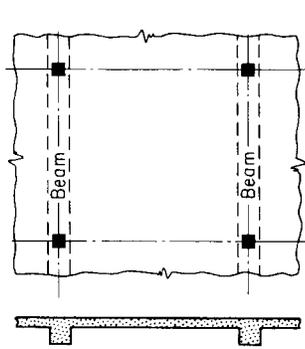
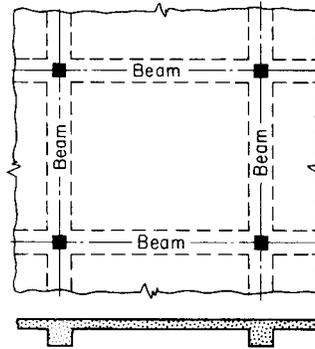


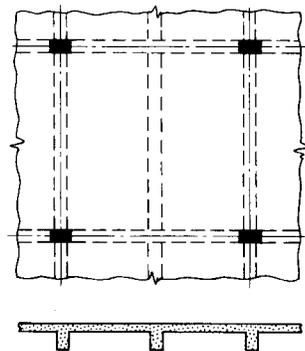
VARIOUS TYPES OF SLABS



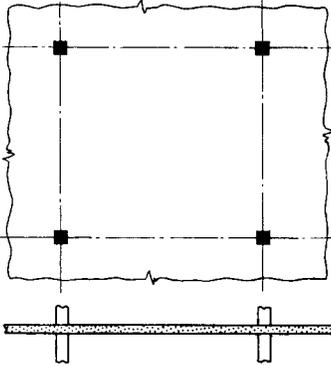
(a) One-way slab



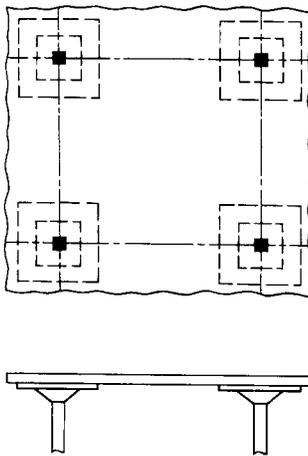
(b) Two-way slab



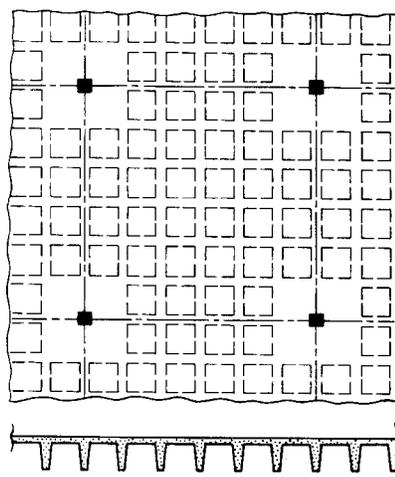
(c) One-way slab



(d) Flat plate slab



(e) Flat slab



(f) Grid slab

CHOICE OF TYPE OF SLAB FLOOR

The choice of type of slab for a particular floor depends on many factors. Economy of construction is obviously an important consideration, but this is a qualitative argument until specific cases are discussed, and is a geographical variable. The design loads, required spans, serviceability requirements, and strength requirements are all important.

For beamless slabs, the choice between a flat slab and a flat plate is usually a matter of loading and span. Flat plate strength is often governed by shear strength at the columns, and for service live loads greater than perhaps 100 lb/ft^2 (4.8 kN/m^2) and spans greater than about 20 to 24 ft (7 to 8 m) the flat slab is often the better choice. If architectural or other requirements rule out capitals or drop panels, the shear strength can be improved by using metal shear heads or some other form of shear reinforcement, but the costs may be high.

Serviceability requirements must be considered, and deflections are sometimes difficult to control in reinforced concrete beamless slabs. Large live loads and small limits on permissible deflections may force the use of large column capitals. Negative-moment cracking around columns is sometimes a problem with flat plates, and again a column capital may be useful in its control.

Deflections and shear stresses may also be controlled by adding beams instead of column capitals. If severe deflection limits are imposed, the two-way slab will be most suitable, as the introduction of even moderately stiff beams will reduce deflections more than the largest reasonable column capital is able to. Beams are also easily reinforced for shear forces.

The choice between two-way and beamless slabs for more normal situations is complex. In terms of economy of material, especially of steel, the two-way slab is often best because of the large effective depths of the beams. However, in terms of labor in building the floor, the flat plate is much cheaper because of the very simple formwork and less complex arrangement of steel. The flat slab is somewhat more expensive in labor than is the flat plate, but the forms for the column capitals are often available as prefabricated units, which can help limit costs. The real cost parameter is the ratio of costs of labor relative to material. Few two-way slabs are built in areas of high labor costs unless there are definite structural reasons, and many are built where steel is the most costly item. Hollow-tile slabs are still built in some places, but only where the cost of both steel and cement is very high relative to labor.

Local customs among builders, designers, and users should not be overlooked when selecting the slab type. There is a natural human tendency to want to repeat what one has previously done successfully, and resistance to change can affect costs. However, old habits should not be allowed to dominate sound engineering decisions.

If a flat plate or flat slab is otherwise suitable for a particular structure, it will be found that there is the additional benefit of minimizing the story height. In areas of absolute height restrictions, this may enable one to have an additional floor for approximately each 10 floors, as compared with a two-way slab with the same clearstory heights. The savings in height lead to other economies for a given number of floors, since mechanical features such as elevator shafts and

pipings are shorter. There is less outside wall area, so wind loadings may be less severe and the building weighs less, which may bring cost reductions in foundations and other structural components. There are other cost savings when the ceiling finishes can be applied directly to the lower surfaces of the slabs.

Beamless slabs will be at a disadvantage if they are used in structures that must resist large horizontal loads by frame action rather than by shear walls or other lateral bracing. The transfer of moments between columns and a slab sets up high local moments, shears, and twisting moments that may be hard to reinforce for. In this situation, the two-way slab is the more capable structure because of the relative ease with which its beams may be reinforced for these forces. In addition, it will provide greater lateral stiffness because of both the presence of the beams and the greater efficiency of the beam-column connections.

The possible choice of a precast one-way floor system, consisting of prestressed concrete members placed side-by-side and spanning between the beams, girders, or walls and generally covered by a cast-in-place concrete topping slab, should not be overlooked.

DESIGN OF ONE-WAY SLABS

The structural action of a one-way slab may be visualized in terms of the deformed shape of the loaded surface. Figure 1 shows a rectangular slab, simply supported along its two opposite long edges and free of any support along the two opposite short edges. If a uniformly distributed load is applied to the surface, the deflected shape will be as shown by the solid lines. Curvatures, and consequently bending moments, are the same in all strips s spanning in the short direction between supported edges, whereas there is no curvature, hence no bending moment, in the long strips l parallel to the supported edges. The surface is cylindrical.

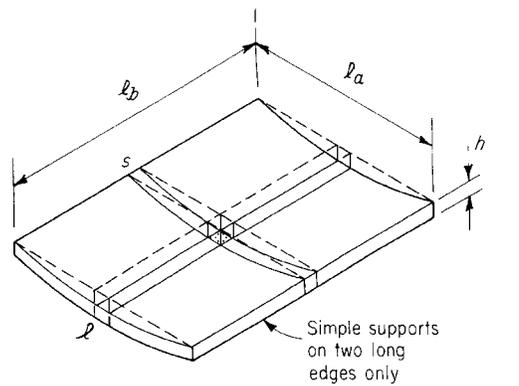


Figure 1. Deflected Shape of Uniformly Loaded One-Way Slab.

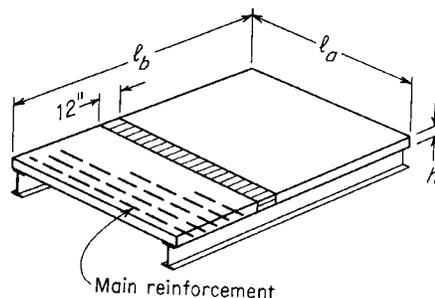


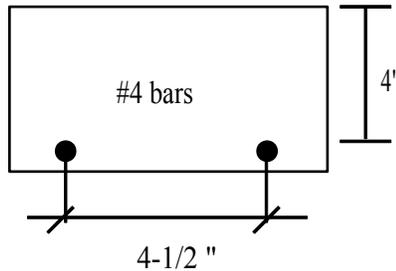
Figure 2. Deflected Shape of Uniformly Loaded One-Way Slab.

For purposes of analysis and design, a unit strip of such a slab cut out at right angles to the supporting beams, as in Figure 2, may be considered as a rectangular beam of unit width, with a depth h equal to the thickness of the slab and a span l_a equal to the distance between supported edges. This strip can then be analyzed by the methods that were used for rectangular beams, the bending moment being computed for the strip of unit width. The load per unit area on the slab becomes the load per unit length on the slab strip. Since all the load on the slab must be transmitted to the two supporting beams, it follows that all the reinforcement should be placed at right angles to these beams, with the exception of any bars that may be placed in the other direction to control shrinkage and temperature cracking. A one-way slab thus consists of a set of rectangular beams side by side.

This simplified analysis, which assumes Poisson's ratio to be zero, is slightly conservative. Actually, flexural compression in the concrete in the direction of l_a will result in lateral expansion in the direction of l_b unless the compressed concrete is restrained. In a one-way slab, this lateral expansion is resisted by adjacent slab strips, which tend to expand also. The result is a slight strengthening and stiffening in the span direction, but this effect is small and can be disregarded.

The ratio of steel in a slab can be determined by dividing the sectional area of one bar by the area of concrete between two successive bars, the latter area being the product of the depth to the center of the bars and the distance between them, center to center. The ratio of steel can also be determined by dividing the average area of steel per foot of width by the effective area of concrete in a 1 ft strip. The average area of steel per foot of width is equal to the area of one bar times the average number of bars in a 1 ft strip (12 divided by the spacing in inches), and the effective area of concrete in a 1 ft (or 12 in.) strip is equal to 12 times the effective depth d .

To illustrate the latter method of obtaining the steel ratio r , assume a 5 in. slab with an effective depth of 4 in., with No. 4 bars spaced 4 1/2 in, center to center.



$$\rho \text{ in slab} = \frac{A_s \text{ of one bar}}{\text{area of concrete between two successive bars}}$$

$$\rho = \frac{\text{average area of steel per ft of width}}{\text{effective area of concrete in 1 ft of strip}}$$

where average area of steel per ft of width = A_s of one bar times average number of bars in one foot.

$$\text{average number of bars} = \frac{12}{4.5} = 2.7$$

$$\text{average steel area in 12 in strip} = 2.7 \times (0.2 \text{ in}^2) = 0.54 \text{ in}^2$$

$$\rho = \frac{0.54}{12 \times 4} = 0.0112$$

or

$$\rho = \frac{0.2}{4.5 \times 4} = 0.0112$$

The spacing of bars that is necessary to furnish a given area of steel per foot of width is obtained by dividing the number of bars required to furnish this area into 12. For example, to furnish an average area of 0.46 in²/ft, with No. 4 bars, requires 0.46 / 0.20 = 2.3 bars per foot; the bars must be spaced not more than 12/2.3 = 5.2 in. center to center. The determination of slab steel areas for various combinations of bars and spacing is facilitated by the following table.

Areas of bars in slabs, in²/ft

Spacing, in.	Bar No.								
	3	4	5	6	7	8	9	10	11
3	0.44	0.78	1.23	1.77	2.40	3.14	4.00	5.06	6.25
3½	0.38	0.67	1.05	1.51	2.06	2.69	3.43	4.34	5.36
4	0.33	0.59	0.92	1.32	1.80	2.36	3.00	3.80	4.68
4½	0.29	0.52	0.82	1.18	1.60	2.09	2.67	3.37	4.17
5	0.26	0.47	0.74	1.06	1.44	1.88	2.40	3.04	3.75
5½	0.24	0.43	0.67	0.96	1.31	1.71	2.18	2.76	3.41
6	0.22	0.39	0.61	0.88	1.20	1.57	2.00	2.53	3.12
6½	0.20	0.36	0.57	0.82	1.11	1.45	1.85	2.34	2.89
7	0.19	0.34	0.53	0.76	1.03	1.35	1.71	2.17	2.68
7½	0.18	0.31	0.49	0.71	0.96	1.26	1.60	2.02	2.50
8	0.17	0.29	0.46	0.66	0.90	1.18	1.50	1.89	2.34
9	0.15	0.26	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	0.13	0.24	0.37	0.53	0.72	0.94	1.20	1.52	1.87
12	0.11	0.20	0.31	0.44	0.60	0.78	1.00	1.27	1.56

Design moments and shears in one-way slabs can be found either by elastic analysis or through the use of the same coefficients as used for beams (ACI 8.3.3). If the slab rests freely on its supports, the span length may be taken equal to the clear span plus the depth of the slab but need not exceed the distance between centers of supports, according to ACI Code 8.7.1. In general, center-to-center distances should be used in continuous slab analysis, but a reduction is allowed in negative moments to account for support width.

For slabs with clear spans not more than 10 ft that are built integrally with their supports, ACI Code 8.7.4 permits analysis as a continuous slab on knife-edge supports with spans equal to the clear spans and the width of the beams otherwise neglected. If moment and shear coefficients are used, computations should be based on clear spans.

One-way slabs are normally designed with tensile steel ratios well below the maximum permissible value to have tension failure. Typical steel ratios range from about 0.004 to 0.008. This is partially for reasons of economy, because the saving in steel associated with increasing

the effective depth more than compensates for the cost of the additional concrete, and partially because very thin slabs with high steel ratios would be likely to permit large deflections. Thus, flexural design may start with selecting a relatively low steel ratio, say about $0.20\rho_b$, setting $M_u = \phi M_n$ and solving for the required effective depth d , given that $b = 12$ in. for the unit strip.

ACI Code 9.5.2 specifies the minimum thickness in Table 1 for non-prestressed slabs of normal weight concrete ($w_c = 145$ pcf) using Grade 60 reinforcement, provided that the slab is not supporting or attached to construction that is likely to be damaged by large deflections. Lesser thicknesses may be used if calculation of deflections indicates no adverse effects. For concretes having unit weight w_c , in the range from 90 to 120 pcf, the tabulated values should be multiplied by $(1.65 - 0.005w_c)$, but not less than 1.09. For reinforcement having a yield stress f_y other than 60,000 psi, the tabulated values should be multiplied by $(0.4 + f_y/100,000)$. Slab deflections may be calculated, if required, by the same methods as for beams.

Table 1. Minimum thickness h of nonprestressed one-way slabs

Simply supported	1/20
One end continuous	1/24
Both ends continuous	1/28
Cantilever	1/10

Shear will seldom control the design of one-way slabs, particularly if low tensile steel ratios are used. It will be found that the shear capacity of the concrete, ϕV_c will almost without exception be well above the required shear strength V_u at factored loads.

Practical Considerations

The total slab thickness h is usually rounded to the next higher 1/4 in. for slabs up to 6 in. thickness, and to the next higher 1/2 in. for thicker slabs. The concrete protection below the reinforcement should follow the requirements of the ACI Code 7.7.1, calling for 2 in. below the bottom of the steel. In a typical slab, 1 in. below the center of the steel may be assumed. The

lateral spacing of the bars, except those used only to control shrinkage and temperature cracks, should not exceed 3 times the thickness h or 18 in., whichever is less, according to ACI Code 7.6.5. Generally, bar size should be selected so that the actual spacing is not less than about 1.5 times the slab thickness, to avoid excessive cost for bar fabrication and handling. Also, to reduce cost, straight bars are usually used for slab reinforcement.

TEMPERATURE AND SHRINKAGE REINFORCEMENT

Concrete shrinks as the cement paste hardens. It is advisable to minimize such shrinkage by using concretes with the smallest possible amounts of water and cement compatible with other requirements, such as strength and workability, and by thorough moist-curing of sufficient duration. However, no matter what precautions are taken, a certain amount of shrinkage is usually unavoidable. If a slab of moderate dimensions rests freely on its supports, it can contract to accommodate the shortening of its length produced by shrinkage. Usually, however, slabs and other members are joined rigidly to other parts of the structure and cannot contract freely. This results in tension stresses known as shrinkage stresses. A decrease in temperature relative to that at which the slab was poured, particularly in outdoor structures such as bridges, may have an effect similar to shrinkage. That is, the slab tends to contract and if restrained from doing so becomes subject to tensile stresses.

Since concrete is weak in tension, these temperature and shrinkage stresses are likely to result in cracking. Cracks of this nature are not detrimental, provided their size is limited to what are known as hairline cracks. This can be achieved by placing reinforcement in the slab to counteract contraction and distribute the cracks uniformly. As the concrete tends to shrink, such reinforcement resists the contraction and consequently becomes subject to compression. The total shrinkage in a slab so reinforced is less than that in one without reinforcement; in addition, whatever cracks do occur will be of smaller width and more evenly distributed by virtue of the reinforcement.

In one-way slabs the reinforcement provided for resisting the bending moments has the desired effect of reducing shrinkage and distributing cracks. However, as contraction takes place equally in all directions, it is necessary to provide special reinforcement for shrinkage and temperature

contraction in the direction perpendicular to the main reinforcement. This added steel is known as temperature or shrinkage reinforcement, or distribution steel.

Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement should be provided in a structural slab in which the principal reinforcement extends in one direction only. ACI Code 7.12.2 specifies the minimum ratios of reinforcement area to gross concrete area but in no case shall such reinforcing bars be placed farther apart than 5 times the slab thickness or more than 18 in. In no case is the steel ratio to be less than 0.0014.

The steel required by the ACI Code for shrinkage and temperature crack control also represents the minimum permissible reinforcement in the span direction of one-way slabs; the usual minimums for flexural steel do not apply.

Example. One-way slab design. A reinforced concrete slab is built integrally with its supports and consists of two equal spans, each with a clear span of 15 ft. The service live load is 116 psf and 4000 psi concrete is specified for use with steel of yield stress equal to 60,000 psi. Design the slab, following the provisions of the ACI Code.

Solution. The thickness of the slab is first estimated, based on the minimum thickness of Table 1 we have

$$\frac{l}{28} = \frac{15 \times 12}{28} = 6.43 \text{ in}$$

A trial thickness of 6.50 in will be used, for which the weight is $150 \times 6.50 / 12 = 81$ psf. The specified live load and computed dead load are multiplied by the ACI load factors:

$$\begin{aligned} \text{Dead load} &= 81 \times 1.2 = 97.2 \quad \text{psf} \\ \text{Live load} &= 116 \times 1.6 = \underline{185.6} \quad \text{psf} \\ \text{Total} &= 283 \text{ psf} \end{aligned}$$

For this case, design moments at critical sections may be found using the ACI moment coefficients:

$$\begin{aligned} \text{At interior support: } -M &= \frac{1}{9}(0.238)(15)^2 = 7.06 \quad \text{ft-kips} \\ \text{At midspan: } +M &= \frac{1}{14}(0.238)(15)^2 = 4.53 \quad \text{ft-kips} \\ \text{At exterior support: } -M &= \frac{1}{24}(0.238)(15)^2 = 2.65 \quad \text{ft-kips} \end{aligned}$$

If that maximum value of steel ratio were actually used to make sure tension failure, the minimum required effective depth, controlled by negative moment at the interior support.

The effective depth of

$$d = 6.50 - 1.00 = 5.50$$

At the interior support, if the stress-block depth $a = 1.00$ in., the area of steel required per foot of width in the top of the slab is

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{7.06 \times 12}{0.90 \times 60 \times (5.5 - 1/2)} = 0.31 \text{ in}^2$$

Checking the assumed depth a we get

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.31 \times 60}{0.85 \times 4 \times 12} = 0.46 \text{ in}$$

A second trial will be made with $a = 0.46$ in. Then

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{7.06 \times 12}{0.90 \times 60 \times (5.5 - 0.46/2)} = 0.30 \text{ in}^2$$

for which

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.46 \times 60}{0.85 \times 4 \times 12} = 0.45 \text{ in}$$

No further revision is necessary.

Check to make sure we have tension failure:

$$a = 0.45 \text{ in} \rightarrow c = \frac{a}{\beta_1} = \frac{0.45}{0.85} = 0.53 \text{ in}$$

$$\frac{c}{d} = \frac{0.53}{5.5} = 0.096 < \epsilon_t = 0.005 \text{ Tension Failure (see Page 100 of ACI)}$$

At other critical-moment sections, it will be satisfactory to use the same lever arm to determine steel areas, and

$$\text{at mid span } A_s = \frac{4.53 \times 12}{0.90 \times 60 \times 5.27} = 0.19 \text{ in}^2$$

$$\text{at exterior support } A_s = \frac{2.65 \times 12}{0.90 \times 60 \times 5.27} = 0.11 \text{ in}^2$$

The minimum reinforcement is that required for control of shrinkage and temperature cracking. This is

$A_s = 0.0018 \times 12 \times 6.5 = 0.14 \text{ in}^2$ per 12 in. strip. This requires a small increase in the amount of steel used at the exterior support. The factored shear force at a distance d from the face of the interior support is

$$V_u = 1.15 \times \frac{283 \times 15}{2} - 283 \times \frac{5.5}{12} = 2310 \text{ lbs}$$

the nominal shear strength of the concrete slab is

$$V_n = V_c = 2\sqrt{f'_c}bd = 2\sqrt{4000} \times 12 \times 5.5 = 8350 \text{ lbs}$$

Thus, the design strength of the concrete slab, $\phi V_c = 0.75 \times 8350 = 6260 \text{ lbs}$, is well above the required strength in shear of $V_u = 2310 \text{ lbs}$.

The required tensile steel areas may be provided in a variety of ways, but whatever the selection, due consideration must be given to the actual placing of the steel during construction. The arrangement should be such that the steel can be placed rapidly with the minimum of labor costs even though some excess steel is necessary to achieve this end.

Two possible arrangements are shown in Figure 3. In Figure 3a bent bars are used, while in Figure 3b all bars are straight.

In the arrangement of Figure 3a, No. 4 bars at 10 in. furnish 0.24 in^2 of steel at midspan, slightly more than required. If two-thirds of these bars are bent upward for negative

reinforcement over the interior support, the average spacing of such bent bars at the interior support will be $(10 + 20)/2 = 15$ in. Since an identical pattern of bars is bent upward from the other side of the support, the effective spacing of the No. 4 bars over the interior support is 7.5 in. This pattern closely satisfies the required steel area of 0.30 in^2 per foot width of slab over the support. The bars bent at the interior support will also be bent upward for negative reinforcement at the exterior support, providing reinforcement equivalent to No. 4 bars at 15 in., or 0.16 in^2 of steel.

Note that it is not necessary to achieve uniform spacing of reinforcement in slabs, and that the steel provided can be calculated safely on the basis of average spacing as in the example. Care should be taken to satisfy requirements for both minimum and maximum spacing of principal reinforcement, however.

The arrangement of Figure 3b uses only straight bars. Although it is satisfactory according to the ACI Code (since the shear stress does not exceed two-thirds of that permitted), cutting off the shorter positive and negative bars as shown leads to an undesirable condition at the ends of those bars, where there will be concentrations of stress in the concrete. The design would be improved if the negative bars were cut off at 3 ft from the face of the interior support rather than 2 ft 6 in. as shown, and if the positive steel were cut off at 2 ft 2 in. rather than at 2 ft 11 in. This would result in an overlap of approximately $2d$ of the cut positive and negative bars.

The required area of steel to be placed normal to the main reinforcement for purposes of temperature and shrinkage crack control is 0.14 in^2 . This will be provided by No. 4 bars at 16 in. spacing, placed directly on top of the main reinforcement in the positive-moment region and below the main steel in the negative-moment zone.

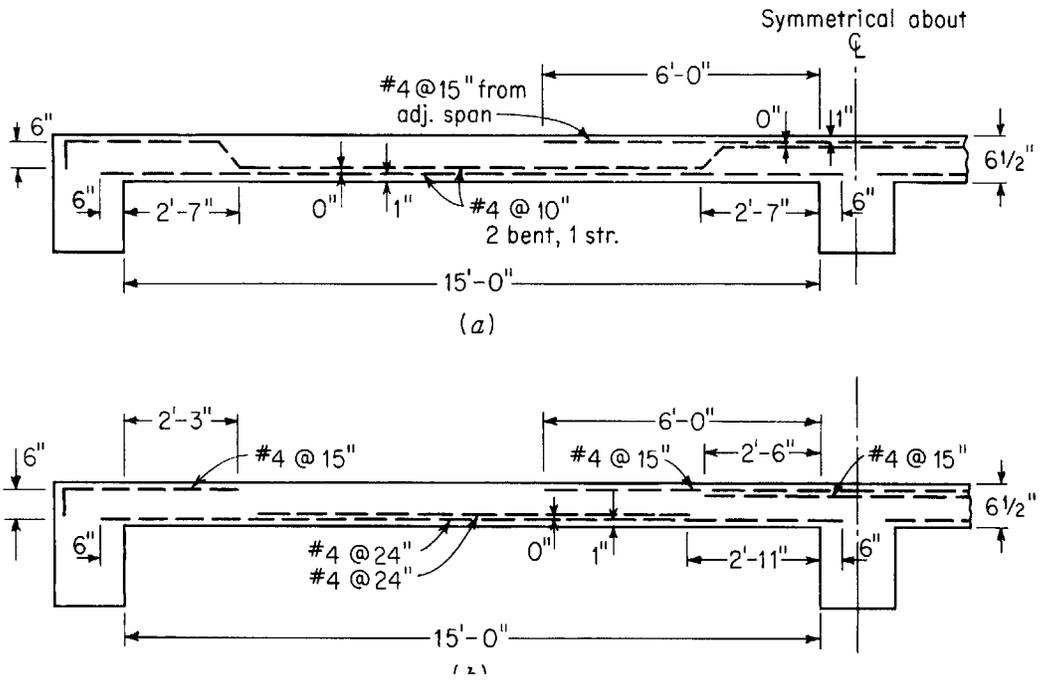


Figure 3. One-way Slab Design Example.