts three conditions typically used in a multistory wall bearing building where hollow core slabs are used in a platform detail. Several expressions [27-31] have been proposed to describe the transfer of axial load through this horizontal joint.

With hollow core slabs used for floors, the most efficient detail is to build the slab ends into the wall. Depending on the butt joint size, the strength of the joint for transfer of vertical loads can be enhanced with the addition of grout in the butt joint, Fig. 3.6.1(b), and in both the joint and cores, Fig. 3.6.1(c). Grout fill in the cores increases the net slab width and provides confinement for a grout column.

The strength of the joint for vertical load transfer can be predicted using Eq. 3.6.1 for an ungrouted joint, Fig. 3.6.1(a). For a grouted joint, Fig. 3.6.1(b) or (c), the greater of Eq. 3.6.1 and Eq. 3.6.2 can be used. Both grouted and ungrouted joints can have the slab cores either filled or not filled. Both equations include a capacity reduction term for load eccentric from the centerline of the joint. With single story walls braced at the top and bottom, this eccentricity will be negligible.

$$\phi P_n = \phi 0.85 A_e f'_c R_e \qquad \text{(Eq. 3.6.1)}$$

$$\phi P_n = \phi t_g \ell f_u C R_e / k \qquad \text{(Eq. 3.6.2)}$$

where

P_n = nominal strength of the joint

A_e = effective bearing area of slab in joint = $2wb_w$

w = bearing strip width

b_w = net web width of slab when cores are not filled

= unit width as solid slab when cores are filled

f'_c = design compressive strength of slab concrete or grout whichever is less

t_g = grout column thickness