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 = $\text{ø} f_y (1 - .59\, w) / 12000$

ø = 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 $2\text{-}^1/_2$ inch slab

It is a simple matter, then, to determine M_u for any combination of A_s and d. Taking into account $^3/_4$ inch concrete cover, "d" is taken to be *1.6* inches for the $2\text{-}^1/_2$ 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.

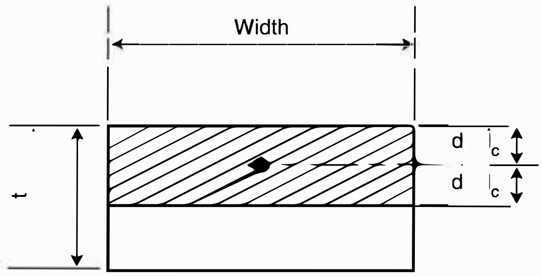


Fig. 1

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-$^1/_2$ *inch slab*) and $f'_c = 3{,}000$ *psi* is shown:

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

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

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

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

With a Load Factor of *1.7*, the *2-$^1/_2$ inch* slab spanning *4 feet 1-$^1/_4$ 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 $\Delta\ /\ L$ ratios of different floors can be used to assess relative deflection.

Example:

Hambro 2-$^1/_2$ inch slab / - 4 foot 1-$^1/_4$ 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-$^1/_2$ 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 $\Delta\ /L$ are less with Hambro than with a 7-$^1/_2$ 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.

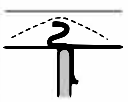
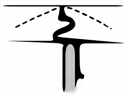
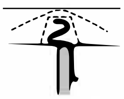
When $L_1 = L_2$, maximum mesh stress occurs at location 2.

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.

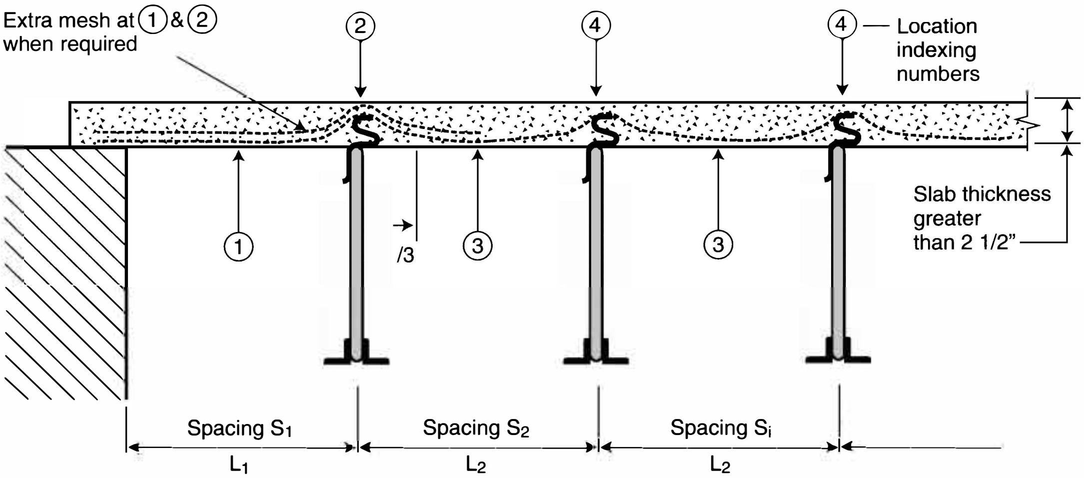


Fig. 2

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 $2\text{-}^{1}/_{2}$ *feet* by $2\text{-}^{1}/_{2}$ *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.

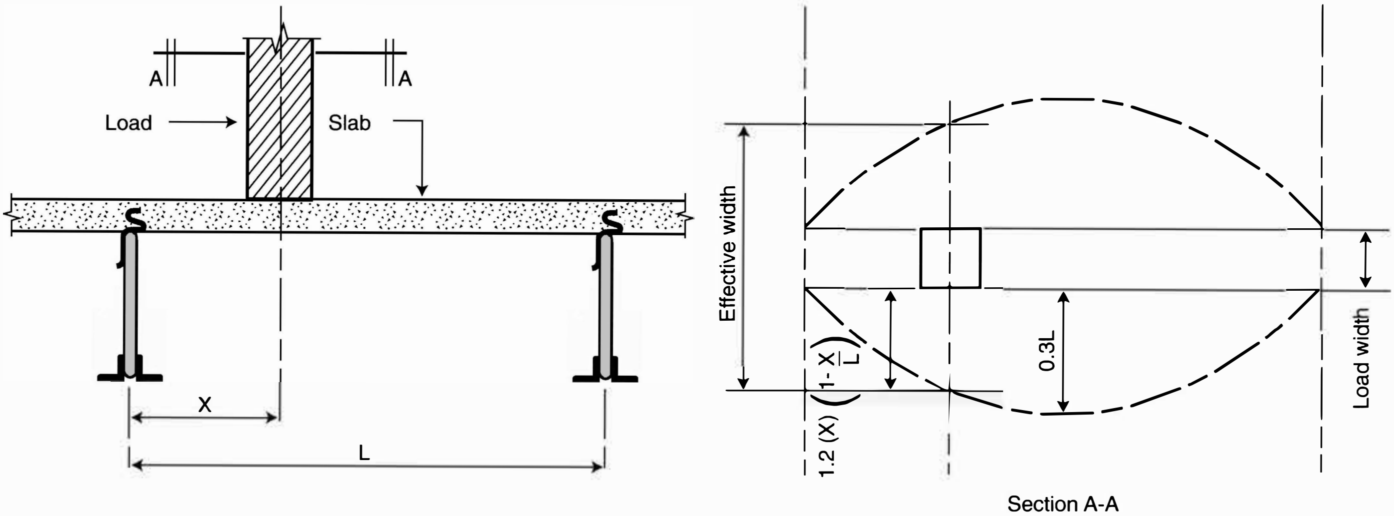


Fig. 3
Lateral Distribution of Concentrated Loads

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.

Design Principles and Calculations -Slab Design

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.	

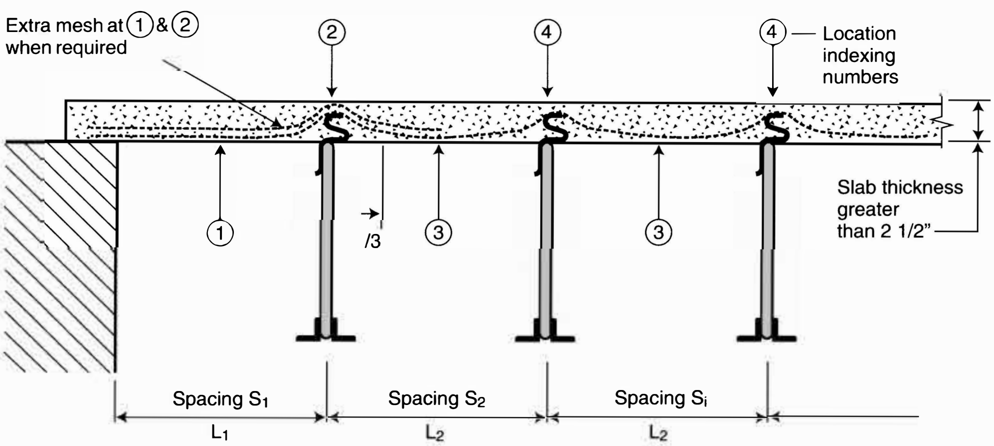
*Some building codes use different bearing areas.

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.



This information is not for construction, not to scale and subject to change without notice.

NON-COMPOSITE DESIGN

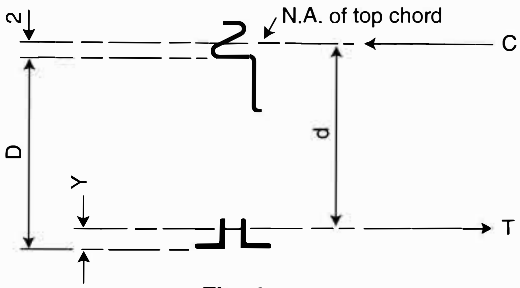


Fig. 4

The top chord must be verified for the loads applied at the non-composite stage. From the previous example, we have the following results:

- Dead load:

Slab $2\text{-}^{1}/_{2}$ inch:	*31 psf*
Formwork and joist:	*5 psf*
	36 psf

- Live load:

Construction live load:	*20*psf*
	56 psf

* Reduces beyond *25* foot span at a rate of *1 psf for each 2.5* feet of span.

Moment Capacity of Joist = $C \times d$ or $T \times d$

i.e. $\frac{W_{nc} L^2}{8} \times 12 = C \times d$ or $T \times d$, whichever is the lesser

W_{nc} = *56* x joist spacing = *plf*

L = clear span + *4" (ft.)*

C = area of top chord *(sq. in.)* x working stress *(psi)*

T = area of bottom chord *(sq. in.)* x working stress *(psi)*

d = effective lever arm in *inches*

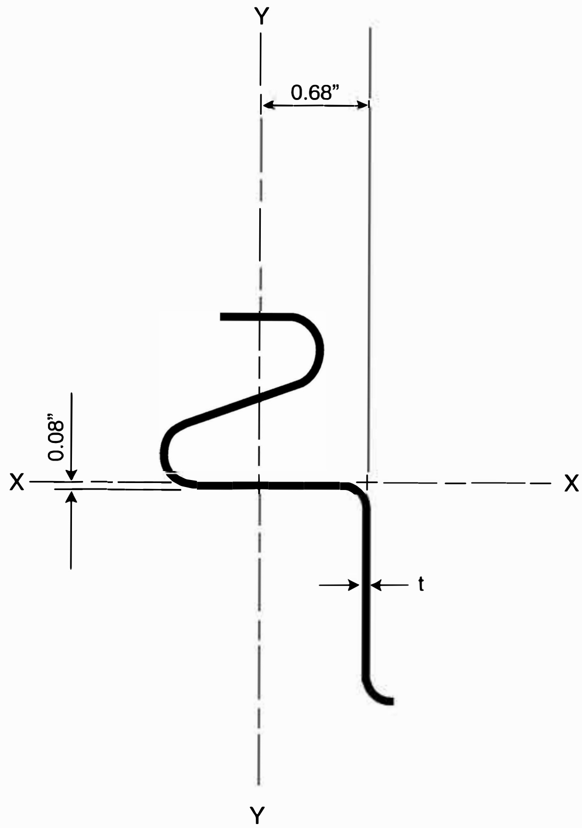
$= D + 0.08 - y$

From the above formula, the maximum "limiting span" may be computed for the non-composite (construction stage) condition. For spans beyond this value, either the top chord must be strengthened or joist propped. Strengthening of the top chord, when required, is usually accomplished by installing one or two rods in the curvatures of the "S" part of the top chord.

The bottom chord is sized for the total factored load which is more critical than the construction load.

Hambro top chord properties are provided to assist you in computing the non-composite joist capacities.

TOP CHORD PROPERTIES



$I_x = 0.66 \text{ in.}^4$

Top Chord $S_x = 0.45 \text{ in.}^3$

Bottom Chord $S_x = 0.287 \text{ in.}^3$

$L_y = 0.187 \text{ in.}^4$

$S_y = 0.167 \text{ in.}^3$

t = *13 ga.* = *0.090 in.*

$A_{net} = 0.56 \text{ in.}^2$

= *(deducting 3/8" deep slot = 6.25 x 0.090)*

F_y = *50 ksi min.*

F_a = *29.1 ksi*

FLEXURE DESIGN

In the past, conventional analysis of composite beam sections has been linearly elastic. Concrete and steel stresses have been determined by transforming the composite section to a section of one material, usually steel, from which stresses are then determined with the familiar formula, $f = My / I$, and then compared to some limiting values which have been set to ensure an adequate level of safety. Although this procedure is familiar to most engineers, it does not predict the level of safety with as much accuracy as does an ultimate strength approach which is based on the actual failure strengths of the component materials.

It is now known that the flexural behavior at "ultimate" failure stages of composite concrete/steel beams and joists is similar to that of reinforced concrete beams - the elastic neutral axis begins to rise under increasing load as the component materials are stressed into their inelastic ranges. The typical stress-strain characteristics of the concrete and steel components are shown in fig. 5.

The various loading stages of the Hambro composite joist are indicated in fig. 6. As load is first applied to the composite joist, the strains are linear. The "elastic" neutral axis, concrete and steel stresses can be predicted from the conventional transformed area method. Generally speaking, the Hambro composite joist behaves in this "elastic" manner when subjected to the total working loads. With increasing load, failure always begins initially with yielding of the bottom chord. In (a), all of the bottom chord has just reached the yield stress, F_y. The maximum concrete strains will likely have just progressed into the inelastic concrete range, but the maximum concrete stress will still be less than $0.85 f'_c$.

With a further increase in load, large inelastic strains occur in the bottom chord and the ultimate tensile force, T_u , remains equal to $A_s F_y$. The strain neutral axis rises, as does the centroid of the compresssion force. Part (b) depicts the stage when the maximum concrete stress has just reached $0.85 f'_c$. At this stage, the ultimate resisting moment has increased slightly due to a small increase in lever arm.

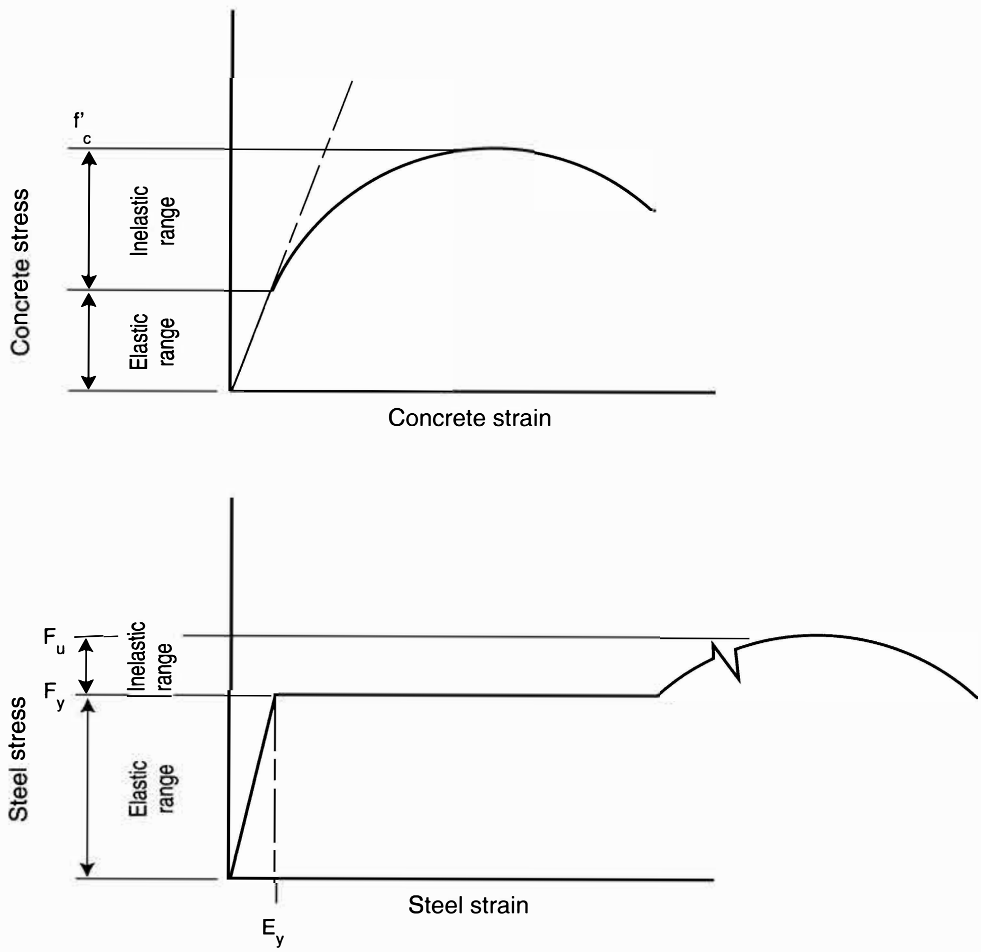


Fig. 5
Concrete and Steel Stress - Strain Curves

VARIOUS FLEXURAL FAILURE STAGES

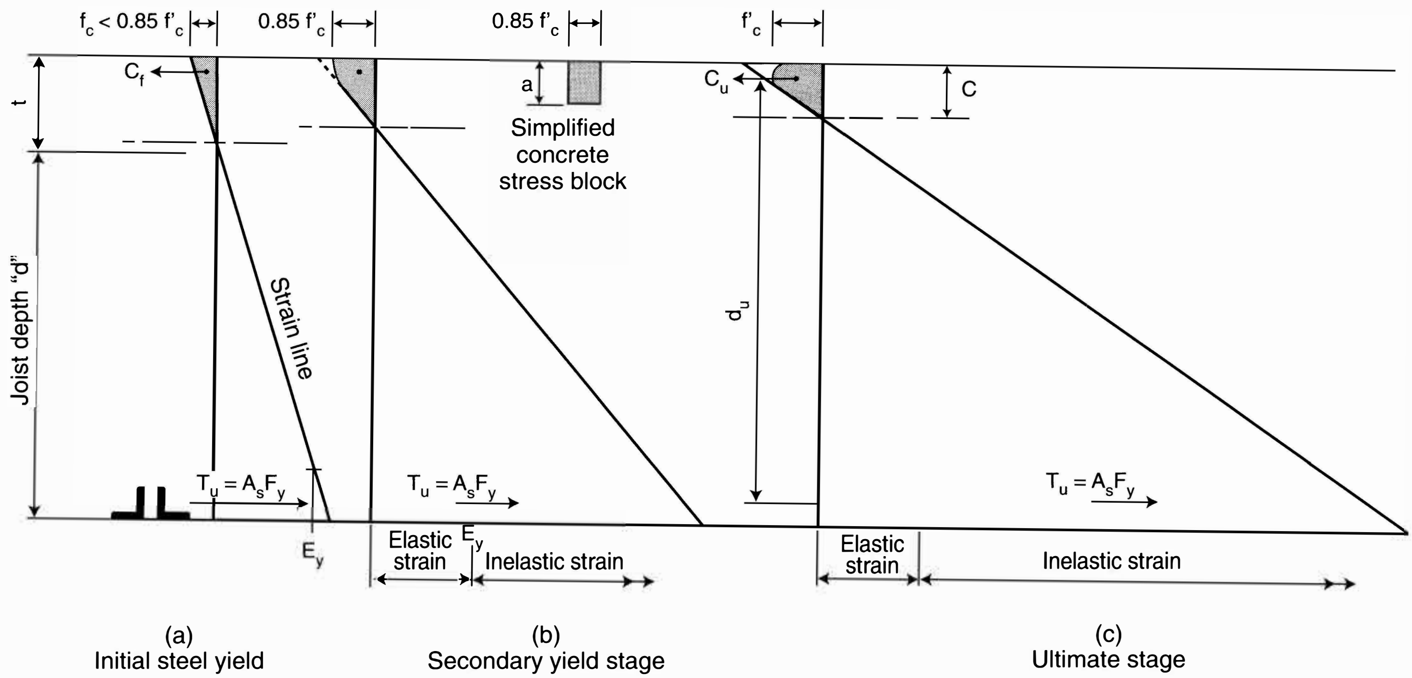


Fig. 6

Upon additional load application, the steel and concrete strains progress further into their inelastic ranges. The strain neutral axis continues to rise and the lever arm continues to increase as the centroid of compression force continues to rise. In (c), final failure occurs with crushing of the upper concrete fibers. At this point, the maximum lever arm, du, has been reached. In load capacity calculations, the simplified concrete stress block as shown in (c) is universally used.

It is a simple matter to calculate the ultimate moment capacity of any Hambro composite joist, knowing the bottom chord size.

$$T_u = A_s F_y \quad \text{.......} \quad (7)$$

Also, C_u must equal T_u. Using an additional 0.9 concrete stress factor

$$C_u = 0.9 \times 0.85 f'_c ab \quad \text{.......} \quad (8)$$

Where b = lesser of span / 4, joist spacing, or $16\ t;\ f'_c = 3{,}000\ psi$

Equating (7) and (8) results in *"a"* becoming the only unknown and is easily calculated by the expression:

$$a = \frac{T_u}{0.9 \times 0.85 f'_c b} \quad \text{.......} \quad (9)$$

The lever arm, d_u, can now be determined and the ultimate moment capacity, M_u, is calculated from the expression:

$$M_u = T_u\ d_u \quad \text{.......} \quad (10)$$

Now M_u also $= M_u = \frac{W_u L^2}{8}$ .... (11)

Equating (10) and (11), $W_u \equiv \frac{8\ T_u\ d_u}{L^2}$

Using a load factor of 1.7, the working load capacity (total dead and live load) of the composite joist per unit length,

$$W_s = \frac{W_u}{1.7}$$

For a more exact analysis, factored loads will be considered.

Design Principles and Calculations - Web Design

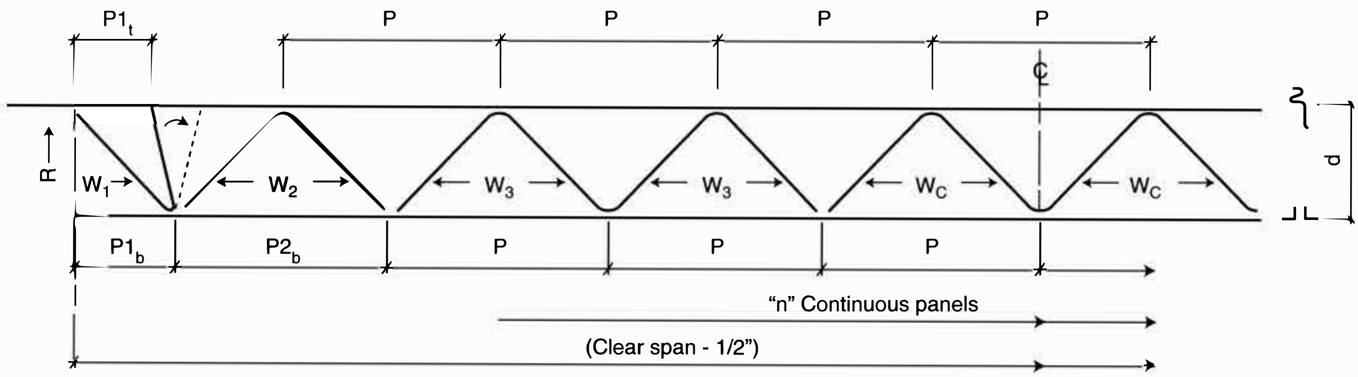
Vertical Shear (Web Design)

The vertical shear forces are assumed to be carried entirely by the web member, forces being calculated using the conventional pin jointed truss analysis method. These assumptions result in calculated bar forces which have been shown by tests to be as much as 15% higher than the actual values because the slab, acting compositely with ⺄ section, is stiff enough to transmit some load directly to the support. This is particularly true of web members at the joist ends - those which are subjected to the highest vertical shear.

Effective Length of Compression Diagonal

With the web member forces calculated as below, the bar sections are sized to prevent failure in either axial tension or axial compression using conventional working stress design procedures. As per AISC specifications fig. 7 is used as a reference in determining the effective length, k_l, of the compression diagonals.

It is important to note that the web members are sized for the specified load capacity including concentrated loads where applicable. Furthermore, the webs are designed according to the latest requirements of the Steel Joist Institute.



NOTE: W_3 for longer span.

Fig. 7
D500™ and MD2000® Geometry

WEB GEOMETRY (in.)				
NOM. DEPTH "d"	**$P1_t$**	**$P1_b$**	**$P2_b$**	**P**
8, 10	6 @ 12	6 @ 16	12	20
12	10 @ 16	10 @ 21	16	24
14, 16	15 @ 24	15 @ 32	20	24
18, 20, 22, 24	19 @ 24	19 @ 32	24	24

Design Principles and Calculations - Interface Shear

Shear

Composite action between the section and the concrete slab exists because of the unique shear resistance developed along the interface between the two materials. This shear resistance, which has been called "bond" or "interface shear" is primarily the result of a "locking" or "clamping" action in the longitudinal direction between the concrete and the section when the composite joist is deflected under load. Another contributing factor to the shear resistance is the lateral compression stress or "poisson's effect" which results from slab continuity in the lateral direction. This continuity prevents lateral expansion from occuring as a result of longitudinal compression stresses and thus lateral compression stress results. However, this effect has been ignored in determining interface shear capacity which has been based on full scale testing of spandrel joists having only a 6 inch slab overhang on one side for its entire span length. A cross-section of a test specimen is illustrated in fig. 8.

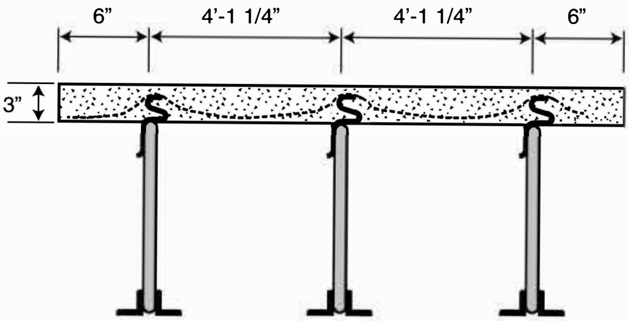


Fig. 8

It was decided to base the limiting interface shear value on this most critical condition as this could often occur in practice with large duct openings. Also, one would expect some additional shear resistance to occur due to some form of friction (or plain "bond") mechanism, however, **full scale tests have not shown any significant differences in results among specimens whose section were clean or painted.**

Shear Force

Shear resistance of the steel-concrete interface can be evaluated by either elastic or ultimate strength procedures; both methods have shown good correlation with the test results. The interface shear force resulting from superimposed loads on the composite joist may be computed, using the "elastic approach", by the well known equation:

$$q = \frac{VQ}{I_C} \qquad \text{.......} \qquad (12)$$

Where q = *horizontal shear flow per mm of length (lb./in.)*

V = *vertical shear force at the section (lb.) due to superimposed loads*

Q = *statical moment of the effective concrete in compression (hatched area) about the elastic N.A. of the composite section (in.³)*

I_C = *moment of inertia of the composite joist (in.⁴)*

And $Q = \frac{by}{n}(Y_c - y/2)$ *and* $y = y_c$ *but* $\ngtr t$

Where b = *effective concrete flange width (in.) = lesser of L / 4 or joist spacing*

n = *modular ratio* $= E_s / E_c = 9.2$ *for* $f'_c = 3$ *ksi*

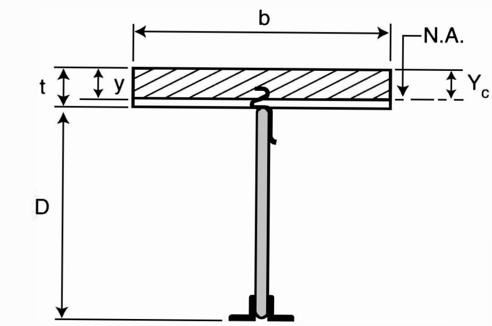
t = *slab thickness (in.)*

Y_c = *depth of N.A. from top of concrete slab*

y = Y_c *when N.A. lies within slab*

= t *when N.A. lies outside slab*

case 1: N.A. within slab $(y = Y_c)$



case 2: N.A. outside slab $(y = t)$

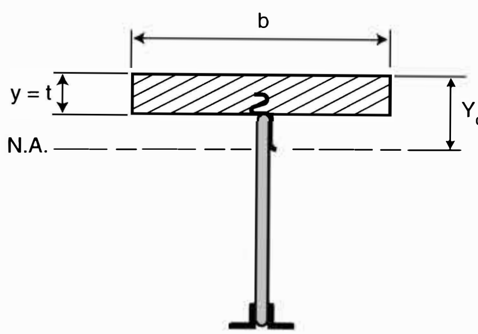


Fig. 9

Design Principles and Calculations - Interface Shear

For a uniformly loaded joist, the average interface shear s, at ultimate load when calculated by ultimate strength principles, would be:

$$s = \frac{2T_u}{L} \quad \text{...} \quad (13)$$

and would represent the average shear force, per unit length, between the points of zero and maximum moment. Some modification to this formula would occur when the strain neutral axis at failure would be located within the section. As this modification is slight and would only occur with bottom chord areas greater that 1.84 sq. in., it is neglected.

Compare the elastic and ultimate approaches:

Since $M_u = T_u d_u$ equation (13) can be rewritten:

$$S = \frac{2M_u}{d_u L} \quad \text{...} \quad (14)$$

Also, for a uniformly distributed load,

$$M_u = \frac{V_u L}{4} \quad \text{...} \quad \textbf{(15)}$$

Subscipts u, are added to equation (12) to **represent the** arbitrary "q" force at failure:

$$q_u = \frac{V_u Q}{I_c} \quad \text{...} \quad (16)$$

Combining (14) and (16) results in:

$$\frac{q_u}{s} = \frac{D_u L}{2M_u} \times \frac{V_u Q}{I_c} \quad \text{..} \quad (17)$$

and, substituting (15) into equation (17)

$$\frac{q_u}{s} = \frac{2QD_u}{I_c} \quad \text{..} \quad (18)$$

The value I_c/Qd_u has been calculated for the various Hambro composite joist sizes, is a constant, and = 1.1. Substituting this in (18),

$$\frac{q_u}{s} = 1.82$$

This verifies that q and s are closely related and that the interface shear force does, in fact, vary from a maximum at zero moment (maximum vertical shear) to a minimum at maximum moment (zero vertical shear). The more recent full scale testings programs have consistently established a failure value for $q_u = 1{,}300$ *lb./in.* and using a safety factor of *1.85*, the safe limiting interface shear, $q = 700$ *lb./in.* This is sometimes converted to "bond stress" $u = q$ / *embedded* perimeter $= q / 7.0$. Hence, the safe limiting "bond stress" $u = 700 / 7 = 100$ *psi.*

As a further safety factor, the bond stresses are usually limited to a value less than *90 psi.*

DESIGN PRINCIPLES AND CALCULATIONS (MINI-JOIST SERIES)

THE MINI-JOIST SERIES

The standard Hambro section, being *3-$^3/_4$ inches* deep, possesses sufficient flexural strength to become the major steel component of the mini-joist series. The three sizes that are currently being used are illustrated in the figure below and spans beyond *8 feet* can be achieved with the heavier SRTC unit. Other sizes are also available.

The composite capacities of the TC, RTC & SRTC units are calculated on the basis of "elastic tee beam analysis". The effective flange width, *b*, equals the lesser of span/4, or joist spacing. With the mini-joist spaced at *4 foot -1-$^1/_4$ inches and 1 6 t = 40 inches*, *b* is dictated by span/4. The calculations are simplified somewhat by using only two values for "*b*", *12" up to 7* foot *-6 inches spans and 24 inches for 8 foot spans.* The load table lists total load capacity in plf.

Full scale tests have demonstrated consistently that shoe plates are not required - the Zee section is simply notched at each bearing end with the lower horizontal portion of the "Zee" becoming the actual bearing surface. **Note that where the non-composite end reaction exceeds *1,000 lbs.* the notched ends are reinforced with a 1$^1/_2$ inch diameter bar 8 inches long.**

This is to prevent the Zee section from "straightening out" at the bearing ends. It is interesting to note that this is not a problem during the composite service stage, even with its higher total loads, as the *2-$^1/_2$* inch slab carries the vertical shears.

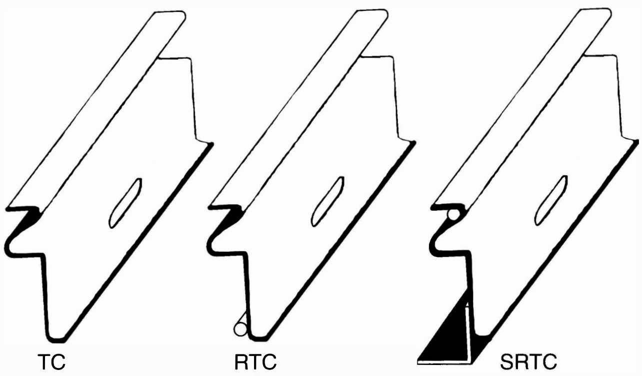


Fig. 10

TABLE 5: Mini-joist H Series Chart Capacity (Maximum Total Unfactored Load in plf)

TYPE	CONDITIONS	PROPERTIES		CLEAR SPAN								
		I	S									
		in.^{4}	in.^{3}	3' - 0"	3' - 6"	4' - 0"	4' - 6"	5' - 0"	6' - 0"	7' - 0"	7' - 6"	8' - 0"
TC	COMPOSITE	2.29	0.60	1198	895	694	550	443				
	NON-COMP.	0.66	0.29	575	427	332	263	213				
RTC	COMPOSITE	5.09	1.31	2627	1949	1519	1215	985	689	514	447	394
	NON-COMP.	1.63	0.75	1502	1125	870	694	566	398	291	254	222
SRTC	COMPOSITE	b = 12" 9.84	2.36									
	NON-COMP.	b = 24" 11.60	2.55						1231	919	804	763
			1.60						845	624	546	497

Design Loads

The engineer of record and the architect of record should specify the joist depth, slab thickness, mesh size and the design loads (dead, live and total load together with special point loads where applicable).

For maximum economy, the Hambro composite joist will be designed to specifically meet these loading requirements. Live load deflection will be limited to $L / 360$.

Example of Joist Identification:

$$\text{Joist Depth} = 16 \text{ in.}$$

$$\begin{aligned} \text{Live Load} &= 40 \text{ lbs. / sq. ft.} \\ \text{Dead Load} &= 60 \text{ lbs. / sq. ft.} \\ \hline \text{Total Load} &= 100 \text{ lbs. / sq. ft.} \end{aligned}$$

Designate joist as H16/410

$$410 = 100 \text{ psf} \times 4.1 \text{ ft. (spacing)}$$

Or simply: H16 with the live, dead and total loads clearly listed on the framing plans.

The Hambro Slab as a Diaphragm

With the increasing use of the Hambro system in earthquake prone areas such as Anchorage, Los Angeles, etc., as well as in high buildings, a testing program was conducted to clearly establish how the Hambro Composite Floor System behaved as a diaphragm in transferring horizontal shears to the supporting structure.

Objective

To study the behavior and strength of the Hambro System acting as a diaphragm, and to determine a suitable structural design method.

Description

Fourteen small-scale test specimens, *2 feet* x *3 feet* were tested in the Structures Laboratory of Carleton University, using the *120 kip / 54.5 M* tonne capacity Tinius Olsen Universal testing machine. The specimens were set up in such a way as to induce extremely high shear forces in the slab, thereby leading to shear failures. The specimens incorporated the following variable conditions:

- Variable slab thicknesses
- Variable concrete strengths
- Variable direction of embedded top chord parallel and perpendicular to direction of applied load
- Variable mesh size

Control specimens which did not have any top chord were used as the basis for comparison.

Close observations were made of each test specimen to determine general behavior, such as cracking and actual failure modes.

Results and Discussion

The test specimens yielded meaningful results – test data correlated very well with the “shear friction” design approach, which is outlined by the ACI (American Concrete Institute) Standard 318. The tests clearly established that the horizontal shear resistance of the slab is dependent **only** on the “available” mesh steel area that passes over the top chord.

The following is a synopsis of the significant test results:

1. Shear friction, i.e. cracking along the top chord, is the dominant mode of failure and always occurs before flexural or diagonal tension failure.
2. Diaphragm buckling, i.e. vertical movement of the slab due to lateral loads, will not occur provided that the joists are prevented from vertical movement at their supports and thus are forced to bend and provide resistance to any out of plane movement of the slab.
3. For shear force transfer perpendicular to the direction of the , the test specimen behaved as if the were not present.
4. The weakest condition is shear force transfer parallel to the direction of the .
5. The calculated shear friction failure load (via ACI 318) is conservative and always less than the actual test specimen failure loads.
6. Drift or lateral movement of the slab can be calculated as the sum of the flexural and shear deflections.
7. Using the shear friction approach, the procedure to design Hambro slab as a diaphragm is the same as a conventional slab.

LATERAL LOAD DISTRIBUTION FOR LINE LOADS

Line loads are often encountered in construction, i.e. a concrete block wall or even a load bearing concrete block wall. It is always desirable to have a floor system that is stiff enough to allow these line loads to be distributed to adjacent joists rather than it be carried by the joist that happens to be directly under it.

The Hambro Composite Floor System provides the designer with this desirable feature.

This was conclusively proven by randomly selecting a sample of five similar adjacent joists in a bay in an apartment structure and line loading the center one.

The joists were *12 inches* deep, had a clear span of *21 feet-3 inches* and a *3 inch* thick slab. The loads were applied using brick pallets. At every load stage, steel strains as well as deflections were measured.

The distribution of load to each of the five joists can be determined by comparing deflections or stresses at similar locations in the five joists under investigation.

Tests have demonstrated that for a line load applied to a typical joist in a bay, the actual distribution of load to that joist is approximately 40% of the applied load. The distribution of load to the adjacent joist on either side is approximately 21% of the applied load and to the next adjacent joist approximately 9% of the applied load.