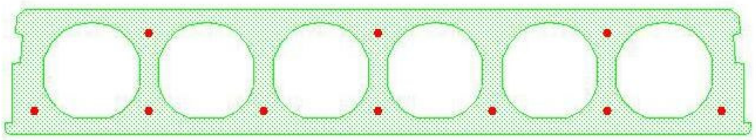


Hollow-Core Slabs

A hollow-core slab is a precast prestressed concrete member with continuous voids provided to reduce weight, costs and for electrical and mechanical runs. Primarily used as floor or roof deck systems, hollow-core slabs are also have applications as wall panels, sound barriers, spandrel members and bridge deck units.

Advantages of Hollow-Core Slabs

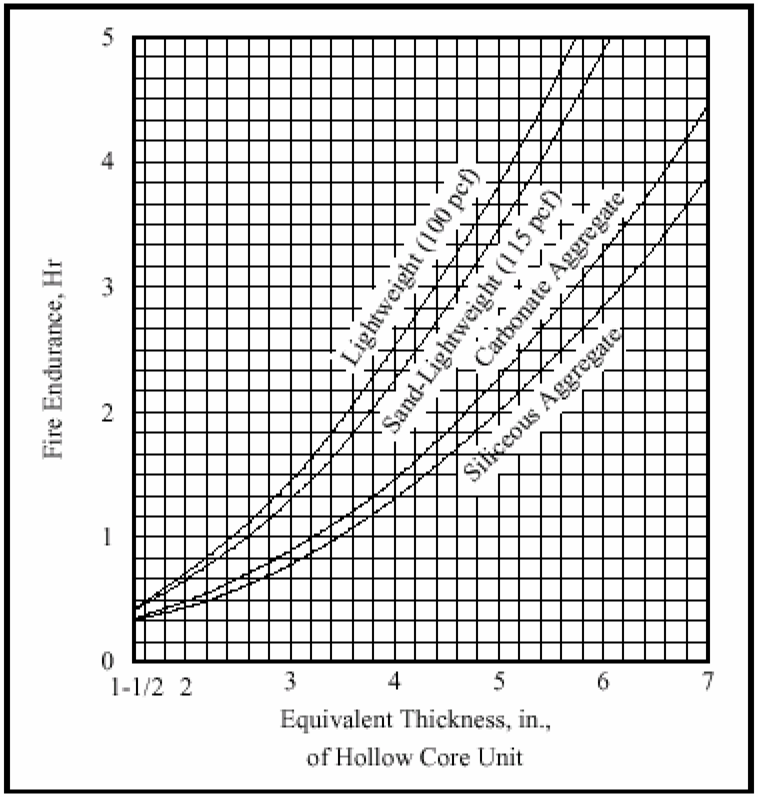
- Provides economical and efficient floor and roof systems.
- The top surface can be prepared for installation of floor covering or concrete topping of up to 2in.
- The underside can be used as a finished ceiling.
- The voids can be used for electrical or mechanical runs.
- Provides the efficiency of a prestressed member for load capacity, span range and deflection control.

Advantages of Hollow-Core Slabs (Cont.)

- HC slabs can be used as diaphragms to transfer lateral loads.
- Provides excellent fire resistance. Depending on the strand cover it can endure up to 4 hour.
- A variety of architectural finishes are available.

Hollow-Core Slabs Fire Endurance

Fig. 6.2 Fire endurance (heat transmission) of hollow core units



8” Hollow-Core Slab

Equivalent thickness = 4.1in

10” Hollow-Core Slab

Equivalent thickness = 5.2in

Hollow-Core Slabs Connections

- Simply supported.
- Fixed connections:

Fixed connection can be achieved by the use of top cables or toppings with proper reinforcement.

Hollow-Core Slabs Connections Samples

Design Considerations:

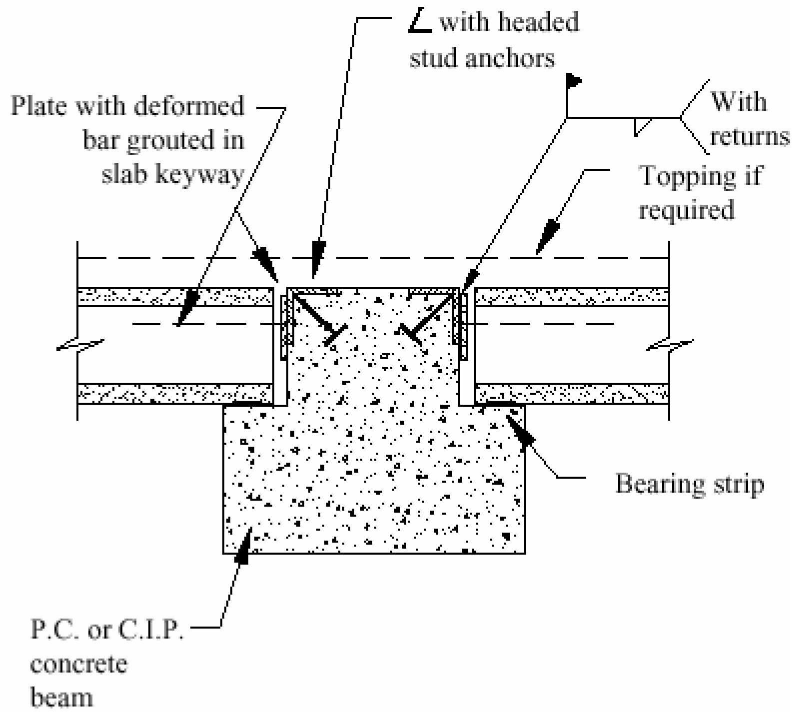
- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie

Fabrication Considerations:

- Advantageous to have no hardware in slab
- Beam embedments must line up with slab joints
- Accommodates variations in slab length

Erection Considerations:

- Advantageous to have connection completed by follow-up crew
- Difficult for welder to hold loose plate in position



Hollow-Core Slabs Connections Samples

Design Considerations:

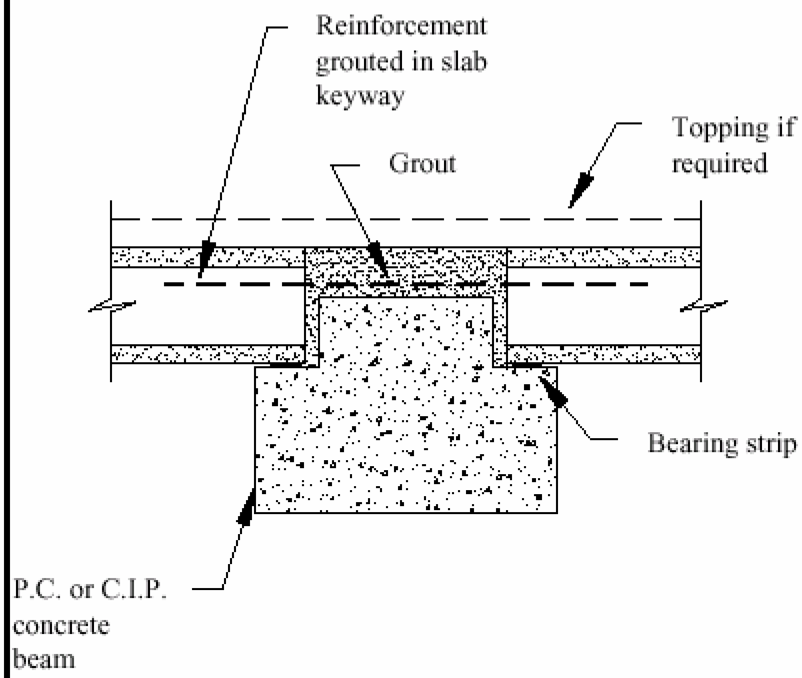
- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie

Fabrication Considerations:

- May increase beam reinforcement for shallower beam
- Layout must have opposing slab joints lined up

Erection Considerations:

- Clean and simple



Hollow-Core Slabs Connections Samples

Design Considerations:

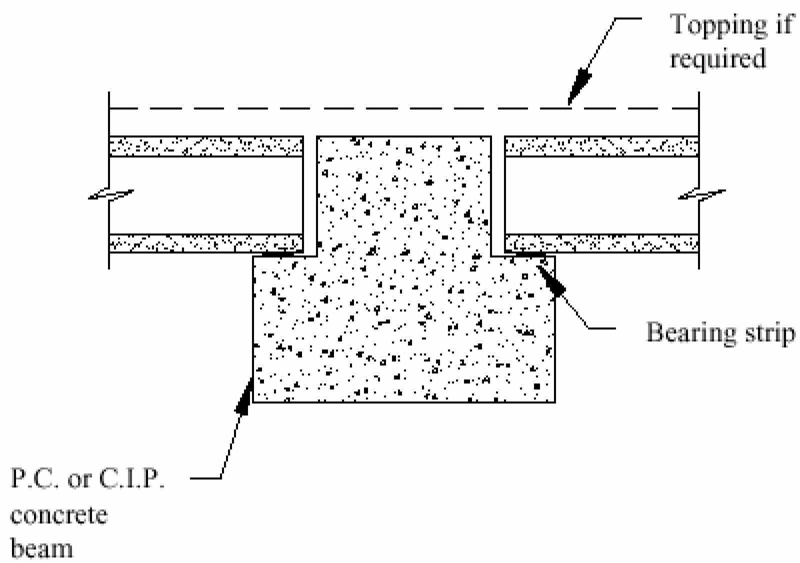
- With large factors of safety, friction may transfer nominal forces
- Additional structural integrity ties may be required

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Clean and simple



Hollow-Core Slabs Connections Samples

Design Considerations:

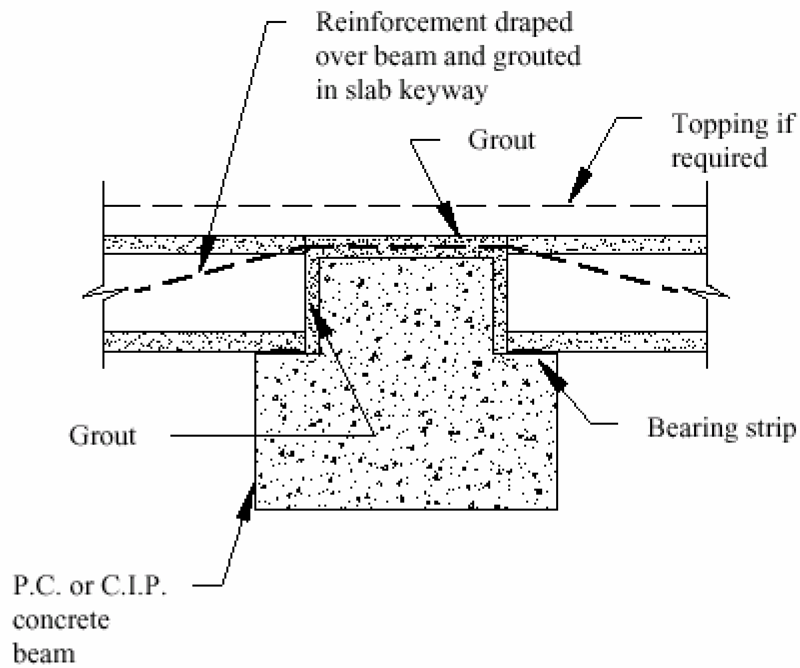
- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie
- Consider concrete cover on reinforcement over beam

Fabrication Considerations:

- Slab layout must have opposing joints lined up

Erection Considerations:

- Clean and simple



Hollow-Core Slabs Connections Samples

Design Considerations:

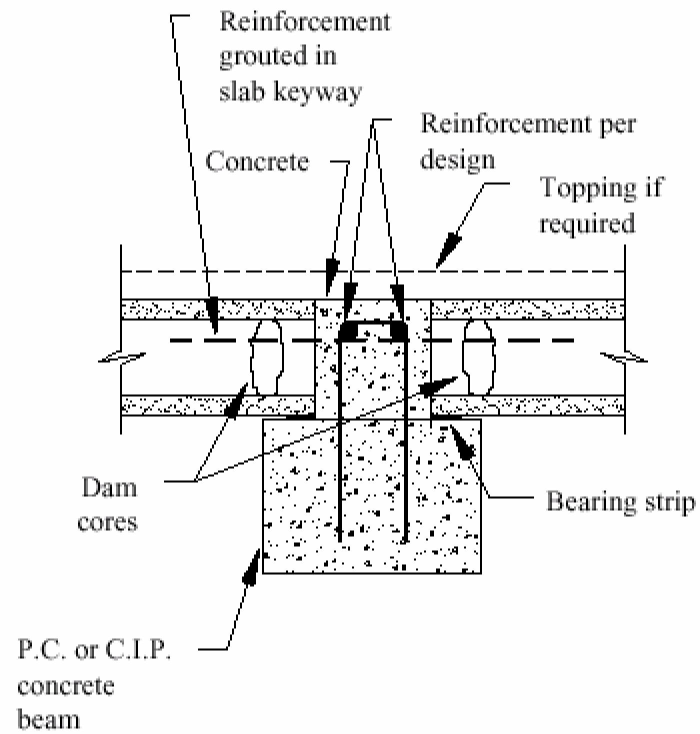
- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie
- Horizontal shear from beam cap must be transferred
- Opposing slab joints must line up

Fabrication Considerations:

- Clean and simple for slabs

Erection Considerations:

- Beam may have to be shored until cap is cured
- Horizontal shear reinforcement may present safety hazard for erector
- Core dams must be placed



Hollow-Core Slabs Connections Samples

Design Considerations:

- Can transfer internal diaphragm forces
- Provides lateral brace for steel beam

Fabrication Considerations:

- Slab layout must align slab joints
- Stabilizer bars might be field or shop installed depending on local regulations or agreements
- Beam flange width must be sufficient for minimum slab bearing

Erection Considerations:

- Grouting of slabs must include the butt joint
- Steel erection may require that stabilizer bars be field installed
- Steel beam will not be laterally braced until grout cures
- Unsymmetrical loading may cause beam instability

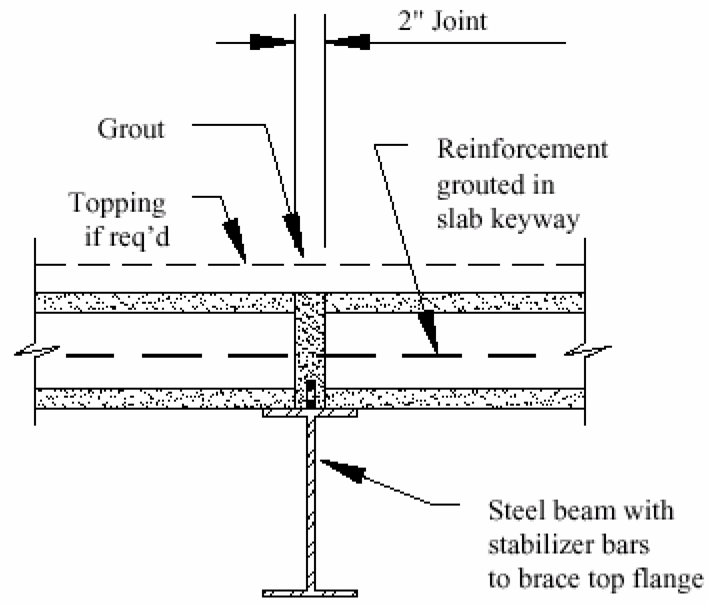


Fig. 5.5.2

ACI 318-99 Design Considerations

The following design stages should be considered when designing a hollow-core slab:

- Immediately after prestress transfer.
- Transportation and installation.
- At service (installed).
- Ultimate capacity.
- Check Deflections.

ACI 18.4.1
Stresses in concrete after transfer:

1. Extreme fiber stress in compression:

   $f'_{cai} = 0.60 f'_{ci}$

2. Extreme fiber stress in tension:

   $f_{tai} = 3\sqrt{(f'_{ci})}$

3. Extreme fiber stress in tension at ends of simply supported members:

   $f_{tai} = 6\sqrt{(f'_{ci})}$

ACI 18.4.2
Stresses in concrete at service :

1. Extreme fiber stress in compression:

   $f'ca = 0.45f'c$

2. Extreme fiber stress in compression due to prestress plus total loads:

   $f'ca = 0.60f'c$

3. Extreme fiber stress in tension at ends of simply supported members:

   $fta = 6\sqrt{(f'c)}$

ACI 18.5.1
Permissible Stresses in PS Tendons :

1. Due to tendon jacking force:

   The lesser of $0.94f_{py}$ or $0.80f_{pu}$

2. Immediately after prestress transfer:

   The lesser of $0.82f_{py}$ or $0.74f_{pu}$

3. Effective stress in prestress reinforcement:

   $f_{se} \geq 0.50f_{pu}$

ACI 18.6.1 Loss of Prestress

- Tendon seating at transfer.
- Elastic shortening of concrete.
- Creep of concrete.
- Shrinkage of concrete.
- Relacation of tendons.

ACI 18.7.2 Ultimate Moment Capacity

- Stress in prestress at nominal strength:

$$fps = fpu * \left\{1 - \frac{\gamma p}{\beta 1}\left[\rho p * \frac{fpu}{f'c}\right]\right\}$$

$\gamma p = 0.28$ fpu = tensile strength of tendonds

$\rho p = Aps/b*dp$ $\beta 1$ = factor defined in ACI 10.2.7.3

- Ultimate Moment:

$$\phi Mn = 0.90 fps * Aps\left(dp - \frac{a}{2}\right) \qquad a = \frac{Aps * fps}{0.85 f'c * b}$$

ACI 11.4.1 Shear Strength

- For member with effective prestress force not less than 40 percent:

$$\phi Vc = 0.85 * \left(0.6\sqrt{f'c} + 700 \frac{Vu * d}{Mu} \right) bw * d$$

f’c = compresive strength at 28 days   Mu = factored moment

Vu = factored shear force   bw = web width

d = reinforcement depth

Note: A more detailed calculation can be made in accordance with ACI 11.4.2

Concrete shear capacity need not to be less than:

$$\phi Vc\min = 0.85 * 2 * \sqrt{f'c} * bw * d$$

Nor greater than:

$$\phi Vc\max = 0.85 * 5 * \sqrt{f'c} * bw * d$$

$$\frac{Vu * d}{Mu} \le 1.0$$

ACI Table 9.5(b) Deflections

TABLE 9.5(b)—MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\frac{l}{180}$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\frac{l}{360}$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\frac{l}{480}$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\frac{l}{240}$§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

† Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.2, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ But not greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

ACI 16.6.2.2 Support Details

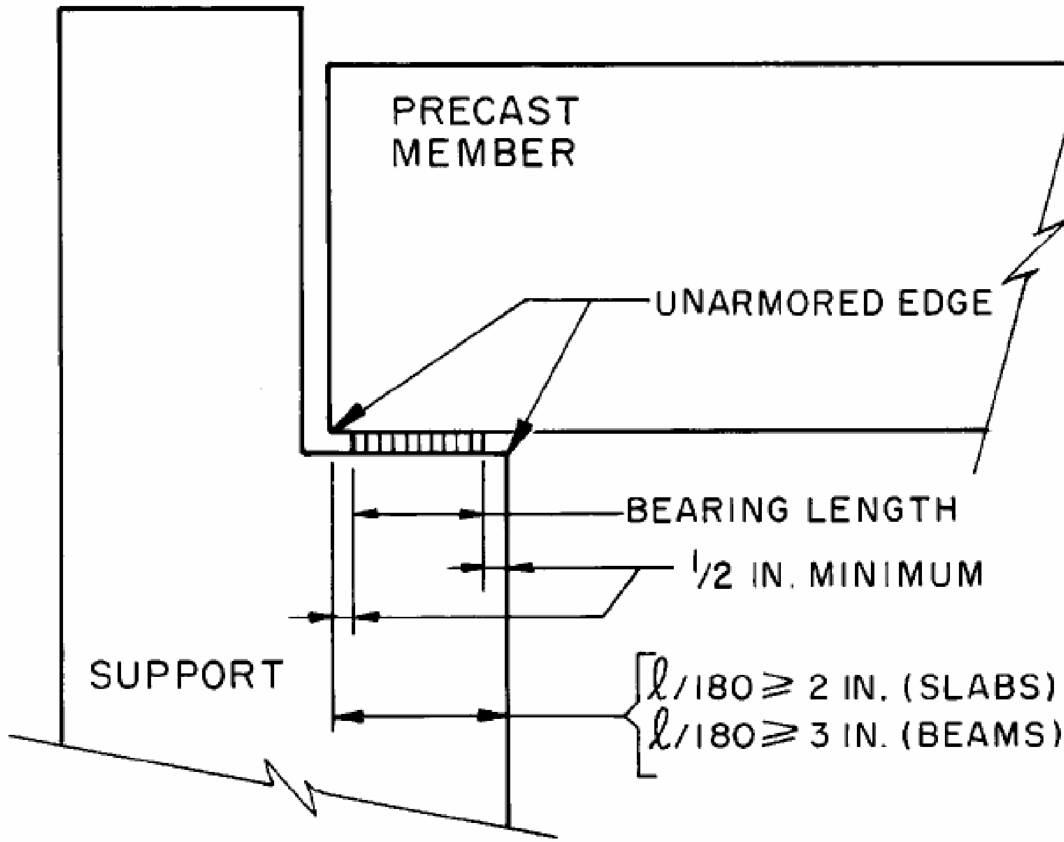


Fig. R16.6.2—Bearing length versus length of member on support

Hollow-Core Cross Sectional Properties

8" Slab		10" Slab	
No Topping	2" Topping	No Topping	2" Topping
$A=196in^2$	$A=292in^2$	$A=249in^2$	$A=345in^2$
$I=1580in^4$	$I=3024in^4$	$I=5280in^4$	$I=5280in^4$
Yt=4.03in	Yt=4.60in	Yt=5.00in	Yt=5.56in
Yb=3.97in	Yb=5.40in	Yb=5.00in	Yb=6.44in
bw=11.39in	bw=11.39in	bw=13.34in	bw=13.34in
Wt=51psf	Wt=76psf	Wt=65psf	Wt=90psf