Design Principles and Calculations - Slab Design

The Hambro Slab

The slab component of the Hambro D500 Composite Floor System behaves as a continuous one-way slab carrying loads transversely to the joists, and often is required to also act as a diaphragm carrying lateral loads to shear walls or other lateral load resisting elements.

At the present time, the Hambro slab has been designed by conventional ultimate strength design procedures of ACI 318 and section capacities are based on ultimate strength principles while the moments are still determined by using the elastic moment coefficients for continuous spans. This procedure is present in most other building codes.

In accordance with most building code requirements, the Hambro slab capacity is determined for two basic loading arrangements: a) uniform dead and live load extending in all directions, and b) a "standard" concentrated live load, applied anywhere, together with the slab dead loads. It is important to remember that the live load arrangements of a) and b) do not occur simultaneously. These loading arrangements will be discussed in detail.

Moment:

The basic ultimate strength moment expressions from ACI are shown below:

$$M_u = A_s a_u d \quad \text{.............} \quad (1)$$

Where

M_u = ultimate moment capacity of slab *(ft.-kips/ft. width)*

A_s = area of reinforcing mesh in the direction of the slab span *(in.²/ft. width)*

a_u = $\phi f_y (1 - .59\,w) / 12000$

ϕ = flexure factor = 0.9

f_y = yield strength of reinforcing mesh = $60{,}000$ *psi* (or as calculated by the ACI offset provision)

w = $p \dfrac{f_y}{f'_c}$

p = tension steel ratio = A_s/bd

f'_c = compressive strength of concrete = $3{,}000$ *psi*

b = unit slab width = 12 inches

d = distance from extreme compression fibre to centroid of reinforcing mesh (in.) = 1.6 inches for 2-½ inch slab

It is a simple matter, then, to determine M_u for any combination of A_s and d. Taking into account ¾ inch concrete cover, "d" is taken to be 1.6 inches for the 2-½ inch slab thickness. The ACI Ultimate Strength Design Handbook Vol. 1, Publication SP17, contains tabulated values for "a_u" (note that a_u increases as the tension steel ratio "p" decreases).

Crack Control Provisions:

When design yield strength f_y for tension reinforcement exceeds $40{,}000$ *psi*, cross sections of maximum positive and negative moment shall be so proportioned that the quantity z given by:

$$z = f_s \sqrt[3]{d_c A} \quad \text{.............} \quad (2) \text{ [ACI 10.6.3.4]}$$

does not exceed 175 *kips* per in. for interior exposure and 145 *kips* per in. for exterior exposure. Calculated stress in reinforcement at service load f_s (kips per sq. in.) shall be computed as the moment divided by the product of steel area and internal moment arm. In lieu of such computations, f_s may be taken as 60 percent of specified yield strength f_y.

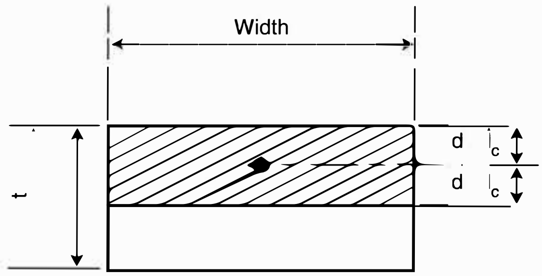


Fig. 1

Considering the negative moment region where the mesh rests directly on the embedded top chord connector, the centroid of the mesh is 1.6 inches above the extreme concrete compression fibre. With 6 inches wire spacing and $t = 2\text{-}1/2$ inches, $d_c = 0.9$ inches;

A (hatched area) $= 1.8 \times 6 = 10.8$ *in.²*;

Using $f_s = 60\%$ of 60 *ksi* $= 36$ *ksi*;

"z" in formula 2 becomes 77 *kips per inch.*

Even for a 3 inch slab with the lever arm still at 1.6 inch and $d = 1.4$ inch, "z" $= 103$ *kips per inch*, well under the allowable.

Shear Stress

The ultimate shear capacity, v_{cu}, which is a measure of diagonal tension, is unaffected by the embedment of the top chord section as this principal tensile crack would be inclined and radiate away from the z section. Furthermore, there is no vertical weak plane through which a premature "punching shear" type of failure could occur.

A check of the v_{cu} capacity for $d = 1.6$ *inch* ($2\text{-}^1/_2$ *inch slab*) and $f'_c = 3{,}000$ *psi* is shown:

$$v_{cu} = \frac{V_u}{\phi bd} \quad \text{...} \ (3)$$

$$V_u = \phi v_{cu} bd \quad \text{...} \ (4)$$

Substituting the following values in (4): $v_{cu} = 2\sqrt{f'_c} = 100\ psi$, $b = 12"$, $d = 1.6"$, $\phi = .85$, results in:

$$V_u = 1{,}785\ lbs.$$

With a Load Factor of 1.7, the $2\text{-}^1/_2$ *inch* slab spanning 4 *feet* $1\text{-}^1/_4$ *inch* c/c has a safe shear capacity of 525 *psf.*

Deflection:

The span / thickness ratio $49.5 / 2.5 = 20$ is less than the maximum allowable 24 for the one end continuous condition. Slab deflection, Δ, due to a uniformly distributed load, can be written as:

$$\Delta = \frac{K_1 w L^4}{E_c I_c} \quad \text{..} \ (5)$$

$$\frac{\Delta}{L} = \frac{K_1 w L^3}{E_c I_c} \quad \text{..} \ (6)$$

For the same w, E_c and slab end conditions, (6) can be rewritten:

$$\frac{\Delta}{L} = K_2 \frac{L^3}{I_c}$$

Hence, the Δ / L ratios of different floors can be used to assess relative deflection.

Example:

Hambro $2\text{-}^1/_2$ inch slab / - 4 foot $1\text{-}^1/_4$ inch span

$$I_c = 12\,(2.5)^3 / 12 = 15.6\ in.^4$$

$$\Delta / L = L^3 / I_c = (4.1)^3 / 15.6 = 4.4$$

$7\text{-}^1/_2$ inch slab / 20 foot span

$$I_c = 12\,(7.5)^3 / 12 = 422\ in.^4$$

$$\Delta / L = L^3 / I_c = (20)^3 / 422 = 19$$

Clearly, then, the Hambro slab deflections expressed in terms of Δ / L are less with Hambro than with a $7\text{-}^1/_2$ inch slab spanning 20 feet.

Uniform Live Load Arrangement:

Generally, the slab capacity is checked against the condition where the dead and live loads are uniformly distributed.

A load factor of 1.7 is used for both dead and live load capacities. For a more exact analysis, factored loads of $1.4D$ and $1.7L$ should be considered.

Refer to Fig. 2 for location of the design moments below.

Ultimate positive moment, M_u:

- exterior span $1.7\ w_s L_1^2/11$ .... location 1
- interior span $1.7\ w_s L_2^2/16$ .... location 3

Ultimate negative moment, M_u:

- exterior span $1.7\ w_s L^2/10$ .... location 2
- interior span $1.7\ w_s L_2^2/11$ .... location 4

Where

- L_1, L_2 = clear span (ft.) is less than $1\ 3/4"$ less than joist spacing.
- L = average of L_1 and L_2 (ft.)
- w_s = total design load (dead + live) in psf

Note that a conservative load factor of 1.7 has been used for dead and live load.

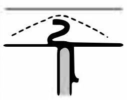
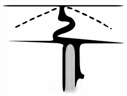
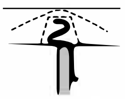
When $L_1 = L_2$, maximum mesh stress occurs at location 2.

When $L_1 \leq 0.9\ L_2$, maximum mesh stress occurs at location 4.

The slab load tables are reproduced in Table 1.

DESIGN PRINCIPLES AND CALCULATIONS - SLAB DESIGN

Table 1 - Slab Capacity Chart (Total Load in psf)

SLAB THICKNESS (t)		d (inch)	MESH SIZE (all 6 in. x 6 in.) F'c = 3,000 psi F_y = 60,000 psi	JOIST SPACING			
				4'-1 1/4"		5'-0"	
				Exterior	Interior	Exterior	Interior
 No Chair	$t \geq 2\text{-}{}^{3}/_{4}$ in. and $t \leq 3\text{-}{}^{5}/_{8}$ in.	1.6 in.	6 x 6 - 2.9 / 2.9 6 x 6 - 4.0 / 4.0	159 212	175 233	108 143	118 157
	$t \geq 3\text{-}{}^{5}/_{8}$ in. and $t \leq 5$ in.	1.6 in.	6 x 6 - 4.0 / 4.0 2 layers 6 x 6 - 2.1 / 2.1 2 layers 6 x 6 - 2.9 / 2.9	219 226 304	241 248 334	148 152 204	162 167 223
 1/2" Rod Shop Welded to Top Chord	$t \geq 3$ in. and $t \leq 3\text{-}{}^{5}/_{8}$ in.	2.1 in.	6 x 6 - 2.9 / 2.9 6 x 6 - 4.0 / 4.0	220 295	241 324	148 198	162 217
	$t \geq 3\text{-}{}^{5}/_{8}$ in. and $t \leq 5$ in.	2.1 in.	6 x 6 - 4.0 / 4.0 2 layers 6 x 6 - 2.1 / 2.1 2 layers 6 x 6 - 2.9 / 2.9	296 307 415	326 336 457	200 206 279	219 226 306
 With 2 1/2" Chair	$t \geq 3\text{-}{}^{5}/_{8}$ in. and $t \leq 5$ in.	2.6 in.	6 x 6 - 4.0 / 4.0 2 layers 6 x 6 - 2.1 / 2.1 2 layers 6 x 6 - 2.9 / 2.9	353 363 497	387 399 545	236 244 333	259 267 365

Note: Slab capacities are based on mesh over joists raised as indicated.

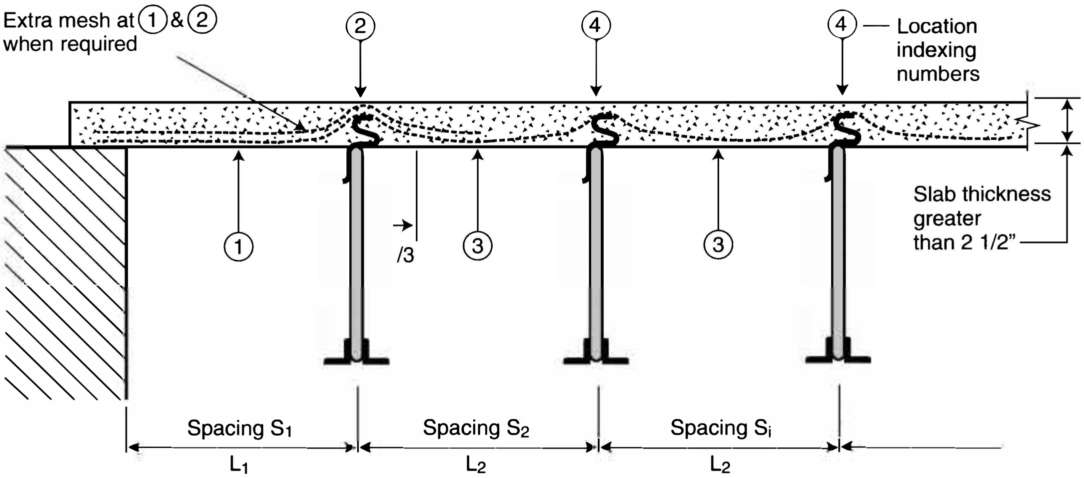


Fig. 2

Design Principles and Calculations - Slab Design

Concentrated Live Load Requirements

Building Codes usually stipulate designing to possible concentration of live loads. Typical examples of some of these situations are listed in Table 2. Check your local codes for exact requirements.

The loads are applied over an area *2-1/2 feet* by *2-1/2 feet* and it is important to remember that they are not applied simultaneously with the uniformly distributed live loads.

Table 2

USE	MINIMUM CONCENTRATED LOAD (lb.)
Classrooms	1000
Floors of offices, manufacturing buildings, hospital wards, stages	2000
Floor areas used by passenger cars	2500*

* Some building codes use 2000 lbs.

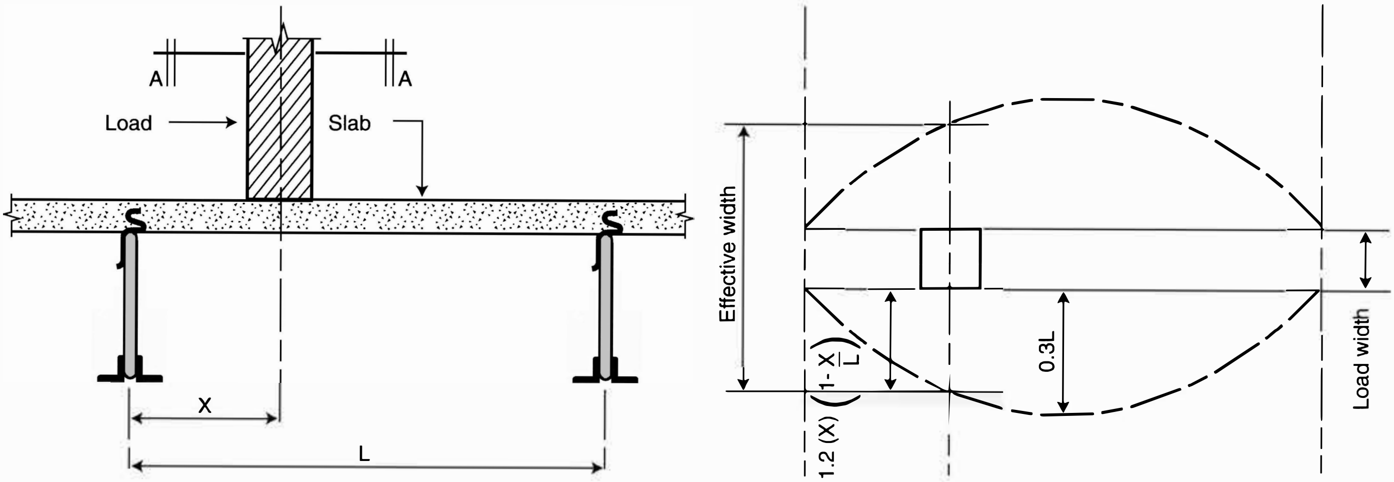


Fig. 3
Lateral Distribution of Concentrated Loads

The intensity of concentrated loads on slabs is reduced due to lateral distribution. One of the accepted methods of calculating the "effective slab width," which is used by Hambro, actually appears in Section 317 of the British Standard Code of Practice CP114 and is reproduced in Fig. 3. Note that the amount of lateral distribution increases as the load moves closer to mid span, and reaches a maximum of $0.3L$ to each side; the effective slab width resisting the load is a maximum of load width + $0.6L$.

An abbreviated summary of the calculations is shown in Tables 3 and 4.

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TABLE 3 - Concentrated Loads with 4 Foot-1-$^1/_4$ inch Joist Spacing

CONCENTRATED LOAD	SLAB THICKNESS	MESH SIZE	SPECIAL REMARKS	
2000 lbs. on 2 feet-6 inch square area (office building)	**2-$^1/_2$ in.**	6 x 6 - W2.9	Extra layer @ ①	No "chairs" on
		6 x 6 - W2.9	Single layer throughout but S_1= 3 feet-10 inch max.	
	3 in.	6 x 6 - W2.9	Extra layer @ ① and ②	
		6 x 6 - W2.9	Single layer throughout but S_1= 4 feet max.	
		6 x 6 - W2.9	Single layer throughout	
2500 lbs. on 2 feet-6 inch square area plus 2 inch asphalt wearing surface	**3 in.**	6 x 6 - W2.9	Extra layer @ ① and ②	No "chairs" on
		6 x 6 - W2.9	Single layer throughout but S_1=2 feet-10 inch max.	
4000 lbs. on 3 feet-6 inch square area (office building for some codes)	**2-$^1/_2$ in.**	6 x 6 - W4.0	S_1 = 4 feet	No "chairs" on
	3 in.	6 x 6 - W2.9	Extra layer @ ① and ②	
		6 x 6 - W2.9	Single layer throughout but S_1=2 feet-10 inch max.	

*Some building codes use different bearing areas.

TABLE 4 - Concentrated Loads with 5 feet-1-$^1/_4$ inch Joist Spacing

CONCENTRATED LOAD	SLAB THICKNESS	MESH SIZE	SPECIAL REMARKS	
2000 lbs. on 2 feet- 6 inch square area (office building)	**3 in.**	6 x 6 - W2.9	Extra layer @ ① and ②	No "chairs" on
4000 lbs. on 3 feet- 6 inch square area (office for some codes)	**3 in.**	6 x 6 - W4.0	Extra layer @ ① and ②	No "chairs" on

Concrete Mix

Top size of the coarse aggregate should not exceed $^3/_4$ *inch* or as dictated by applicable codes. A slump of *4* inches is recommended.