SOME PRACTICAL ASPECTS OF STRUCTURAL DESIGN FOR BUILDINGS

by

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Preface

The principle of standardization of methods of designing individual members may be advantageously applied to structural frames, though the completed buildings may differ widely. The structural members, of course, can not be made uniform for all buildings nor would that be desirable or even possible. The practice of standardizing general methods of design and details however will save both time and money for the client. Elements of structural theory should not be disregarded to achieve this end, but rather combined with good practice.

The majority of the structural problems in the usual building can readily be solved by the use of tables. The designer should always be familiar with the derivation of the mathematical formulas necessary to devise these tables so their limitations can be appreciated. The recent standardization of structural shapes and elimination of unnecessary bar sizes has permitted the tables to be reduced considerably. The tables, however, do not cover all the combinations of stresses permitted by the various building codes, but follow the recommendations of certain national societies and the United States Department of Commerce. The matter of assumptions of loads will not be discussed since that is governed by local building codes.

The manner of presenting structural designs on drawings differs somewhat among engineers, but those shown are generally accepted as good practice. The details perhaps appear obvious but are intended to illustrate common faults and the preferred solutions. Complicated structural designs are combinations of many relatively simple details therefore careful attention should be given to all the minor ones.

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The practical aspects of the ordinary structural design are so numerous that each structure is an individual problem. The final result depends on the skill of the engineer and the nature of the projects on which he has been engaged. Structural designs are therefore rather individual. The views presented in this thesis can only cover the most common problems and there will probably be some difference of opinion as to the degree of practicability. The ideal condition is the agreement of practice with theory but this is not always practical.

The progress of building construction is continually upsetting any previously accepted practice and cost of materials and labor has a decided effect on structural design. The present trend is to reduce field work and substitute shop fabricated members. The benefits are obvious but the most significant one is uniformity of product. Shop fabrication has indeed progressed to the extent that certain classes of buildings are entirely fabricated and are simply erected in the field. These buildings however are only semi-permanent warehouses and small factories. The majority of buildings fortunately for architects and engineers still must be individually designed and are not adaptable to standardization to such a degree.

The following tables have been devised principally for design purposes, but may also be used for investigating stresses in existing structural members. The data is presented in tabular form rather than curves since there is less possibility of error in practical use. The stresses adopted for these tables may differ from the requirements of some building codes, but, are in general use by most designers. The structural steel industry has agreed on a certain stress which is widely accepted by building codes. Assumptions governing the structural design have been eliminated and no attempt is made to restrict the designer's ingenuity. The use of tables should actually free his mind from tedious details and afford the time to simplify the structural design. The tables have been arranged in the usual order of preparing a structural design.

The table on page seven gives the allowable unit stresses of steel columns subject to an axial load and is based on the formula

$$\frac{P}{A} = \frac{18{,}000}{1 + \frac{L}{18{,}000r}}$$

which is presented by the American Institute of Steel Construction. The application of the table is quite simple and the following example shows its use in actual design. The axial load is assumed to be two hundred fifty thousand pounds and the unsupported length is fourteen feet.

ALLOWABLE UNIT STRESSES FOR STEEL COLUMNS

Primary Members	L/r	60.	61.	62.	63.	64.	65.	66.	67.	68.	69.
	.0	15000.	14916.	14832.	14748.	14663.	14578.	14493.	14407.	14321.	14235.
	.5	14958.	14874.	14790.	14706.	14621.	14536.	14450.	14364.	14278.	14192.
	L/r	70.	71.	72.	73.	74.	75.	76.	77.	78.	79.
	.0	14148.	14062.	13975.	13888.	13801.	13714.	13627	13540.	13453.	13366.
	.5	14105.	14019.	13932.	13845.	13758.	13671.	13584.	13497.	13410.	13323.
	L/r	80.	81.	82.	83.	84.	85.	86.	87.	88.	89.
	.0	13279.	13192.	13105.	13018.	12931.	12844.	12758.	12672.	12585.	12500.
	.5	13236.	13149.	13062.	12975	12888.	12801.	12715.	12629.	12543.	12457.
	L/r	90.	91.	92.	93.	94.	95.	96.	97.	98.	99.
	.0	12414.	12328.	12243.	12158.	12073.	11989.	11905.	11821.	11737.	11654.
	.5	12371.	12286.	12201.	12116.	12031.	11947.	11863.	11779.	11696.	11613.
	L/r	100.	101.	102.	103.	104.	105.	106.	107.	108	109.
	.0	11571.	11489.	11407.	11325.	11244	11163.	11082.	11002.	10922.	10843.
	.5	11530.	11448.	11366.	11285.	11204.	11123.	11042.	10962	10883	10804.
	L/r	110.	111.	112.	113.	114.	115.	116.	117.	118.	119.
	.0	10764.	10686.	10608.	10530.	10453.	10376.	10300.	10224.	10149.	10074.
	.5	10725.	10647.	10569.	10492.	10415.	10338.	10262.	10187.	10112.	10037.
Secondary Members	L/r	120	121.	122.	123.	124.	125.	126.	127.	128.	129.
	.0	10000.	9926.	9853.	9780.	9708.	9636.	9565.	9494.	9424.	9354.
	L/r	130.	131.	132.	133.	134.	135.	136.	137.	138.	139.
	.0	9284.	9215.	9146.	9078.	9011.	8944.	8878.	8812.	8747.	8682.
	L/r	140.	141.	142.	143.	144.	145.	146.	147.	148.	149.
	.0	8617.	8553.	8489.	8426.	8364.	8302.	8241.	8180.	8120.	8060.
	L/r	150.	151.	152.	153.	154	155.	156.	157.	158.	159.
	.0	8000.	7941.	7882.	7824	7767.	7710.	7653.	7597.	7541.	7486.
	L/r	160.	161.	162.	163.	164.	165.	166	167.	168.	169.
	.0	7431.	7377.	7323.	7270.	7217.	7164.	7112.	7060.	7009.	6958.
	L/r	170.	171.	172.	173.	174.	175.	176.	177.	178.	179.
	.0	6908.	6858.	6808.	6759.	6711.	6663.	6615	6568.	6521.	6475.
	L/r	180.	181.	182.	183.	184.	185.	186.	187.	188.	189.
	.0	6429.	6383.	6338.	6293.	6248.	6204.	6160.	6117.	6074.	6031.
	L/r	190.	191.	192.	193.	194.	195.	196.	197.	198.	199.
	.0	5989.	5947.	5905.	5864.	5823.	5783.	5743.	5703.	5664.	5625.

SAFE LOADS IN THOUSANDS OF POUNDS

	d	W	A_s	A_c	Unsupported length in feet														
					6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Light weight	3½	13	1.27	8.35	26.1	24.2	22.2	20.3	18.3	16.4	14.5								
	4	17	1.63	10.94	35.6	33.4	31.2	29.0	26.8	24.6	22.4	20.2							
Heavy weight	3½	15	2.23	7.39	37.9	35.1	32.3	29.4	26.7	24.0									
	4	20	2.68	9.89	49.2	46.1	43.1	40.1	37.0	33.9	30.9	27.9							
	4½	24	3.17	12.73	61.8	58.5	55.3	52.0	48.8	45.5	42.3	39.0	35.8	32.5					
	5	29	3.69	15.95	75.6	72.0	68.6	65.2	61.7	58.2	54.7	51.3	47.8	44.3	40.9	37.4			
	5½	36	4.30	20.01	92.1	88.3	84.6	80.8	77.1	73.3	69.6	65.8	62.1	58.3	54.6	50.8	47.1	43.3	
	6⅝	49	5.58	28.89	128.3	124.2	120.0	115.8	111.7	107.5	103.4	99.2	95.0	90.9	86.7	82.6	78.4	74.2	70.1
	7⅝	64	6.92	38.74	166.0	161.4	156.9	152.3	147.8	143.2	138.6	134.1	129.7	125.0	120.5	115.9	111.4	106.8	102.3
	8⅝	81	8.40	50.03	211.1	206.1	201.1	196.1	191.0	186.0	181.0	175.9	170.9	165.9	160.8	155.8	150.8	145.8	140.7
	9⅝	100	9.97	62.79	259.2	253.8	248.3	242.8	237.4	231.0	226.5	221.0	215.6	210.1	204.6	199.2	193.7	188.3	182.8
	10¾	123	11.91	78.86	319.1	313.1	307.2	301.3	295.4	289.4	283.5	277.6	271.6	265.7	259.7	253.8	247.9	241.9	236.0
	12¾	169	14.58	113.10	421.9	415.4	408.8	402.3	395.8	389.2	382.8	376.2	369.7	363.2	356.7	350.1	343.6	337.1	330.6

The following empirical formula was used in computing the safe loads for the above concrete filled pipe columns.

$P = (A_c + 12 A_s)(1600 - 24 \frac{L}{d})$ Limit of length equals 40 diameters.

P = Safe load in pounds.

A_s = Area of steel in square inches.

A = Area of concrete in square inches.

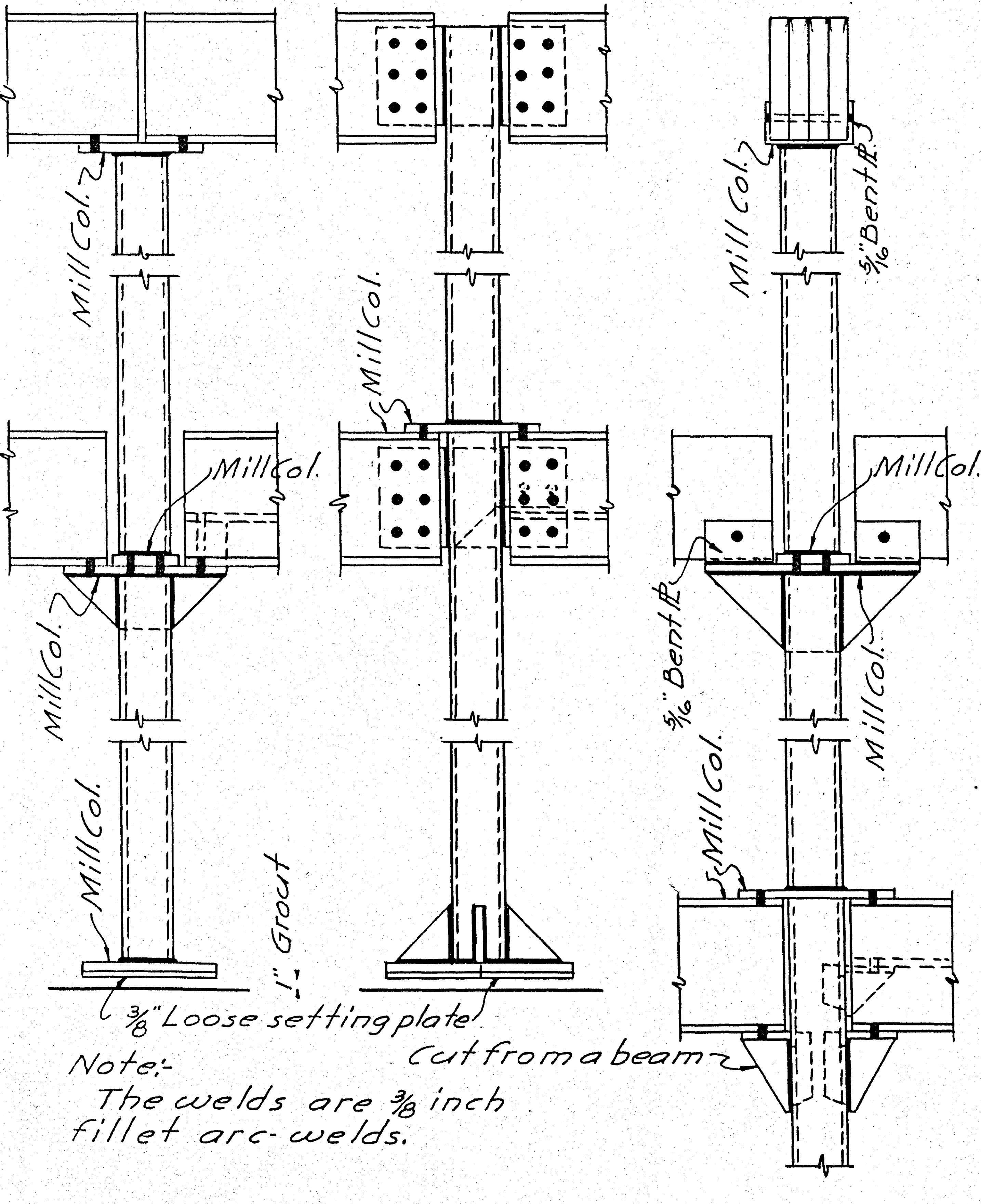
L = Unsupported length of column in inches.

d = Outside diameter of column in inches.

W = Weight of column in pounds a lineal foot.

PIPE COLUMNS
(Filled with Concrete)

PIPE COLUMN CONNECTIONS



Note:-
The welds are 3/8 inch fillet arc-welds.

Assume 10" H64# least $r = 2.38$ $A_s = 18.81$ $\frac{l}{r} = \frac{168}{2.38} = 70.5$ $f_s = 14105$ $P = 14105 \times 18.81 = 266{,}000\#$. The column first assumed is adequate.

The table on page eight covers the safe axial loads for concrete filled pipe column and is computed from an empirical formula based on laboratory tests. Proper column sizes can be selected directly from the table if the axial load and the unsupported length is known. The section of the shaft if ideal and this type of column has been gaining a wide use since the development of arc welding. The details on page nine illustrate several applications of this type of column. The section of the shaft is relatively small considering the carrying capacity and is popular in small buildings since it can readily be concealed in the partition.

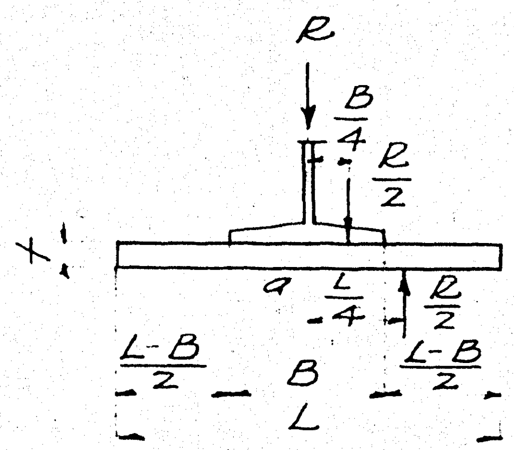
The table on page eleven covers the load value of wall bearing plates to be placed at the end of steel beams supported by walls. Plate sizes can readily be selected if the end reactions of the beams and the wall material is known. The width of the plate will depend on the thickness of the wall or pilaster. The edge of the plate should be flush with the inside face of masonary and the opposite edge of plate four to five inches from the exterior face. The plate may extend entirely through interior brick walls. The end of beams should rest on the entire width of the plate. The thickness of the plate can be determined by the tables on pages twelve and thirteen.

LOAD VALUE OF BEARING PLATES

Size	Area	Unit Load on Masonry		
		100.# □"	250.# □"	500.# □"
4" x 6"	24.□"	2,400.#	6,000.#	12,000.#
4 x 8	32.	3,200.	8,000.	16,000.
6 x 6	36.	3,600.	9,000.	18,000.
6 x 8	48.	4,800.	12,000.	24,000.
6 x 10	60.	6,000.	15,000.	30,000.
6 x 12	72.	7,200.	18,000.	36,000.
8 x 8	64.	6,400.	16,000.	32,000.
8 x 10	80.	8,000.	20,000.	40,000.
8 x 12	96.	9,600.	24,000.	48,000.
8 x 14	112.	11,200.	28,000.	56,000.
8 x 16	128.	12,800.	32,000.	64,000.
10 x 10	100.	10,000.	25,000.	50,000.
10 x 12	120.	12,000.	30,000.	60,000.
10 x 14	140.	14,000.	35,000.	70,000.
10 x 16	160.	16,000.	40,000.	80,000.
10 x 18	180.	18,000.	45,000.	90,000.
10 x 20	200.	20,000.	50,000.	100,000.
12 x 12	144.	14,400.	36,000.	72,000.
12 x 14	168.	16,800.	42,000.	84,000.
12 x 16	192.	19,200.	48,000.	96,000.
12 x 18	216.	21,600.	54,000.	108,000.
12 x 20	240.	24,000.	60,000.	120,000.
12 x 22	264.	26,400.	66,000.	132,000.
12 x 24	288.	28,800.	72,000.	144,000.

100.# Load bearing tile (Portland cement mortar)
250.# Brick (Portland cement mortar)
500.# Concrete (2000.# on 28 day test)

BEARING PLATES



$$M_a = \frac{L}{4} \times \frac{R}{2} - \frac{B}{4} \times \frac{R}{2} = \frac{LR}{8} - \frac{BR}{8} = \frac{R}{8}(L-B)$$

$$R = \omega WL$$

$$M_a = \frac{\omega WL(L-B)}{8} \qquad M_a = fS \qquad S = \frac{Wt^2}{6}$$

$$\therefore M_a = \frac{fWt^2}{6} = \frac{\omega WL(L-B)}{8}$$

$$\frac{ft^2}{3} = \frac{\omega L(L-B)}{4} \qquad L(L-B) = \frac{4ft^2}{3\omega}$$

$$t^2 = \frac{3\omega L(L-B)}{4f}$$

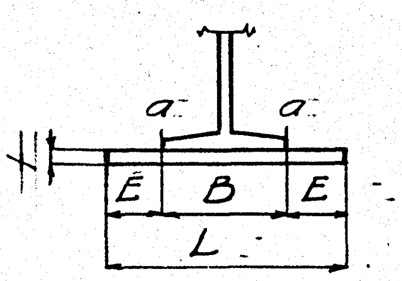
$$t = \sqrt{\frac{3\omega L(L-B)}{4f}}$$

VALUES OF "L(L-B)"

ω	Thickness of plate in inches													
	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	2
100.	33.8	60.0	93.8	135.	184.	240.	304.	376.	454.	450.	635.	735.	845.	960.
250.	13.5	24.0	37.6	54.0	73.6	96.0	122.	150.	182.	216.	254.	294.	338.	384.
500.	6.75	12.0	18.8	27.0	36.8	48.0	61.0	75.0	91.0	108.	127.	147.	169.	192.

ω = 100.# Load bearing tile (Portland cement mortar)
ω = 250.# Brick (Portland cement mortar)
ω = 500.# Concrete (2000.# on 28 day test)

BEARING PLATES



R = Total load on plate in pounds.
L = Length of plate in inches.
W = Width of plate in inches.
B = Width of beam flange in inches.
A = Area of plate in square inches.
w = Allowable pressure on the masonary a square inch.
t = Thickness of plate in inches.
f_s = 18,000. pounds a square inch.
M_a = Moment for one inch width of plate $= w \times E \times \frac{E}{2}$

$$M_a = \frac{wE^2}{2}$$

$$S = \frac{M}{f_s} = \frac{wE^2}{36000.} \qquad S = \frac{t^2}{6} \quad \text{(One inch width of plate)}$$

$$t^2 = \frac{6wE^2}{36000.} = \frac{wE^2}{6000} \qquad t = \sqrt{\frac{wE^2}{6000.}}$$

VALUES OF "E" IN INCHES

w	Thickness of plate in inches													
	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	2
100.	3	3 7/8	4 7/8	5 7/8	6 3/4	7 3/4	8 3/4	9 3/4	10 5/8	11 5/8	12 5/8	13 5/8	14 1/2	15 1/2
250.	1 7/8	2 1/2	3 1/8	3 3/4	4 1/4	4 7/8	5 1/2	6 1/8	6 3/4	7 3/8	8	8 5/8	9 1/4	9 7/8
500.	1 3/8	1 3/4	2 1/8	2 5/8	3	3 1/2	3 7/8	4 3/8	4 3/4	5 1/4	5 5/8	6	6 1/2	7

w = 100.# Load bearing tile (Portland cement mortar).
w = 250.# Brick (Portland cement mortar)
w = 500.# Concrete (2000.# on 28 day test)

The second table of the last two is more widely accepted and is the simpler to use. The thickness can be selected directly if the projection of the plate beyond the flange and the unit beaming stress on the masonary is known.

The individual structural steel members are fastened together by either riveting, bolting or welding. The rivets have recently been classified as power driven and hand driven instead of shop or field driven. The table on page fifteen gives the stress value of rivets and bolts under various conditions. The designer need only know the stress in the member and the type of connection to determine the number of rivets or bolts required for various diameters. The diameter of rivets or bolts should be uniform if possible throughout the structure. The connection should be designed so the rivet is not subject to bending stresses but shear and bearing only. The table on page sixteen gives the recently accepted values for weld fillet and this type of connection is gaining favor. The weld must be proportioned so that the stress in the member attached is axial. The dependibility of electrical welds depends considerably on the skill of the operator and for that reason many designers are hesitating to adopt welding.

The usual design of a truss assumes that the stresses are concurrent, yet it is surprising how often this assumption is ignored when the joints are de-

RIVET VALUES

POWER DRIVEN RIVETS AND TURNED BOLTS IN REAMED HOLES

UNIT SHEAR 13500. SINGLE BEARING 24000.

Rivet size	Single shear	Thickness of plate 3/16"	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"
1/2"	2650.	2250.								
5/8"	4140.	2810.	3750.							
3/4"	5960.	3380.	4500	5630.						
7/8"	8120.	3940.	5250.	6560.	7880.					
1"	10600.	4500.	6000.	7500.	9000.	10500.				

UNIT SHEAR 13500. DOUBLE BEARING 30000.

Rivet size	Double shear	Thickness of plate 3/16"	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"
1/2"	5300.	2810.	3750.	4690.						
5/8"	8280.	3520.	4690.	5860.	7030					
3/4"	11930.	4220.	5630.	7030.	8440.	9840.	11240.			
7/8"	16240.	4920.	6560.	8200.	9840.	11480.	13130.	14770.		
1"	21200.	5630.	7500.	9380.	11250.	13130.	15000.	16880.	18750.	20630.

HAND DRIVEN RIVETS AND UNFINISHED BOLTS

UNIT SHEAR 10000. SINGLE BEARING 16000.

Rivet size	Single shear	Thickness of plate 3/16"	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"
1/2"	1960.	1500.								
5/8"	3070.	1880.	2500.							
3/4"	4420.	2250.	3000.	3750.						
7/8"	6010.	2630.	3500.	4380.	5250.					
1"	7850.	3000.	4000.	5000.	6000.	7000.				

UNIT SHEAR 10000. DOUBLE BEARING 20000.

Rivet size	Double shear	Thickness of plate 3/16"	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"
1/2"	3930.	1880.	2500.	3130.	3750.					
5/8"	6140.	2340.	3130.	3910.	4690.	5470.				
3/4"	8840.	2810.	3750.	4690.	5630.	6560.	7500.	8440.		
7/8"	12030.	3280.	4380.	5470.	6560.	7660.	8750.	9850.	10940.	12030.
1"	15710.	3750.	5000.	6250.	7500.	8750.	10000.	11250.	12500.	13750.

STRUCTURAL ARC WELDING

DESIGN The size of any weld shall be the width of its contact with the material to be welded. The following fillet weld values shall be used in tension and in shear for arc-welded connections:

Full fillet weld	Pounds a lineal inch
3/16 inch	1500.
1/4 inch	2000.
5/16 inch	2500.
3/8 inch	3000.
1/2 inch	3500.

The above values may be increased 20 percent for compression. The value of electric rivets shall be the value of the fillet weld used multiplied by the perimeter of the center of section of the fillet weld. The connections should be as direct as possible and tack welds shall not be less than two inches long.

MACHINES The welding machines employed shall be of an approved direct current, low voltage type, of a make that has been previously used successfully on structural welding. The amperage capacity shall be sufficient to overcome line drop and to give adequate welding heat.

OPERATORS The operators shall be skilled and experienced men employed regularly, not occasionally, on structural arc-welding.

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tailed. The figures on pages eighteen, nineteen, and twenty illustrate the common cases of eccentric joints. The stresses of the joints in figures one and two are not concurrent and subject the members to bending in addition to the direct stress. The additional stresses due to the bending often are relatively high. The connection shown by figures three and four are much better but of the two the last one is preferred. The cases are rare when eccentric joints in trusses cannot be avoided, but then the extra stress should be considered in the design either by thickening the gusset plate and adding rivets or by increasing the size of the members. Figures five and six on page nineteen illustrate rather uncommon cases of eccentric joints, but occasionally one is found on a drawing. The preferred connection is the one shown by figure seven. The knee brace connection shown by figure eight is very common but should be made as shown by figure nine. The figures ten and eleven on page twenty again show examples of ecdentric connections and the preferred connection is shown by figures eleven and thirteen. The members of a truss should be symmetrical in section to avoid eccentric connections. The joint shown by figure fourteen is eccentric and subjects the single angle to bending and the two top angles to a twist. The ideal connection is illustrated by figure fifteen. A member composed of two angles is preferred to a single angle and the minimum size member is two angles two and one-

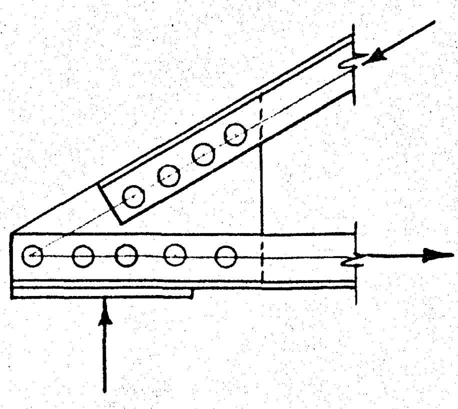


FIGURE 1

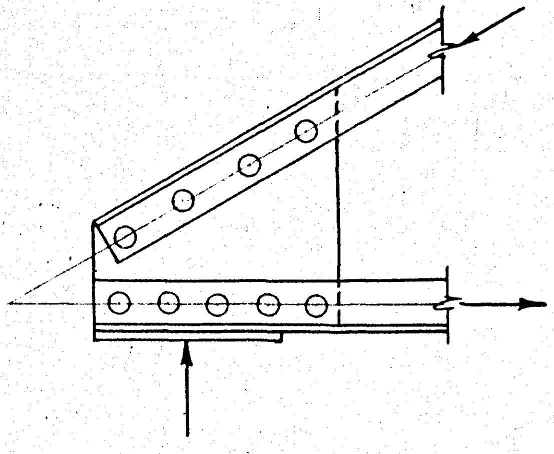


FIGURE 2

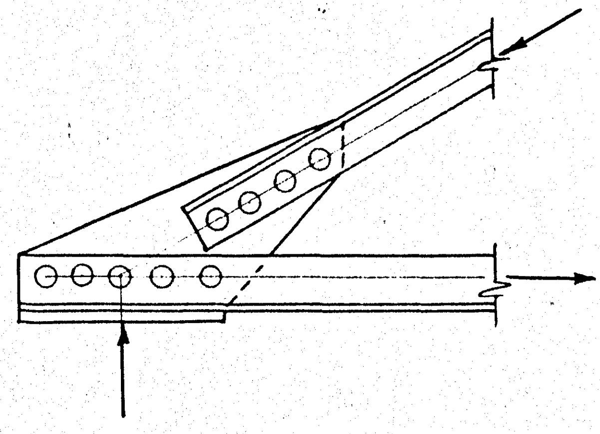


FIGURE 3

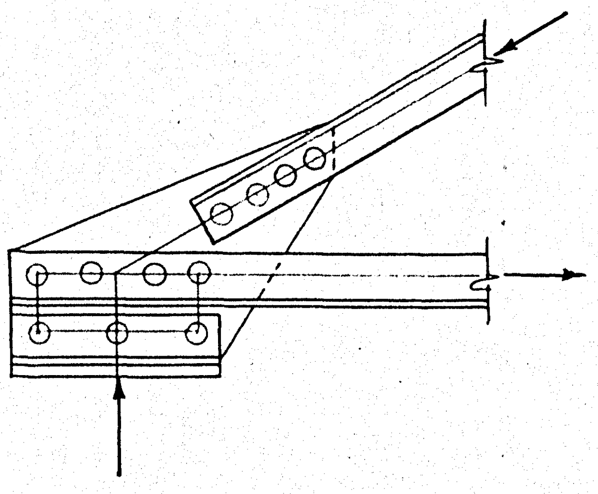


FIGURE 4

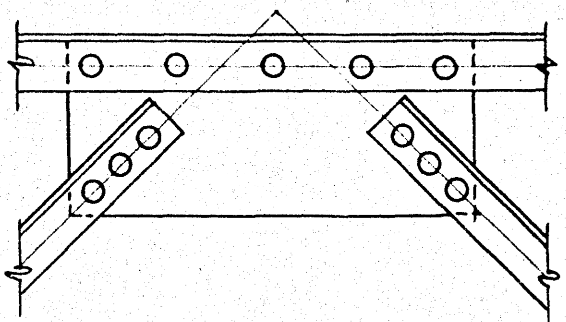


FIGURE 5

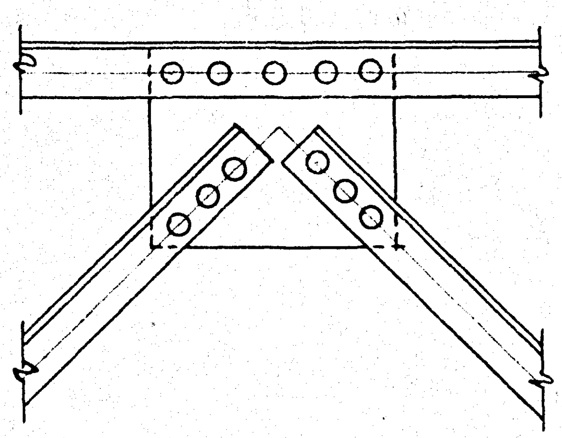


FIGURE 6

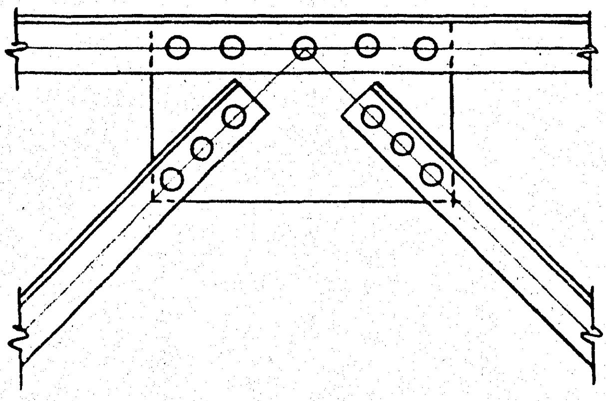


FIGURE 7

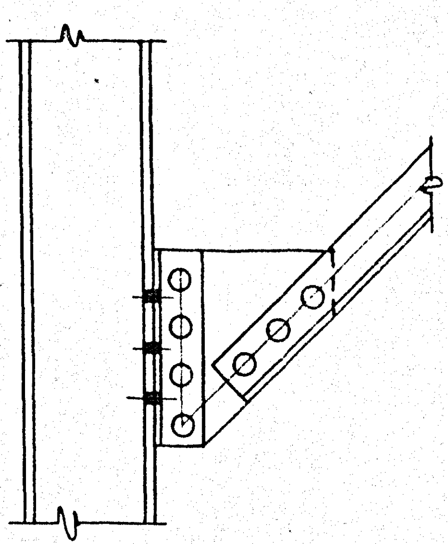


FIGURE 8

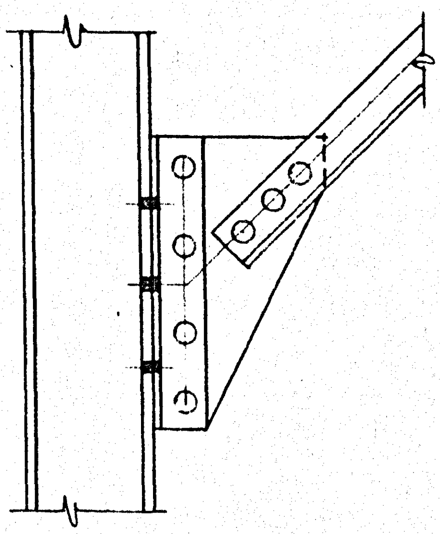


FIGURE 9

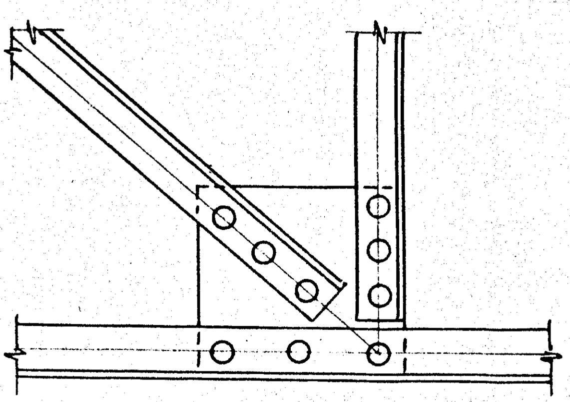


FIGURE 10

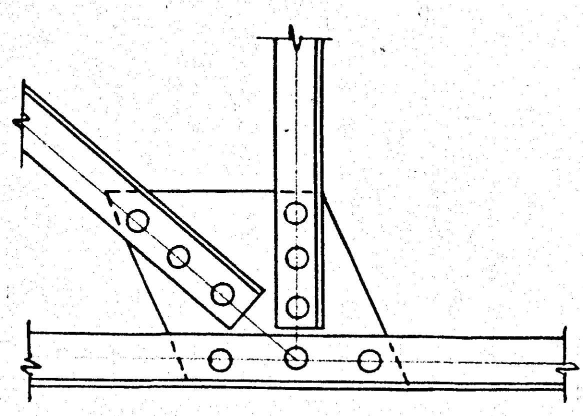


FIGURE 11

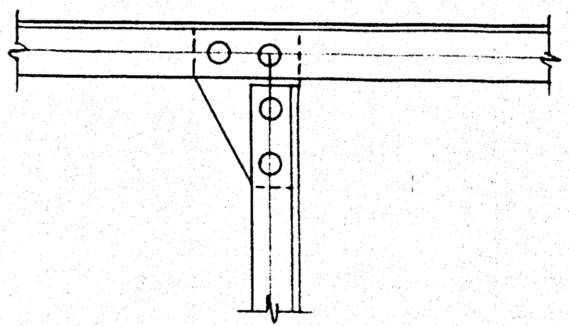


FIGURE 12

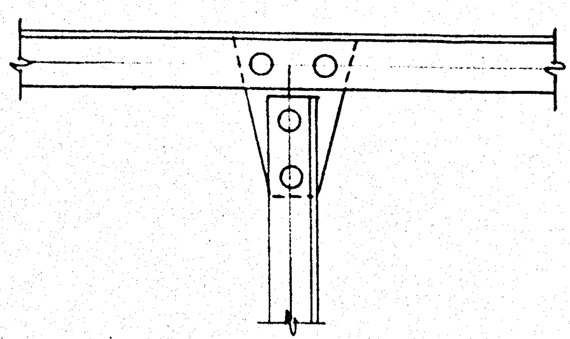


FIGURE 13

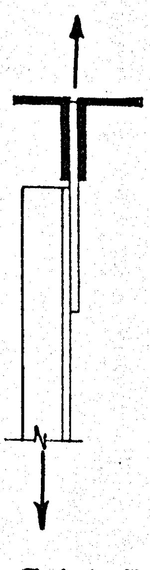


FIGURE 14

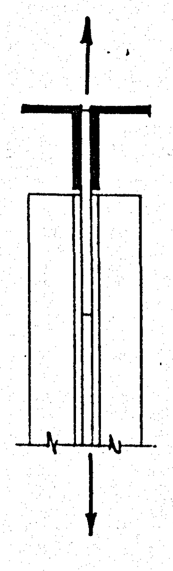


FIGURE 15

half by two and one-fourth inches. The two and one half inch leg is wide enough to drive a three-fourths inch rivet.

The weights of reinforced concrete members must be included in the loads supported. The table on page twenty-two gives the weights of slabs, beams and columns in pounds a lineal foot. Slabs are considered as strips one foot wide and the weight of concrete a cubic foot is assumed to be one hundred fifty pounds including the weight of the reinforcing bars.

The following slab and beam tables are based on the straight line formula. The design stresses do not cover a wide range and are comparatively low. Tensile stresses in concrete are neglected. The parabolic curve most nearly expresses the relation between stress and deformation and is followed where there is a wide range of stresses such as in experimental work. The error resulting from the use of the straight line formula is small and is on the safe side for the usual range of design stress.

The slab and beam table gives results in area of reinforcing and depth to center line of reinforcing. The table on page twenty-three gives the area of bars in square inches a foot width of slab. The size and spacing of bars can readily be determined if the area of the reinforcing a foot width of slab is known. The table on page twenty-four gives the perimeter of bars a foot

WEIGHTS OF CONCRETE MEMBERS

IN POUNDS A LINEAL FOOT

Depth in inches	Width in inches											
	8.	10.	12.	14.	16.	18.	20.	22.	24.	26.	28.	30.
4.			50.									
4.5			56.									
5.			62.									
5.5			69.									
6.			75.									
6.5			81.									
7.			87.									
7.5			94.									
8.			100.									
8.5			106.									
9.			112.									
10.			125.									
12.	100.	125.	150.									
14.	117.	146.	175.	204.								
16.	133.	167.	200.	233.	267.							
18.	150.	188.	225.	263.	300.	338.						
20.	167.	208.	250.	292.	333.	375.	417.					
22.	183.	229.	275.	321.	367.	413.	458.	504.				
24.	200.	250.	300.	350.	400.	450.	500.		600.			
26.	217.	271.	325.	379.	433.	488.	542.			704.		
28.	233.	292.	350.	408.	467.	525.	584.				817.	
30.	250.	313.	375.	438.	500.	563.	625.					938.
32.	267.	333.	400.	467.	533.	600.	667.					
34.	283.	354.	425.	496.	567.	638.	709.					
36.	300.	375.	450.	525.	600.	675.	750.					
38.	317.	396.	475.	554.	633.	713.	792.					
40.	333.	417.	500.	583.	667.	750.	834.					
42.	350.	438.	525.	613.	700.	788.	875.					

Weight of one cubic foot of concrete = 150.#

AREA OF BARS IN SQUARE INCHES

A FOOT WIDTH OF SLAB

Spacing of Bars in inches	Size of Bars in inches: 1/4 ●	3/8 ●	1/2 ●	1/2 ■	5/8 ●	3/4 ●	7/8 ●	1 ●	1 ■	1 1/8 ■	1 1/4 ■
3	0.20	0.44	0.78	1.00	1.23	1.77	2.40	3.14	4.00		
3½	0.17	0.38	0.67	0.86	1.05	1.51	2.06	2.69	3.43	4.34	
4	0.15	0.33	0.59	0.75	0.92	1.33	1.80	2.36	3.00	3.80	4.69
4½	0.13	0.29	0.52	0.67	0.82	1.18	1.60	2.09	2.67	3.37	4.17
5	0.12	0.26	0.47	0.60	0.74	1.06	1.44	1.88	2.40	3.04	3.75
5½	0.11	0.24	0.43	0.55	0.67	0.96	1.31	1.71	2.18	2.76	3.41
6	0.10	0.22	0.39	0.50	0.61	0.88	1.20	1.57	2.00	2.53	3.13
6½	0.09	0.20	0.36	0.46	0.57	0.82	1.11	1.45	1.85	2.34	2.89
7	0.08	0.19	0.34	0.43	0.53	0.76	1.03	1.35	1.71	2.17	2.68
7½	0.08	0.18	0.31	0.40	0.49	0.71	0.96	1.26	1.60	2.02	2.50
8	0.07	0.17	0.29	0.38	0.46	0.66	0.90	1.18	1.50	1.90	2.34
8½	0.07	0.16	0.28	0.35	0.43	0.62	0.85	1.11	1.41	1.79	2.21
9	0.07	0.15	0.26	0.33	0.41	0.59	0.80	1.05	1.33	1.69	2.08
9½	0.06	0.14	0.25	0.32	0.39	0.56	0.76	0.99	1.26	1.60	1.97
10	0.06	0.13	0.24	0.30	0.37	0.53	0.72	0.94	1.20	1.52	1.88
10½	0.06	0.13	0.22	0.29	0.35	0.51	0.69	0.90	1.14	1.45	1.79
11	0.05	0.12	0.21	0.27	0.33	0.48	0.66	0.86	1.09	1.38	1.70
11½	0.05	0.11	0.20	0.26	0.32	0.46	0.63	0.82	1.04	1.32	1.63
12	0.05	0.11	0.20	0.25	0.31	0.44	0.60	0.79	1.00	1.27	1.56

PERIMETER OF BARS IN INCHES A FOOT WIDTH OF SLAB

Spacing of Bars in inches	Size of Bars in inches: 1/4 ●	3/8 ●	1/2 ●	1/2 ■	5/8 ●	3/4 ●	7/8 ●	1 ●	1 ■	1 1/8 ■	1 1/4 ■
3	3.12	4.72	6.28	8.00	7.84	9.40	11.00	12.56	16.00		
3 1/2	2.67	4.05	5.48	6.85	6.71	8.05	9.45	10.75	13.70	15.40	
4	2.34	3.54	4.71	6.00	5.88	7.05	8.25	9.42	12.00	13.50	15.00
4 1/2	2.07	3.15	4.18	5.34	5.22	6.27	7.34	8.37	10.65	11.95	13.30
5	1.87	2.84	3.76	4.80	4.70	5.64	6.61	7.54	9.60	10.78	12.00
5 1/2	1.70	2.58	3.42	4.36	4.27	5.13	6.00	6.85	8.73	9.80	10.90
6	1.56	2.36	3.14	4.00	3.92	4.70	5.50	6.28	8.00	9.00	10.00
6 1/2	1.44	2.18	2.90	3.69	3.62	4.34	5.09	5.80	7.39	8.29	9.22
7	1.34	2.03	2.69	3.43	3.35	4.03	4.72	5.38	6.86	7.69	8.56
7 1/2	1.25	1.89	2.51	3.20	3.13	3.76	4.41	5.03	6.40	7.20	8.00
8	1.17	1.77	2.35	3.00	2.94	3.53	4.13	4.71	6.00	6.75	7.50
8 1/2	1.10	1.67	2.21	2.83	2.76	3.32	3.88	4.43	5.64	6.34	7.05
9	1.04	1.58	2.09	2.67	2.61	3.13	3.66	4.19	5.34	6.00	6.66
9 1/2	0.99	1.49	1.98	2.53	2.47	2.97	3.48	3.96	5.06	5.67	6.31
10	0.94	1.42	1.88	2.40	2.35	2.82	3.30	3.77	4.80	5.39	6.00
10 1/2	0.89	1.35	1.79	2.29	2.24	2.69	3.14	3.58	4.56	5.13	5.70
11	0.85	1.29	1.71	2.18	2.13	2.56	3.00	3.42	4.36	4.90	5.45
11 1/2	0.81	1.23	1.64	2.09	2.04	2.45	2.87	3.27	4.17	4.68	5.20
12	0.78	1.18	1.57	2.00	1.96	2.35	2.75	3.14	4.00	4.50	5.00

width of slab and is convenient for investigating bond stresses. The present range of eleven bar sizes allows the designer a wide enough range of bar sizes for design purposes. The number of different sizes used in a structure should be held to a minimum so as to expedite fabrication, and simplify handling in the field. The recommended grade of steel for reinforcing bars is the intermediate grade. This grade possesses a high yield point and is sufficiently ductile to permit bending. The number of different length of bars should be held to a minimum and should be given in multiples of three inches. The lengths of bars which must be exact should be noted on the bar list.

The table on page twenty-six gives the reinforcing for concrete slabs and rectangular beams a foot width for various depths and bending moments. Slabs are treated as shallow rectangular beams of some unit width, usually a foot. The area of steel required can be computed by direct ratio. The following example will show the use of the table for finding the reinforcing need for concrete slabs: Moment 3300 foot pounds; depth 5 inches.

$A_s = \frac{3300}{3470} \times .53 = .505$ square inches. Referring to the table on page twenty-three we find that one-half inch square bars spaced six inches on centers is satisfactory. The same type of solution can be applied to rectangular beams except the width of beam must be sufficient to properly space

REINFORCING FOR CONCRETE SLABS AND RECTANGULAR BEAMS

1. $f_c = .4 f_c' = 800.$ $f_c' = 2000.$ $f_s = 18000.$
2. $R = 138.7$ $n = 15$ $p = 0.0089$
3. d = effective depth in inches
4. A_s = area of steel in square inches a foot width of slab or beam
5. M = moment in foot pounds a foot width of slab or beam

d	A_s	M	d	A_s	M
2	.21	555.	18	1.92	44,900.
2½	.27	867.	19	2.03	50,100.
3	.32	1,248.	20	2.14	55,500.
3½	.37	1,700.	21	2.24	61,200.
4	.43	2,220.	22	2.35	67,100.
4½	.48	2,810.	23	2.46	73,400.
5	.53	3,470.	24	2.56	79,900.
5½	.59	4,200.	25	2.67	86,700.
6	.64	4,990.	26	2.78	93,800.
6½	.69	5,860.	27	2.88	101,000.
7	.75	6,800.	28	2.99	109,000.
7½	.80	7,800.	29	3.10	117,000.
8	.85	8,880.	30	3.20	125,000.
8½	.91	10,020.	31	3.31	133,000.
9	.96	11,230.	32	3.42	142,000.
9½	1.01	12,520.	33	3.52	151,000.
10	1.07	13,870.	34	3.63	160,000.
10½	1.12	15,300.	35	3.74	170,000.
11	1.17	16,800.	36	3.84	180,000.
12	1.28	20,000.	37	3.95	190,000.
13	1.39	23,400.	38	4.06	200,000.
14	1.50	27,200.	39	4.17	211,000.
15	1.60	31,200.	40	4.27	222,000.
16	1.71	35,500.	41	4.38	233,000.
17	1.82	40,100.	42	4.59	245,000

the reinforcing. The depth of beams is sometimes limited for reasons of head room, or architectural treatment. The following example is a typical case:

Moment 93,000 foot pounds; depth 22 inches

$b = \frac{93,000}{67,000} = 1.39$ feet. Use 1'-5" or 1.42 feet

$A_s = 2.35 \times 1.42 = 3.34$ square inches.

The number of bars required can be determined by referring to the table on page twenty-eight and the minimum width of beam by referring to the table on page twenty-nine. The reinforcing required is six bars, seven-eights inch in diameter placed in one layer and the minimum practical width is sixteen inches. The reinforcing in slabs and beams should also be investigated for bond before finally selecting the bars required. The relative costs of concrete, steel and forms should be considered when determining the dimensions of beams except when limited by architectural treatment or head room requirements. The spacing of beams through out the structure should be as uniform as practicable and the dimensions of beams should be in even inches.

The complete design of reinforced concrete slabs for various bending moments, span and safe superimposed loads is shown on the tables on pages thirty, thirty-one and thirty-two. The practical application is self evident but the actual superimposed load on the slab under consideration is usually less than the safe superimposed load. The reinforcing can be reduced in such cases by direct proportion. The following example

AREAS AND PERIMETERS OF GROUPS OF BARS

Number	Size of bars in inches										
	1/4 ●	3/8 ●	1/2 ●	1/2 ■	5/8 ●	3/4 ●	7/8 ●	1 ●	1 ■	1 1/8 ■	1 1/4 ■
1	.05 .78	.11 1.18	.20 1.57	.25 2.00	.31 1.96	.44 2.35	.60 2.75	.79 3.14	1.00 4.00	1.27 4.50	1.56 5.00
2	.10 1.56	.22 2.36	.40 3.14	.50 4.00	.62 3.92	.88 4.70	1.20 5.50	1.58 6.28	2.00 8.00	2.54 9.00	3.12 10.00
3	.15 2.34	.33 3.54	.60 4.71	.75 6.00	.93 5.88	1.32 7.05	1.80 8.25	2.37 9.42	3.00 12.00	3.81 13.50	4.68 15.00
4	.20 3.12	.44 4.72	.80 6.28	1.00 8.00	1.24 7.84	1.76 9.40	2.40 11.00	3.16 12.56	4.00 16.00	5.08 18.00	6.24 20.00
5	.25 3.90	.55 5.90	1.00 7.85	1.25 10.00	1.55 9.80	2.20 11.75	3.00 13.75	3.95 15.70	5.00 20.00	6.35 22.50	7.80 25.00
6	.30 4.68	.66 7.08	1.20 9.42	1.50 12.00	1.86 11.76	2.64 14.10	3.60 16.50	4.74 18.84	6.00 24.00	7.62 27.00	9.36 30.00
7	.35 5.46	.77 8.26	1.40 10.99	1.75 14.00	2.17 13.72	3.08 16.45	4.20 19.25	5.53 21.98	7.00 28.00	8.89 31.50	10.92 35.00
8	.40 6.24	.88 9.44	1.60 12.56	2.00 16.00	2.48 15.68	3.52 18.80	4.80 22.00	6.32 25.12	8.00 32.00	10.16 36.00	12.48 40.00
9	.45 7.02	.99 10.62	1.80 14.13	2.25 18.00	2.79 17.64	3.96 21.15	5.40 24.75	7.11 28.26	9.00 36.00	11.43 40.50	14.04 45.00
10	.50 7.80	1.10 11.80	2.00 15.70	2.50 20.00	3.10 19.60	4.40 23.50	6.00 27.50	7.90 31.40	10.00 40.00	12.70 45.00	15.60 50.00
11	.55 8.58	1.21 12.98	2.20 17.27	2.75 22.00	3.41 21.56	4.84 25.85	6.60 30.25	8.69 34.54	11.00 44.00	13.97 49.50	17.16 55.00
12	.60 9.36	1.32 14.16	2.40 18.84	3.00 24.00	3.72 23.52	5.28 28.20	7.20 33.00	9.48 37.68	12.00 48.00	15.24 54.00	18.72 60.00
13	.65 10.14	1.43 15.34	2.60 20.41	3.25 26.00	4.03 25.48	5.72 30.55	7.80 35.75	10.26 40.82	13.00 52.00	16.51 58.50	20.28 65.00
14	.70 10.92	1.54 16.52	2.80 21.98	3.50 28.00	4.34 27.44	6.16 32.90	8.40 38.50	11.06 43.96	14.00 56.00	17.78 63.00	21.84 70.00
15	.75 11.70	1.65 17.70	3.00 23.55	3.75 30.00	4.65 29.40	6.60 35.25	9.00 41.25	11.85 47.10	15.00 60.00	19.05 67.50	23.40 75.00
	Weight of each bar in pounds a lineal foot										
	.17	.38	.68	.86	1.06	1.52	2.07	2.70	3.44	4.35	5.37

MINIMUM WIDTH OF BEAM IN INCHES

Number	Size of Bars in inches										
	1/4 ●	3/8 ●	1/2 ●	1/2 ■	5/8 ●	3/4 ●	7/8 ●	1 ●	1 ■	1 1/8 ■	1 1/4 ■
1	3.3	3.4	3.5	3.5	3.7	3.8	3.9	4.0	4.0	4.2	4.3
2	4.6	4.8	5.0	5.0	5.4	5.7	6.1	6.5	7.0	7.6	8.0
3	5.8	6.2	6.5	6.5	7.1	7.6	8.3	9.0	10.0	11.0	11.8
4	7.1	7.6	8.0	8.0	8.8	9.5	10.5	11.5	13.0	14.4	15.5
5	8.3	9.0	9.5	9.5	10.5	11.3	12.7	14.0	16.0	17.8	19.3
6	9.6	10.4	11.0	11.0	12.2	13.2	14.9	16.5	19.0	21.2	23.0
7	10.8	11.8	12.5	12.5	13.9	15.1	17.1	19.0	22.0	24.6	26.8
8	12.1	13.0	14.0	14.0	15.6	17.0	19.3	21.5	25.0	28.0	30.5
9	13.3	14.4	15.5	15.5	17.3	18.8	21.5	24.0	28.0	31.4	34.3
10	14.6	15.8	17.0	17.0	19.0	20.7	23.7	26.5	31.0	34.8	38.0

The above widths of beams are based on the following assumptions.

1. Fireproofing is 1½ inches to face of bar.
2. Clear space between bars in no case less than 1 inch.
3. Clear space between round bars equals 1½ diameters.
4. Clear space between square bars equals 2 times one side.

$f_s = 18000.$; $f_c = 800.$; $f_c = .4 f_c'$; $f_c' = 2000.$; $n = 15$; effective depth, $d = h - 1$; h = thickness of slab in inches; U = weight of slab in pounds a square foot; A_s = area of steel in square inches a foot width of slab; v = 40. pounds or less a square inch; u = 100. pounds or less a square inch for values to right of heavy zig-zag line; u = 101. pounds or more a square inch for values to left of heavy zig-zag line.

SAFE SUPERIMPOSED LOAD IN POUNDS A SQUARE FOOT

h	U	A_s	Bars		Clear span in feet														
			Size	Spac.	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
3	37.	.21	3/8"φ	6"o.c.	240	141.	86.	54.	32.	18.									
3½	43.	.27	3/8"φ	5"o.c.	390.	234	149.	98.	64.	42.	25.	13.							
4	50.	.32	1/2"φ	7"o.c.	575.	350.	228.	154.	106.	73.	50.	32.	19.	9.					
4½	56.	.37	1/2"φ	6"o.c.		490.	323.	223.	157.	113.	81.	57.	39.	25.	14.				
5	62.	.43	1/2"φ	5½"o.c.			431.	301.	216.	157.	115.	85.	61.	43.	29.	17.			
5½	69.	.48	1/2"□	6"o.c.			560.	393.	285	211.	157.	118.	88.	65.	47.	32.	19.		
6	75.	.53	1/2"□	5½"o.c.				491.	359.	268.	203.	154.	118.	89.	67.	48.	33.	21.	11.
6½	81.	.59	1/2"□	5"o.c.					448.	337.	258.	199.	154.	119	92	69.	51.	36.	23.
7	87.	.64	5/8"φ	5½"o.c.					537.	406.	313.	243.	191.	150.	117.	90.	69.	51.	36.
7½	94.	.69	5/8"φ	5"o.c.						486.	376.	294.	232.	185.	146.	115.	89.	69.	51.
8	100.	.75	3/4"φ	7"o.c.							444.	350.	278.	222.	178.	142.	112.	88.	68.
8½	106.	.80	3/4"φ	6½"o.c.								410.	328.	264.	213.	172.	164.	110.	87.
9	112.	.85	3/4"φ	6"o.c.									381.	309.	251.	204.	166.	134.	107.
9½	118.	.91	3/4"φ	5½"o.c.										356.	291.	238.	195.	159.	129.
10	125.	.96	3/4"φ	5½"o.c.										407.	334.	275.	226.	186.	153.
10½	131.	1.01	3/4"φ	5"o.c.										462.	380.	314.	261.	216.	178.
11	137.	1.07	7/8"φ	6½"o.c.										519.	428.	355.	296.	246.	205.
11½	143.	1.12	7/8"φ	6"o.c.										581.	481.	400.	334.	280.	234.
12	150.	1.17	7/8"φ	6"o.c.										645.	535.	447.	375.	315.	265.

$f_s = 18000.$; $f_c = 800.$; $f_c = .4 f'_c$; $f'_c = 2000.$; $n = 15$; effective depth, $d = h - 1$; h = thickness of slab in inches; U = weight of slab in pounds a square foot; A_s = area of steel in square inches a foot width of slab; v = 40. pounds or less a square inch; u = 100. pounds or less a square inch for values to right of heavy zig-zag line; u = 101. pounds or more a square inch for values to left of heavy zig-zag line.

SAFE SUPERIMPOSED LOAD IN POUNDS A SQUARE FOOT

h	U	A_s	Bars Size	Bars Spac.	Clear span in feet: 4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
3	37.	.21	3/8" φ	6" o.c.	310.	185.	117.	76.	50.	31.	18.	9.							
3½	43.	.27	3/8" φ	5" o.c.		303.	197.	133.	91.	63.	43.	28.	16.						
4	50.	.32	½" φ	7" o.c.		450.	297.	205.	145.	104.	75.	53.	37.	24.	14.				
4½	56.	.37	½" φ	6" o.c.			418.	292.	211.	155.	115.	85.	69.	45.	31.	20.	11.		
5	62.	.43	½" φ	5½" o.c.				391.	285.	212.	160.	122.	92.	69.	51.	37.	25.	15.	
5½	69.	.48	½" □	6" o.c.					373.	281.	214.	165.	128.	98.	75.	57.	41.	29.	18.
6	75.	.53	½" □	5½" o.c.					467.	353.	272.	212.	166.	130.	102.	79.	60.	45.	32.
6½	81.	.59	½" □	5" o.c.						441.	342.	269.	213.	170.	135.	107.	84.	65.	50.
7	87.	.64	5/8" φ	5½" o.c.							413.	326.	260.	209.	168.	135.	108.	86.	67.
7½	94.	.69	5/8" φ	5" o.c.								391.	314.	253.	206.	167.	135.	109.	87.
8	100.	.75	¾" φ	7" o.c.									372.	302.	247.	202.	166.	135.	110.
8½	106.	.80	¾" φ	6½" o.c.										356.	293.	241.	199.	164.	135.
9	112.	.85	¾" φ	6" o.c.											341.	283.	235.	195.	162.
9½	118.	.91	¾" φ	5½" o.c.											393.	327.	273.	228.	191.
10	125.	.96	¾" φ	5½" o.c.											449.	374.	314.	264.	222.
10½	131.	1.01	¾" φ	5" o.c.											508.	426.	358.	303.	257.
11	137.	1.07	7/8" φ	6½" o.c.													404.	342.	290.
11½	143.	1.12	7/8" φ	6" o.c.													454.	385.	328.
12	150.	1.17	7/8" φ	6" o.c.													506.	431.	368.

CONCRETE SLABS

Bending Moment, $M = \frac{wL^2}{12}$

$f_s = 18000.$; $f_c = 800.$; $f_c = .4f'_c$; $f'_c = 2000.$; $n = 15$; effective depth, $d = h - 1$; h = thickness of slab in inches; U = weight of slab in pounds a square foot; A_s = area of steel in square inches a foot width of slab; $v = 40.$ pounds or less a square inch; $u = 100.$ pounds or less a square inch for values to right of heavy zig-zag line; $u = 101.$ pounds or more a square inch for values to left of heavy zig-zag line.

SAFE SUPERIMPOSED LOAD IN POUNDS A SQUARE FOOT

h	U	A_s	Bars		Clear span in feet														
			Size	Spac.	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
3	37.	.21	3/8"φ	6"o.c.	379.	229.	148.	99.	67.	45.	30.	18.							
3½	43.	.27	3/8"φ	5"o.c.			245.	168.	119.	84.	60	42.	28.	18.					
4	50.	.32	1/2"φ	7"o.c.			366.	256.	184.	135.	100.	74.	54	39.	26.	17.			
4½	56.	.37	1/2"φ	6"o.c.				362.	264.	197.	149.	113.	86.	65.	48.	35.	24.	15.	
5	62.	.43	1/2"φ	5½"o.c.					354.	267.	204.	158.	123.	96.	74.	56.	42.	30.	20.
5½	69.	.48	1/2"□	6"o.c.						350.	271.	212.	167.	132.	104.	82.	64.	48.	36.
6	75.	.53	1/2"□	5½"o.c.							341.	269.	214.	171.	137.	110.	88.	69.	53.
6½	81.	.59	1/2"□	5"o.c.								339.	272	220.	178.	145.	117.	95.	76.
7	87.	.64	5/8"φ	5½"o.c.									329.	268.	219.	179.	147.	120.	98.
7½	94.	.69	5/8"φ	5"o.c.										323.	266.	219.	181.	150.	123.
8	100.	.75	3/4"φ	7"o.c.											316.	263.	219.	182.	152.
8½	106.	.80	3/4"φ	6½"o.c.												311	260.	218.	183.
9	112.	.85	3/4"φ	6"o.c.													304.	257.	217.
9½	118.	.91	3/4"φ	5½"o.c.														298	253.
10	125.	.96	3/4"φ	5½"o.c.														342.	291.
10½	131.	1.01	3/4"φ	5"o.c.															333.
11	137.	1.07	7/8"φ	6½"o.c.															
11½	143.	1.12	7/8"φ	6"o.c.															
12	150.	1.17	7/8"φ	6"o.c.															

will show a typical case:

Simple supported slab, span 9 feet

Superimposed load 100 pounds a square foot.

Slab $4\frac{1}{2}$ thick

$A_s = \frac{100}{113} \times .37 = .33$. Referring to the table on page twenty-three we find that one-half inch round bars spaced seven inches on centers is satisfactory.

The majority of beams found in the usual reinforced structural design are tee beams. The slab thicknesses are usually determined first and the neutral axis of floor beams usually lie below the slabs. The beam is designed as a rectangular beam in exceptional cases if the neutral axis falls in the slab. The tables on pages thirty-four and thirty-five consider compression in the slab only for each foot width of slab. The flange width is usually limited by the local building code but seldom is ever all needed. The compression in the stem between the neutral axis and bottom of the floor slab is neglected. Tee beams can be more accurately designed by treating the entire stem as a rectangular beam and the adjacent slab as a flange. The following example will illustrate the application of these tables in actual design:

1. Compression in stem neglected.

   Moment 170,000 foot pounds

   Depth 24 inches

   Slab 4 inches thick

   $b = \frac{170,000}{56,200} = 3.03$ feet

   $A_s = 1.69 \times 3.03 = 5.12$ square inches.

REINFORCING FOR CONCRETE TEE BEAMS

1. $f_s = 18000.$ $f_c = 800.$ $n = 15.$ $f_c = .4f_c'$ $f_c' = 2000.$
2. d = effective depth in inches
3. A_s = area of steel in square inches a foot width of flange
4. M = moment in foot pounds a foot width of flange
5. The values above the heavy zig-zag line are for solid slabs and the neutral axis lies in the flange
6. The values below the heavy zig-zag line neglect compression in the stem and the neutral axis lies in stem

d	Thickness of flange in inches									
	4		4½		5		5½		6	
	A_s	M	A_s	M	A_s	M	A_s	M	A_s	M
10.	1.07	13,870.	1.07	13,870.	1.07	13,870.	1.07	13,870.	1.07	13,870.
11.	1.16	16,700.	1.17	16,800.	1.17	16,800.	1.17	16,800.	1.17	16,800.
12.	1.24	19,600.	1.27	19,900.	1.28	20,000.	1.28	20,000.	1.28	20,000.
13.	1.31	22,500.	1.36	23,100.	1.38	23,400.	1.39	23,400.	1.39	23,400.
14.	1.37	25,400.	1.44	26,400.	1.48	27,000.	1.49	27,200.	1.50	27,200.
15.	1.42	28,400.	1.50	29,700.	1.56	30,600.	1.59	31,000.	1.60	31,200.
16.	1.47	31,500.	1.56	33,000.	1.63	34,200.	1.67	35,000.	1.70	35,400.
17.	1.51	34,500.	1.61	36,400.	1.69	37,900.	1.75	39,000.	1.79	39,700.
18.	1.54	37,600.	1.65	39,800.	1.74	41,600.	1.81	43,000.	1.87	44,000.
19.	1.57	40,600.	1.69	43,200.	1.79	45,400.	1.87	47,100.	1.94	48,400.
20.	1.60	43,700.	1.72	46,700.	1.83	49,200.	1.93	51,200.	2.00	52,700.
21.	1.63	46,800.	1.76	50,100.	1.87	53,000.	1.97	55,300.	2.06	57,200.
22.	1.65	49,900.	1.79	53,600.	1.91	56,800.	2.02	59,500.	2.11	61,700.
23.	1.67	53,100.	1.81	57,100.	1.94	60,600.	2.06	63,700.	2.16	66,300.
24.	1.69	56,200.	1.84	60,600	1.97	64,500.	2.09	67,900.	2.20	70,800.
25.	1.71	59,300.	1.86	64,100.	2.00	68,300.	2.13	72,100.	2.24	75,400.
26.	1.72	62,500.	1.88	67,600.	2.03	72,200.	2.16	76,300.	2.28	79,900.
27.	1.74	65,600.	1.90	71,100.	2.05	76,100.	2.19	80,600.	2.31	84,500.
28.	1.75	68,700.	1.92	74,600.	2.07	80,000.	2.21	84,800.	2.34	89,100.
29.	1.77	71,900.	1.93	78,200.	2.09	83,900.	2.24	89,100.	2.37	93,800.
30.	1.78	75,000.	1.95	81,700.	2.11	87,800.	2.26	93,300.	2.40	98,400.
31.	1.79	78,200.	1.96	85,200.	2.13	91,700.	2.28	97,600.	2.43	103,000.
32.	1.80	81,300.	1.98	88,700.	2.15	95,600.	2.30	102,000.	2.45	108,000.

REINFORCING FOR CONCRETE TEE BEAMS

1. $f_s = 18000.$ $f_c = 800.$ $n = 15.$ $f_c = .4 f_c'$ $f_c' = 2000.$
2. d = effective depth in inches
3. A_s = area of steel in square inches a foot width of flange.
4. M = moment in foot pounds a foot width of flange
5. The values above the heavy zig-zag line are for solid slabs and the neutral axis lies in the flange
6. The values below the heavy zig-zag line neglect compression in the stem and the neutral axis lies in stem

d	Thickness of flange in inches									
	6½		7		7½		8		9	
	A_s	M	A_s	M	A_s	M	A_s	M	A_s	M
17.	1.81	40,000.	1.82	40,100.	1.82	40,100.	1.82	40,100.	1.82	40,100.
18.	1.90	44,600.	1.92	44,900.	1.92	44,900.	1.92	44900.	1.92	44,900.
19.	1.98	49,300.	2.01	49,800.	2.03	50,100.	2.03	50,100.	2.03	50,100.
20.	2.06	54,000.	2.10	54,800.	2.12	55,300.	2.13	55,500.	2.14	55,500.
21.	2.13	58,800.	2.18	59,900.	2.21	60,600.	2.23	61,100.	2.24	61,200.
22.	2.19	63,600.	2.25	65,000.	2.30	66,000.	2.33	66,700.	2.35	67,100.
23.	2.24	68,400.	2.31	70,100.	2.37	71,500.	2.41	72,400.	2.45	73,300.
24.	2.29	73,300.	2.37	75,300.	2.44	77,000.	2.49	78,200.	2.55	79,600.
25.	2.34	78,200.	2.43	80,500.	2.50	82,500.	2.56	84,000.	2.64	86,000.
26.	2.38	83,100.	2.48	85,800.	2.56	88,100.	2.63	89,900.	2.72	92,500.
27	2.42	88,000.	2.52	91,100.	2.61	93,700.	2.69	95,800.	2.80	99,000.
28.	2.46	93,000.	2.57	96,400.	2.66	99,300.	2.74	102,000.	2.87	106,000.
29.	2.49	98,000.	2.61	102,000.	2.71	105,000.	2.79	108,000.	2.94	112,000.
30.	2.53	103,000.	2.64	107,000.	2.75	111,000.	2.84	114,000.	3.00	119,000.
31.	2.56	108,000.	2.68	112,000.	2.79	116,000.	2.89	120,000.	3.06	125,000.
32.	2.59	113,000.	2.71	118,000.	2.83	122,000.	2.93	126,000.	3.11	132,000.
33.	2.61	118,000.	2.74	123,000.	2.86	128,000.	2.97	132,000.	3.16	139,000.
34.	2.64	123,000.	2.77	129,000.	2.90	134,000.	3.01	138,000.	3.21	146,000.
35.	2.66	128,000.	2.80	134,000.	2.93	139,000.	3.05	144,000.	3.26	152,000.
36.	2.68	133,000.	2.83	139,000.	2.96	145,000.	3.08	150,000.	3.30	159,000.
37.	2.71	138,000.	2.85	145,000.	2.99	151,000.	3.11	156,000.	3.34	166,000.
38.	2.73	143,000.	2.87	150,000.	3.01	157,000.	3.14	163,000.	3.38	173,000.

Referring to the table on page twenty-eight four one and one-eighth square bars are found to be a slightly under size but are satisfactory. The width of the stem shall be sixteen inches according to the table on page twenty-nine, but the shear should also be investigated before finally deciding on the size.

2. Compression in stem included:

Moment 140,000 foot pounds

Depth 20 inches

Slab $4\frac{1}{2}$ inches

Stem 14 inches wide

Rectangular beam table page twenty-six

$M = 55,500 \times 1.17 = 65,000$ foot pounds

$A_s = 2.14 \times 1.17 = 2.50$ square inches

Flange, table on page thirty-four

$140,000 - 65,000 = 75,000$

$b = \frac{75,000}{46,700} = 1.61$ feet

$A_s = 1.72 \times 1.61 = 2.77$ square inches

Total $b = 1.17 + 1.61 = 2.78$

Total $A_s = 2.50 \times 2.77 = 5.37$ square inches

Referring to table on page twenty-eight seven one inch diameter bars in two layers is satisfactory. The lower layer contains four bars and the upper one three. The width is satisfactory according to the table on page twenty-nine but the shear should also be investigated before finally deciding on the size.

The bond stresses in all slabs and beams must be investigated but cannot readily be tabulated. The number of stirrups and their location must also be computed. The concrete is considered as resisting shear up to a certain limit but for greater stresses reinforcing bars are required in the form of stirrups. The diagram for locating stirrups for uniformly loaded beams on page thirty-nine is very convenient. The following example illustrates its application in actual design:

L = 60 inches

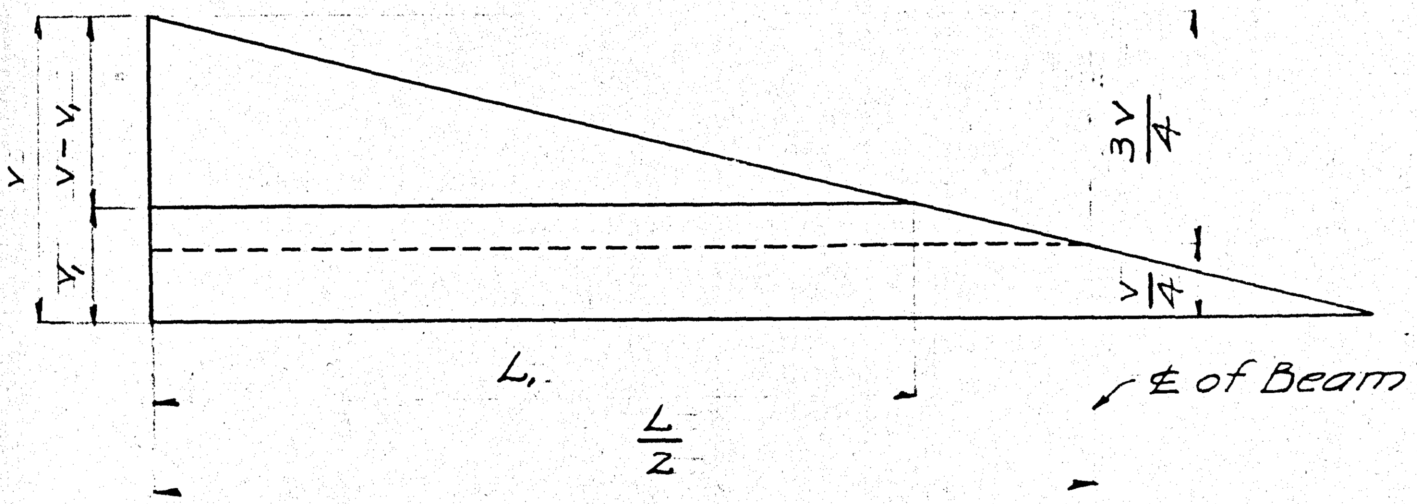
N = 7 inches

d = 16 inches

Lay a scale diagonally on the parallel lines with one end at point eight and rotate until the top horizontal line intersects the scale at sixty. The spacing can be read and is as follows: 4,5,5,5,6,8,12,15 inches. The Joint Committee however limits the spacing to forty-five hundreds of the effective beam depth so the following spacing in inches will be adopted: 4,5,5,5,6,7,7,7,7,7, inches.

The design of continuous tee beams make it necessary to use beams with steel in compression at the supports where negative moment occurs. The mechanical equipment may necessitate holes in the floor on both sides of tee beams for part of the length which will also necessitate steel in compression. The breadth and depth of the rectangular beam reinforced for compression is the same as the entire stem of the tee beam. The

NUMBER OF STIRRUPS



Shear diagram

$$\frac{\frac{L_1}{\frac{L}{2}}}{} = \frac{V - V_1}{\frac{3V}{4}} \qquad L_1 = \frac{\frac{L}{2}(V - V_1)}{\frac{3V}{4}}$$

$$L_1 = \frac{2L(V - V_1)}{3V}$$

NUMBER OF STIRRUPS

$$N = \frac{\frac{L_1}{2}(V - V_1)b}{f_v A_s} = \frac{L_1(V - V_1)b}{2 f_v A_s} \qquad f_v = 16000.^{\#}$$

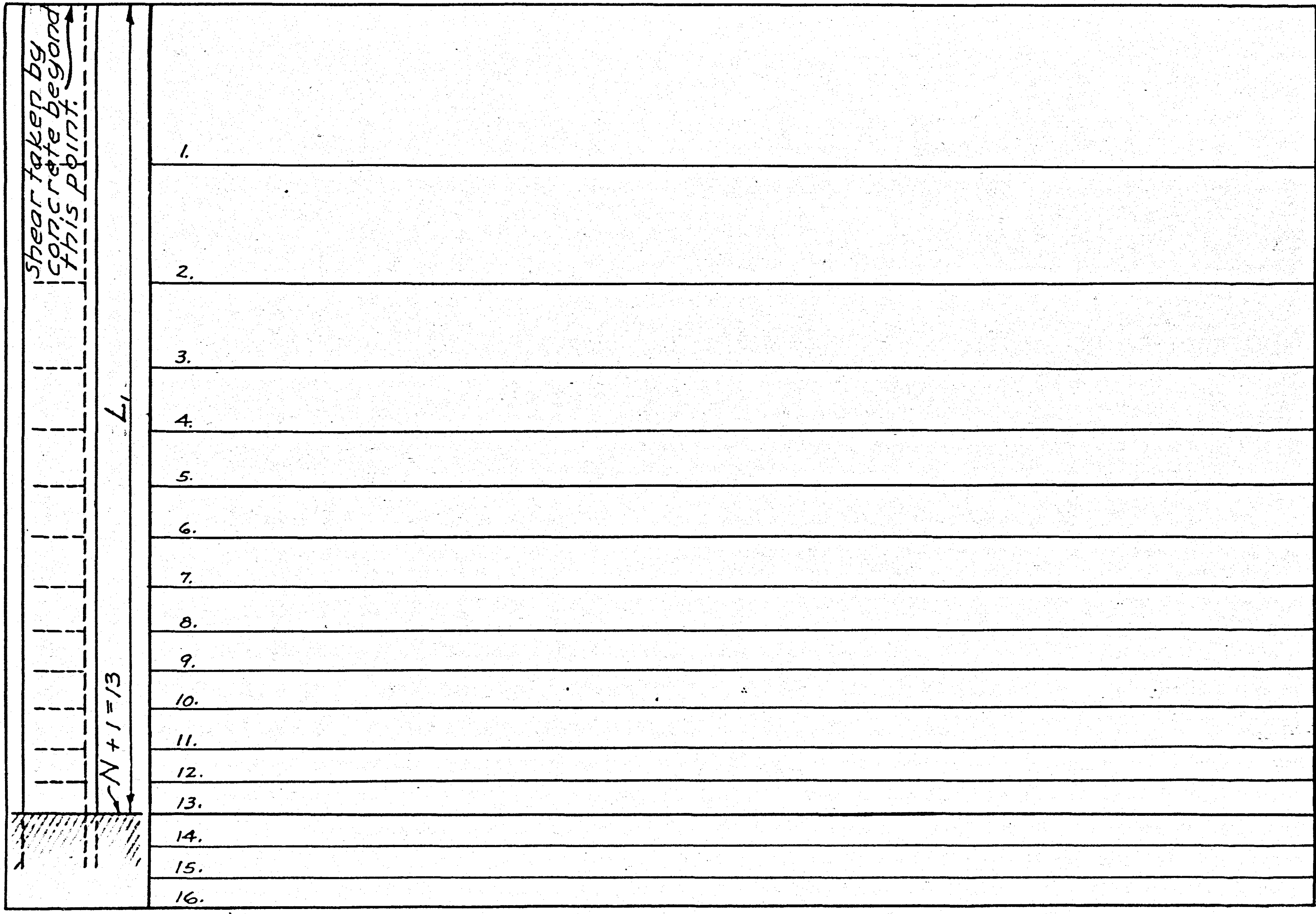
$\frac{3}{8}$"ϕ U Stirrups $N = \dfrac{L_1(V - V_1)b}{7060.}$

$\frac{1}{2}$"ϕ U Stirrups $N = \dfrac{L_1(V - V_1)b}{12600.}$

$\frac{1}{2}$"$\square$ U Stirrups $N = \dfrac{L_1(V - V_1)b}{16000.}$

"v" is the unit shear and $= \dfrac{V}{bjd}$

Diagram for Locating Stirrups in Uniformly Loaded Beams.



following example will illustrate the application of the tables on pages forty-one to forty-four.

Moment 72,000 foot pounds or 864,000 inch pounds

Breadth 12 inches

Depth 20 inches

$p = .015$

$$\frac{d}{d} = \frac{2}{20} = .1$$

$p = .5p$

From table on page forty-three

$L = .261 \qquad R = .0131$

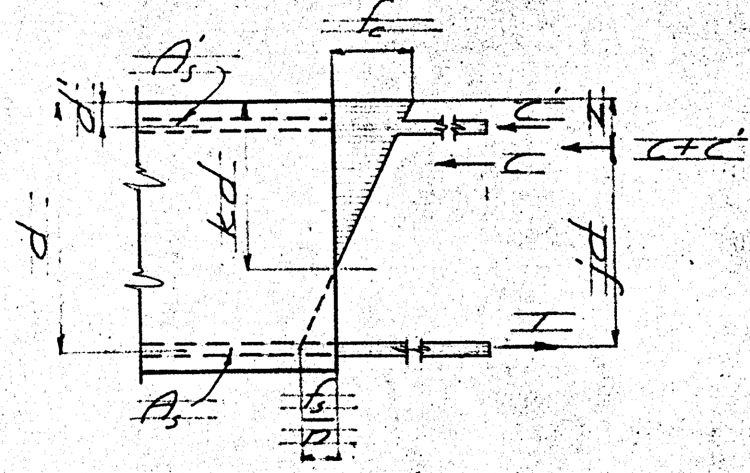
$$fc = \frac{M}{bd\ L} = \frac{864,000}{12 \times 20 \times .261} = 690 \text{ pounds square inch}$$

$$f_s = \frac{M}{bd\ R} = \frac{864,000}{12 \times 20 \times .0131} = 13,480 \text{ pounds a square inch.}$$

The usual design of a rectangular beam reinforced for compression consists of investigating the stresses of the concrete and steel. The size of reinforcing for practical reasons is the same as the tee beams since it is continued from the tee beam. Tee beams may be sufficiently haunched at the supports to be treated as a rectangular beam with reinforcing for tension only.

The usual beam and slab construction divides the ceiling into panels which do not coincide with partitions between rooms. The beams may be spaced to form even ceiling panels in each room, but this would be rather expensive construction. Architectural require-

Rectangular Beams with steel in top and bottom



$$p = \frac{A_s}{bd} \qquad C' = A_s' f_s'$$

$$p' = \frac{A_s'}{bd} \qquad T = A_s f_s$$

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2(p+p')^2} - n(p+p')$$

$$L = \frac{k}{2}\left(1 - \frac{k}{3}\right) + \frac{np'}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)$$

$$R = p\left(1 - \frac{d'}{d}\right) - \frac{k^2}{2n(1-k)}\left(\frac{k}{3} - \frac{d'}{d}\right)$$

$$M_c = bd^2 f_c L \qquad f_c = \frac{M}{bd^2 L}$$

$$M_s = bd^2 f_s R \qquad f_s = \frac{M}{bd^2 R}$$

$$f_s' = n f_c \frac{k - \frac{d'}{d}}{k}$$

$$n = 15$$

Rectangular Beams with steel in top and bottom

$p' = 0.25p$					
	p	p'	k	j	R
$\frac{d'}{d} = 0.05$	0.005	0.00125	0.307	0.153	0.0045
	0.01	0.0025	0.394	0.202	0.0088
	0.015	0.00375	0.451	0.239	0.0130
	0.02	0.005	0.490	0.269	0.0172
	0.025	0.00625	0.521	0.295	0.0214
	0.03	0.0075	0.546	0.321	0.0256
$\frac{d'}{d} = 0.10$	0.005	0.00125	0.309	0.150	0.0045
	0.01	0.0025	0.398	0.198	0.0087
	0.015	0.00375	0.454	0.232	0.0125
	0.02	0.005	0.495	0.260	0.0170
	0.025	0.00625	0.525	0.285	0.0205
	0.03	0.0075	0.551	0.308	0.0251
$\frac{d'}{d} = 0.15$	0.005	0.00125	0.312	0.148	0.0044
	0.01	0.0025	0.402	0.194	0.0086
	0.015	0.00375	0.458	0.226	0.0127
	0.02	0.005	0.499	0.253	0.0167
	0.025	0.00625	0.530	0.275	0.0207
	0.03	0.0075	0.555	0.296	0.0247
$\frac{d'}{d} = 0.20$	0.005	0.00125	0.314	0.146	0.0044
	0.01	0.0025	0.404	0.190	0.0086
	0.015	0.00375	0.462	0.221	0.0126
	0.02	0.005	0.503	0.245	0.0165
	0.025	0.00625	0.534	0.266	0.0204
	0.03	0.0075	0.560	0.285	0.0243
$\frac{d'}{d} = 0.25$	0.005	0.00125	0.316	0.144	0.0045
	0.01	0.0025	0.408	0.187	0.0086
	0.015	0.00375	0.465	0.216	0.0125
	0.02	0.005	0.507	0.239	0.0164
	0.025	0.00625	0.539	0.259	0.0202
	0.03	0.075	0.565	0.275	0.0240

~~Rectangular Beams with~~ steel in top and bottom

$p' = 0.50p$					
	p	p'	k	L	R
$\frac{d'}{d} = 0.05$	0.005	0.0025	0.296	0.163	0.0046
	0.01	0.005	0.373	0.225	0.0090
	0.015	0.0075	0.421	0.275	0.0134
	0.02	0.01	0.454	0.320	0.0178
	0.025	0.0125	0.479	0.361	0.0222
	0.03	0.015	0.499	0.400	0.0266
$\frac{d'}{d} = 0.10$	0.005	0.0025	0.300	0.158	0.0045
	0.01	0.005	0.381	0.216	0.0088
	0.015	0.0075	0.428	0.261	0.0131
	0.02	0.01	0.462	0.302	0.0174
	0.025	0.0125	0.488	0.338	0.0215
	0.03	0.015	0.509	0.374	0.0258
$\frac{d'}{d} = 0.15$	0.005	0.0025	0.305	0.153	0.0044
	0.01	0.005	0.386	0.207	0.0087
	0.015	0.0075	0.436	0.249	0.0128
	0.02	0.01	0.471	0.285	0.0169
	0.025	0.0125	0.498	0.319	0.0210
	0.03	0.015	0.518	0.350	0.0251
$\frac{d'}{d} = 0.20$	0.005	0.0025	0.309	0.149	0.0044
	0.01	0.005	0.392	0.199	0.0086
	0.015	0.0075	0.442	0.238	0.0126
	0.02	0.01	0.479	0.271	0.0166
	0.025	0.0125	0.506	0.301	0.0206
	0.03	0.015	0.527	0.329	0.0245
$\frac{d'}{d} = 0.25$	0.005	0.0025	0.314	0.146	0.0044
	0.01	0.005	0.398	0.194	0.0085
	0.015	0.0075	0.450	0.229	0.0125
	0.02	0.01	0.486	0.258	0.0164
	0.025	0.0125	0.514	0.287	0.0202
	0.03	0.015	0.537	0.311	0.0240

Rectangular Beams with steel in top and bottom

$p' = p$

	p	p'	k	L	R
$\frac{d'}{d} = 0.05$	0.005	0.005	0.274	0.183	0.0046
	0.01	0.01	0.336	0.271	0.0092
	0.015	0.015	0.372	0.348	0.0138
	0.02	0.02	0.395	0.420	0.0184
	0.025	0.025	0.412	0.491	0.0230
	0.03	0.03	0.425	0.561	0.0275
$\frac{d'}{d} = 0.10$	0.005	0.005	0.284	0.172	0.0045
	0.01	0.01	0.349	0.250	0.0089
	0.015	0.015	0.386	0.318	0.0133
	0.02	0.02	0.410	0.381	0.0177
	0.025	0.025	0.428	0.442	0.0221
	0.03	0.03	0.442	0.503	0.0265
$\frac{d'}{d} = 0.15$	0.005	0.005	0.292	0.163	0.0044
	0.01	0.01	0.360	0.233	0.0087
	0.015	0.015	0.399	0.292	0.0129
	0.02	0.02	0.425	0.347	0.0171
	0.025	0.025	0.444	0.400	0.0213
	0.03	0.03	0.458	0.451	0.0255
$\frac{d'}{d} = 0.20$	0.005	0.005	0.299	0.154	0.0044
	0.01	0.01	0.371	0.218	0.0086
	0.015	0.015	0.411	0.270	0.0126
	0.02	0.02	0.439	0.318	0.0166
	0.025	0.025	0.460	0.364	0.0206
	0.03	0.03	0.475	0.408	0.0246
$\frac{d'}{d} = 0.25$	0.005	0.005	0.308	0.149	0.0045
	0.01	0.01	0.382	0.206	0.0085
	0.015	0.015	0.425	0.252	0.0124
	0.02	0.02	0.454	0.294	0.0162
	0.025	0.025	0.475	0.333	0.0200
	0.03	0.03	0.491	0.371	0.0238

Rectangular Beams with steel in top and bottom

$p' = 1.5\,p$					
	p	p'	k	L	R
$\frac{d'}{d} = 0.05$	0.005	0.0075	0.256	0.203	0.0046
	0.01	0.015	0.305	0.316	0.0093
	0.015	0.0225	0.332	0.420	0.0140
	0.02	0.03	0.349	0.521	0.0186
	0.025	0.0375	0.361	0.619	0.0232
	0.03	0.045	0.369	0.716	0.0280
$\frac{d'}{d} = 0.10$	0.005	0.0075	0.268	0.186	0.0045
	0.01	0.015	0.322	0.284	0.0090
	0.015	0.0225	0.351	0.374	0.0134
	0.02	0.03	0.369	0.458	0.0178
	0.025	0.0375	0.382	0.541	0.0222
	0.03	0.045	0.392	0.623	0.0267
$\frac{d'}{d} = 0.15$	0.005	0.0075	0.280	0.171	0.0045
	0.01	0.015	0.338	0.256	0.0087
	0.015	0.0225	0.369	0.333	0.0129
	0.02	0.03	0.389	0.404	0.0171
	0.025	0.0375	0.403	0.475	0.0213
	0.03	0.045	0.414	0.544	0.0256
$\frac{d'}{d} = 0.20$	0.005	0.0075	0.292	0.160	0.0044
	0.01	0.015	0.353	0.234	0.0086
	0.015	0.0225	0.386	0.298	0.0126
	0.02	0.03	0.409	0.360	0.0166
	0.025	0.0375	0.424	0.420	0.0206
	0.03	0.045	0.436	0.478	0.0246
$\frac{d'}{d} = 0.25$	0.005	0.0075	0.304	0.152	0.0044
	0.01	0.015	0.369	0.216	0.0084
	0.015	0.0225	0.404	0.271	0.0123
	0.02	0.03	0.428	0.325	0.0161
	0.025	0.0375	0.444	0.373	0.0199
	0.03	0.045	0.457	0.423	0.0237

ments may require a flat ceiling and for that reason concrete joists with an attached plastered ceiling is required. The concrete joist construction is a succession of small tee beams and is economical for spans up to about thirty feet for live floor loads less than a hundred pounds a square foot. The forms may be made of wood which are removable, or of metal either fixed or removable. The removalbe forms are prefereed because the concrete can be inspected. The sized of the metal forms have been standardized and are available in six, eight, ten, twelve and fourteen inch depths and twenty and thirty inch widths. The forms are also available inttwo special widths, ten and fifteen inches for filling odd spaces. The tables on pages forty-seven and forty-eight and the details on page fifty facilitate the design of concrete joists. The forms twenty inches wide are most widely used so the table of weights a square foot are based on this width. The following example will illustrate the application of the table on page forty-eight:

Moment 20,100 foot pounds

$b = 5$ inches

$s = 25$ inches

$t = 2$ inches

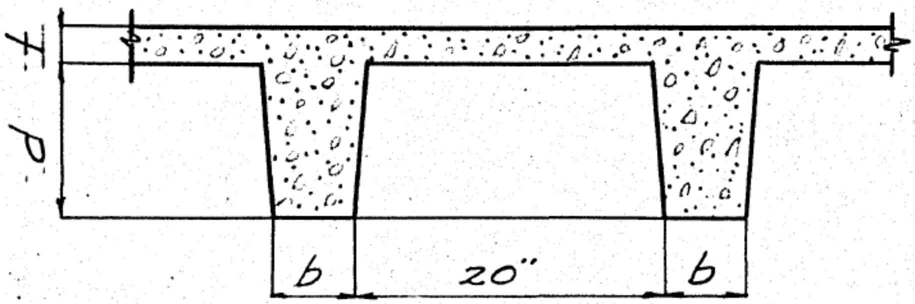
$d = 10\frac{1}{2}$ inches

Breadth of flange required

$b = \frac{20,100}{11,720} = 1.72$ feet (2.08 available)

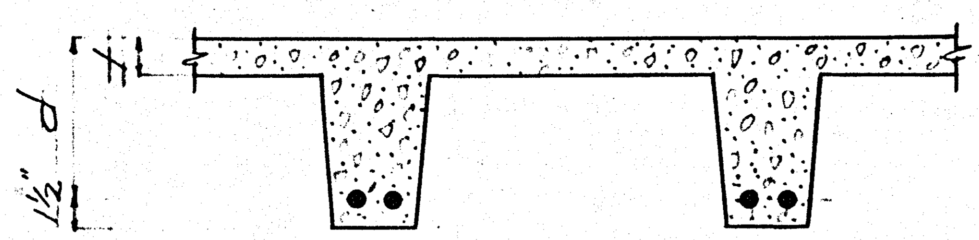
$A_s = .81 \times 1.72 = 1.39$ square inches

WEIGHTS OF CONCRETE JOISTS



"b" in inches	"d" in inches	"t" in inches 2	2½	3	3½	4
WEIGHT IN POUNDS A SQUARE FOOT OF CONCRETE JOISTS						
4.	6.	39.	45.	52.	58.	64.
	8.	45.	51.	57.	64.	70.
5.	6.	41.	47.	54.	60.	66.
	8.	48.	54.	60.	67.	73.
	10.	55.	61.	67.	72.	80.
	12.	61.	67.	73	79.	86.
	14.	65	71.	77.	83.	90.
6.	6.	43.	49.	56.	62.	68.
	8.	50.	56.	62.	69.	75.
	10.	59.	65.	71.	77.	84.
	12.	65.	71.	77.	83.	90.
	14.	70.	76.	82.	88.	95.
7.	10.	64.	70.	76.	82.	89.
	12.	70.	76.	82.	88.	94.
	14.	75.	81.	87.	93.	100.

REINFORCING FOR CONCRETE JOISTS



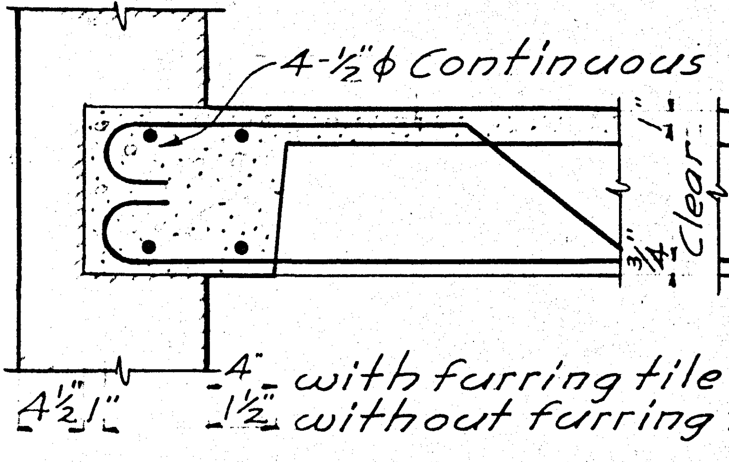
1. $f_s = 18000.$; $f_c = 800.$; $n = 15.$; $f_c = .4f'_c$; $f'_c = 2000.$
2. A_s – area of steel in square inches a foot width of flange.
3. M – moment in foot pounds a foot width of flange.
4. The values above the heavy zig-zag line are for solid slabs and the neutral axis lies in the flange.
5. The values below the heavy zig-zag line neglect compression in the stem and the neutral axis lies in stem.

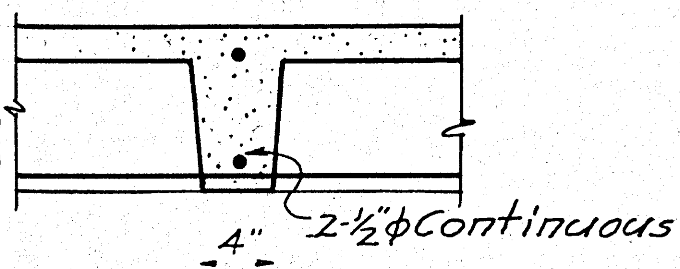
"d" in inches	"t" or thickness of flange in inches									
	2		2½		3		3½		4	
	A_s	M	A_s	M	A_s	M	A_s	M	A_s	M
6	.62	4890.	.64	4900.	.64	4900.	.64	4900.	.64	4900.
6½	.66	5630.	.69	5860.	.69	5860.	.69	5860.	.69	5860.
7	.69	6360.	.74	6740.	.75	6800.	.75	6800.	.75	6800.
7½	.71	7130.	.78	7690.	.80	7800.	.80	7800.	.80	7800.
8	.73	7870.	.81	8550.	.85	8850.	.85	8880.	.85	8880.
8½	.75	8650.	.84	9480.	.89	9950.	.91	10020.	.91	10020.
9	.77	9390.	.87	10410.	.93	11000.	.96	11230.	.96	11230.
9½	.79	10170.	.90	11350.	.97	12100.	1.01	12480.	1.01	12520.
10	.80	10930.	.92	12290.	1.00	13180.	1.05	13710.	1.07	13870.
10½	.81	11720.	.94	13250.	1.03	14280.	1.09	14960.	1.12	15300.
11	.82	12480.	.95	14200.	1.05	15400.	1.12	16200.	1.16	16700.
11½	.83	13270.	.97	15150.	1.08	16450.	1.16	17500.	1.20	18150.
12	.84	14050.	.99	16100.	1.10	17700.	1.19	18800.	1.24	19600.
12½	.85	14830.	1.00	17100.	1.12	18850.	1.22	20100.	1.28	21050.
13	.86	15600.	1.01	18100.	1.14	20000.	1.24	21400.	1.31	22500.
13½	.87	16400.	1.03	19050.	1.16	21150.	1.26	22750.	1.34	23950.
14	.88	17200.	1.04	20000.	1.17	22300.	1.28	24100.	1.37	25400.
14½	.89	18000.	1.05	20950.	1.19	23450.	1.30	25450.	1.40	26900.
15			1.06	21900.	1.20	24600.	1.32	26800.	1.42	28400.
15½					1.21	25850.	1.35	28100.	1.45	29950.
16							1.36	29400.	1.47	31500.
16½									1.49	34 000.

Referring to the table on page twenty-eight, one, seven-eights round bar and one, one inch round bar are required. The shear and bond should be investigated to complete the design. The unit shear often exceeds the allowable limit and in such cases the ends of the joists are flared by the use of tapered forms. The length of the taper has been standardized and is three feet. The typical details on page fifty conform to accepted practice. The purpose of the bridging is to stiffen the floor system and distribute any concentrated load over to adjacent joists.

The foregoing slab and beam tables are based on the usual design stresses, but occasionally some building code permits, or requires other design stresses. The tables on pages fifty-one and fifty-two contain co-effecients for computing slabs and simple beams and cover the permitted range of design stresses. The designer should devise new slab and beam tables to use for any large structural design based on stress other than the customary ones.

CONCRETE JOIST DETAILS

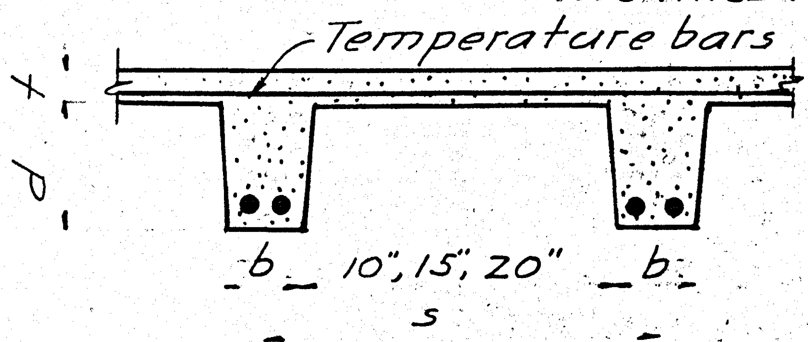




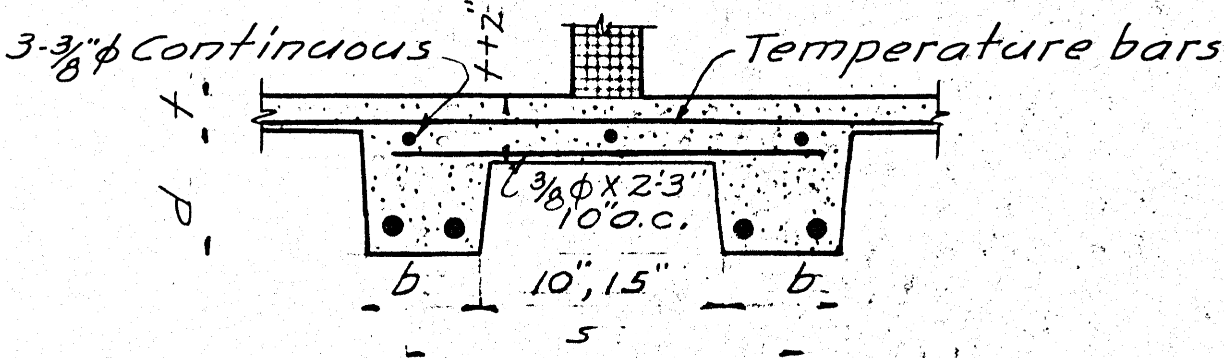
CONTINUOUS WALL BEAM AROUND BUILDING

BRIDGING

One line for spans 14'-0" to 21'-0"
Two lines for spans over 21'-0"



TYPICAL JOIST SECTION



JOISTS UNDER PARTITIONS

Note:-
The bars in the joists under partitions are the next size larger than those in the adjacent joists.

Values of k, j, p & R for Rectangular Beams and Slabs

n = 12

$f_s = 16{,}000.$					
f_c	600.	650.	700.	750.	800.
p	0.0058	0.0067	0.0075	0.0084	0.0094
k	0.310	0.328	0.344	0.360	0.375
j	0.897	0.891	0.885	0.880	0.875
R	83.5	94.9	106.7	118.8	131.3
$f_s = 18{,}000.$					
f_c	600.	650.	700.	750.	800.
p	0.0048	0.0055	0.0062	0.0069	0.0077
k	0.286	0.302	0.318	0.333	0.348
j	0.905	0.899	0.894	0.889	0.884
R	77.6	88.4	99.6	111.1	123.0
$f_s = 20{,}000.$					
f_c	600.	650.	700.	750.	800.
p	0.0040	0.0046	0.0052	0.0058	0.0065
k	0.265	0.281	0.296	0.310	0.324
j	0.912	0.906	0.901	0.897	0.892
R	72.6	82.7	93.3	104.3	115.6

$$k = \frac{1}{1+\frac{f_s}{n f_c}} = \sqrt{2pn+\overline{pn}^2} - pn$$

$$j = 1 - \frac{k}{3}$$

$$p = \frac{f_c k}{2 f_s} = \frac{1}{2\frac{f_s}{f_c}\left(\frac{f_s}{n f_c}+1\right)} = \frac{A_s}{bd}$$

$$R = \frac{1}{2} f_c k j = p f_s j = \frac{M}{bd^2}$$

$$n = \frac{\frac{f_s}{d}}{\frac{f_c}{d_1}} = \frac{E_s}{E_c} = 12$$

$$d = d_1$$

k = ratio of depth of neutral axis to depth "d".

f_s = unit tensile stress in steel

f_c = unit compressive stress in concrete

p = steel ratio

j = ratio of lever arm of resisting couple to depth "d"

b = breadth of beam.

d = depth of beam to center of steel.

M = bending moment or moment of resistance.

A_s = area of steel.

Values of k, j, p & R for Rectangular Beams and Slabs

$n = 15$

$f_s = 16{,}000.$					
f_c	600.	650.	700.	750.	800.
p	0.0068	0.0077	0.0087	0.0097	0.0107
k	0.360	0.379	0.396	0.413	0.429
j	0.880	0.874	0.868	0.862	0.857
R	95.0	107.5	120.4	133.5	146.9
$f_s = 18{,}000.$					
f_c	600.	650.	700.	750.	800.
p	0.0056	0.0063	0.0072	0.0080	0.0089
k	0.333	0.351	0.368	0.385	0.400
j	0.889	0.883	0.877	0.872	0.867
R	88.9	100.8	113.1	125.7	138.7
$f_s = 20{,}000.$					
f_c	600.	650.	700.	750.	800.
p	0.0047	0.0053	0.0060	0.0068	0.0075
k	0.310	0.328	0.344	0.360	0.375
j	0.897	0.891	0.885	0.880	0.875
R	83.5	94.9	106.6	118.8	131.2

$$k = \frac{1}{1+\frac{f_s}{n f_c}} = \sqrt{2pn+\overline{pn}^2} - pn$$

$$j = 1 - \frac{k}{3}$$

$$p = \frac{f_c k}{2 f_s} = \frac{1}{2\frac{f_s}{f_c}\left(\frac{f_s}{n f_c}+1\right)} = \frac{A_s}{bd}$$

$$R = \frac{1}{2} f_c k j = p f_s j = \frac{M}{bd^2}$$

$$n = \frac{\frac{f_s}{d}}{\frac{f_c}{d_1}} = \frac{E_s}{E_c} = 15$$

$$d = d_1$$

k = ratio of depth of neutral axis to depth "d"

f_s = unit tensile stress in steel

f_c = unit compressive stress in concrete

p = steel ratio

j = ratio of lever arm of resisting couple to depth "d"

b = breadth of beam

d = depth of beam to center of steel

M = bending moment or moment of resistance

A_s = area of steel

The design of concrete columns with spiral looping are rather tedious unless a good set of tables are available. The tables on pages fifty-four, fifty-five and fifty-six comply with the 1924 Report of the Joint Committee for safe axial loads. The formula from which the tables have been devised have also been adopted by the American Concrete Institute and the Concrete Reinforcing Steel Institute. The variable in the formula p and f from which $\frac{P}{A}$ can be computed for the values limited by the Joint Committee. The core areas are given for various diameters and the table on page fifty-six gives the standard spiral hooping sizes recommended by the United States Department of Commerce.

The application of the tables is quite simple and they are indeed labor saving. It is only necessary to divide the axial column load by the selected core area and find the corresponding value of p. The selected core size will not be a wise one if the value falls outside of the table and this fact is immediately apparent. The core size will either be too large as indicated by the low ratio of vertical steel, or the ratio of the vertical steel may be too much for greatest economy. The area of vertical steel required can be obtained by multiplying the ratio by the core area. The ratio of the spiral hooping shall not be less than one-fourth the vertical reinforcing according to the Joint Committee.

CONCRETE COLUMNS

The 1924 Report of the Joint Committee gives a formula for the total "safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core." This formula has also been adopted by the American Concrete Institute and the Concrete Reinforcing Steel Institute.

The formula is written as follows;

$P = A[1+(n-1)p] f_c$

$f_c = 300 + (.10 + 4p) f_c'$ (determined by tests)

$P = A[1+(n-1)p][300+(.10+4p) f_c']$

P = total load A = area of core

p = ratio of vertical steel area to core area

f_c = unit stress on concrete within core

f_c' = ultimate strength of 28 day concrete

n = ratio of steel to concrete modulus

The Joint Committee recomendation states that "spiral reinforcement shall not be less than one-fourth the volumne of the longitudinal reinforcement." The value "p" can be divided by 4 to get the spiral hooping ratio.

The values of "p" vary from 1 percent to 6 per cent which are specified by the Joint Committee as minimum and maximum allowable values.

COLUMN TABLE

Values of $\frac{P}{A}$ for Columns. $\frac{P}{A} = [1+(n-1)p][300+(.10+4p)f'_c]$

$n = 15$

"p" in Perct.	Values of $\frac{P}{A}$ 2000. lb. Conc.	2500. lb. Conc.	3000. lb. Conc.	"p" in Perct.	Values of $\frac{P}{A}$ 2000. lb. Conc.	2500. lb. Conc.	3000. lb. Conc.	"p" in Perct.	Values of $\frac{P}{A}$ 2000. lb. Conc.	2500. lb. Conc.	3000. lb. Conc.
1.00	661.	721.	785.	3.00	1051.	1130.	1219.	5.00	1530.	1627.	1740.
1.05	670.	731.	795.	3.05	1062.	1142.	1231.	5.05	1543.	1641.	1754.
1.10	679.	740.	804.	3.10	1073.	1153.	1243.	5.10	1556.	1655.	1768.
1.15	687.	749.	814.	3.15	1084.	1165.	1255.	5.15	1570.	1668.	1783.
1.20	696.	758.	824.	3.20	1095.	1176.	1267.	5.20	1583.	1682.	1797.
1.25	705.	768.	834.	3.25	1106.	1188.	1280.	5.25	1596.	1696.	1811.
1.30	714.	777.	844.	3.30	1117.	1199.	1292.	5.30	1610.	1710.	1826.
1.35	723.	787.	855.	3.35	1128.	1211.	1304.	5.35	1623.	1724.	1840.
1.40	732.	796.	865.	3.40	1139.	1223.	1316.	5.40	1637.	1737.	1855.
1.45	741.	806.	875.	3.45	1151.	1235.	1329.	5.45	1650.	1751.	1869.
1.50	750.	815.	885.	3.50	1162.	1246.	1341.	5.50	1664.	1766.	1884.
1.55	759.	825.	896.	3.55	1174.	1258.	1354.	5.55	1677.	1780.	1898.
1.60	769.	835.	906.	3.60	1185.	1270.	1366.	5.60	1691.	1794.	1913.
1.65	778.	845.	917.	3.65	1197.	1282.	1379.	5.65	1705.	1808.	1928.
1.70	787.	855.	927.	3.70	1208.	1294.	1392.	5.70	1719.	1822.	1943.
1.75	797.	865.	938.	3.75	1220.	1307.	1404.	5.75	1733.	1837.	1958.
1.80	806.	875.	948.	3.80	1232.	1319.	1417.	5.80	1747.	1851.	1973.
1.85	816.	885.	959.	3.85	1244.	1331.	1430.	5.85	1761.	1865.	1988.
1.90	825.	895.	970.	3.90	1255.	1343.	1443.	5.90	1775.	1880.	2003.
1.95	835.	905.	980.	3.95	1267.	1356.	1456.	5.95	1789.	1894.	2018.
2.00	845.	915.	991.	4.00	1279.	1368.	1469.	6.00	1803.	1909.	2033.
2.05	855.	925.	1002.	4.05	1291.	1380.	1482.				
2.10	864.	936.	1013.	4.10	1303.	1393.	1495.				
2.15	874.	946.	1024.	4.15	1315.	1406.	1508.				
2.20	884.	956.	1035.	4.20	1328.	1418.	1521.				
2.25	894.	967.	1046.	4.25	1340.	1431.	1535.				
2.30	904.	977.	1057.	4.30	1352.	1444.	1548.				
2.35	914.	988.	1069.	4.35	1364.	1456.	1561.				
2.40	925.	999.	1080.	4.40	1377.	1469.	1575.				
2.45	935.	1009.	1091.	4.45	1389.	1482.	1588.				
2.50	945.	1020.	1102.	4.50	1402.	1495.	1602.				
2.55	955.	1031.	1114.	4.55	1414.	1508.	1615.				
2.60	966.	1042.	1125.	4.60	1427.	1521.	1629.				
2.65	976.	1053.	1137	4.65	1440.	1534.	1643.				
2.70	987.	1064.	1149.	4.70	1452.	1547.	1656.				
2.75	997.	1075.	1160.	4.75	1465.	1561.	1670.				
2.80	1008.	1086.	1172.	4.80	1478.	1574.	1684.				
2.85	1018.	1097.	1184.	4.85	1491.	1587.	1698.				
2.90	1029.	1108.	1195.	4.90	1504.	1601.	1712.				
2.95	1040.	1119.	1207.	4.95	1517.	1614.	1726.				

Core Dia.	Core Area	Core Dia.	Core Area
9."	63.6	26.	530.9
10.	78.5	27.	572.6
11.	95.0	28.	615.7
12.	113.1	29.	660.5
13.	132.7	30.	706.9
14.	153.9	31.	754.8
15.	176.7	32.	804.2
16.	201.1	33.	855.3
17.	227.0	34.	907.9
18.	254.5	35.	962.1
19.	283.5	36.	1017.9
20.	314.2	37.	1075.2
21.	346.4	38.	1134.1
22.	380.1	39.	1194.6
23.	415.5		
24.	452.4		
25.	490.9		

COLUMN SPIRALS

Pitch of spiral in inches for given percentage and various bar sizes

Diameter of core in inches	Size of bars in inches										
	1/4 ●				3/8 ●				1/2 ●		5/8 ●
	Percentage of spiral reinforcement										
	.50	.75	1.00	1.50	.50	.75	1.00	1.50	1.00	1.50	1.50
9.				1 3/8							
10.				1 3/8							
11.			1 5/8	1 1/2							
12.			1 5/8					2 1/4			
13.		2	1 1/2					2 1/4			
14.	2 3/4	1 7/8	1 3/8					2 1/8			
15.	2 5/8	1 3/4					2 3/4	2			
16.	2 1/2	1 5/8					2 3/4	1 7/8			
17.	2 1/4	1 1/2					2 5/8	1 3/4			
18.	2 1/8	1 1/2					2 1/2	1 5/8		2 7/8	
19.	2 1/8					3	2 3/8	1 1/2		2 3/4	
20.	2					3	2 1/4	1 1/2		2 5/8	
21.	1 7/8					2 3/4	2 1/8			2 1/2	
22.	1 3/4					2 5/8	2			2 3/8	
23.	1 3/4					2 1/2	1 7/8			2 1/4	
24.	1 5/8					2 1/2	1 7/8			2 1/8	
25.	1 5/8					2 3/8	1 3/4			2 1/8	
26.	1 1/2					2 1/4	1 3/4		3	2	
27.	1 1/2					2 1/8	1 5/8		2 7/8	2	3
28.	1 1/2					2 1/8	1 5/8		2 3/4	1 7/8	2 7/8
29.					3	2	1 1/2		2 3/4	1 3/4	2 3/4
30.					3	2	1 1/2		2 3/4	1 3/4	2 3/4
31.					2 7/8	1 7/8			2 1/2	1 3/4	2 5/8
32.					2 3/4	1 7/8			2 1/2	1 5/8	2 1/2
33.					2 5/8	1 3/4			2 3/8	1 5/8	2 1/2
34.					2 5/8	1 3/4			2 1/4	1 1/2	2 3/8
35.					2 1/2	1 5/8			2 1/4	1 1/2	2 1/4
36.					2 1/2	1 5/8			2 1/8	1 1/2	2 1/4
37.					2 3/8	1 5/8			2 1/8		2 1/8
38.					2 3/8	1 1/2			2 1/8		2 1/8
39.					2 1/4	1 1/2			2		2 1/8

The size of the spiral hooping bars and the pitch for various core diameters can be selected directly from the table on page fifty-six. The spiral looping "overlap" in a few cases which allows a choice of two bar sizes to give the required hooping.

The following example will show the use of the tables in actual design:

$P = 390{,}000$ pounds

Diameter 20 inches

$f_c =$ 2000 pounds in 28 days

$\frac{P}{A} = \frac{390{,}000}{314.2} = 1238$ pounds per square inch

$p =$ 3.85 (Table on page fifty-five)

$A_s = 319.2 \times .0385 = 12.12$ square inches

Spiral hooping $p = \frac{3.85}{4} \approx .96$

3/8" ϕ spiral 23/8" pitch (Table on page fifty-six)

The exterior columns should be the same width on the exterior face for the entire height of the building. The cross section of concrete columns should be the same for at least two stories and for low buildings of not more than four stories it is practical to hold to the same section the entire height. The vertical reinforcing would of course decrease in the upper stories of the building. The column size should be in even inches and decrease in size by two inches. The column spacing should be unvarying the entire height of the building and if possible equal. The vertical bars should be as large as practicable and spiral hooping should be used in preference to isolated ties. The vertical bars should be spliced twenty-four bar diameters at floor lines but not less than eighteen inches by projecting the bars above the slab from the columns below.

The majority of column footings are square and of the block type. The design of these footings is rather tedious but the bending moment can be readily determined by referring to the diagram on page fifty-nine. The value C is multiplied by the column load and the result is given in inch pounds. The weight of the footing does not affect the bending moment in the footing. The following example shows the application in actual design:

P = 94,400 pounds

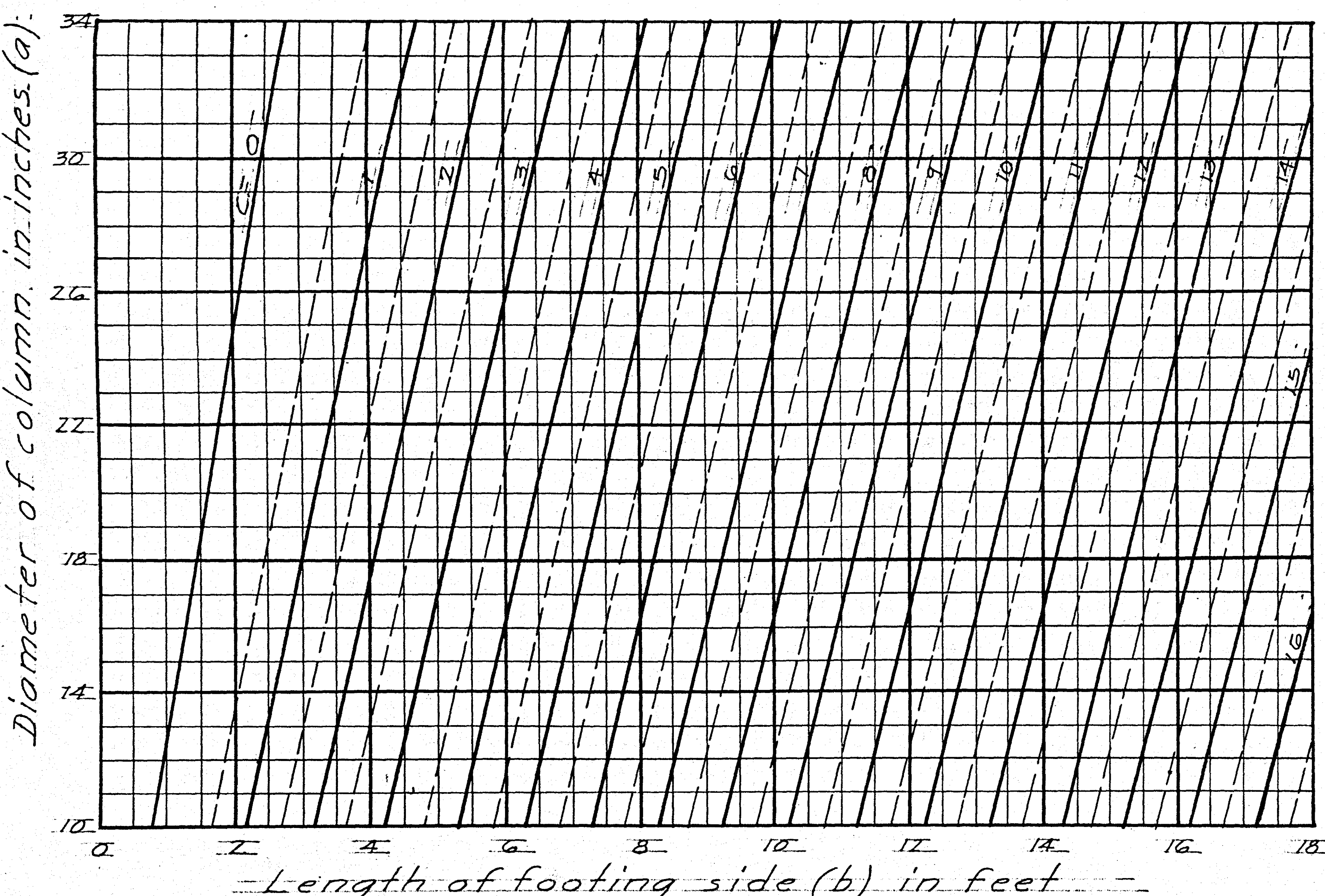
Column, 14 inches square

Size of footing, 5 feet square

The value C is 3.3 (from the diagram on page fifty-nine).

M = CP = 3.3 x 94,400 = 312,000 inch pounds in footing at each face of column. The shear, punching, and bond must be investigated in the usual manner to complete the design. The footing should be connected to the column with dowels of the same size and number as vertical bars in the column. The projecting portion of

VALUES OF "C" FOR SQUARE FOOTINGS



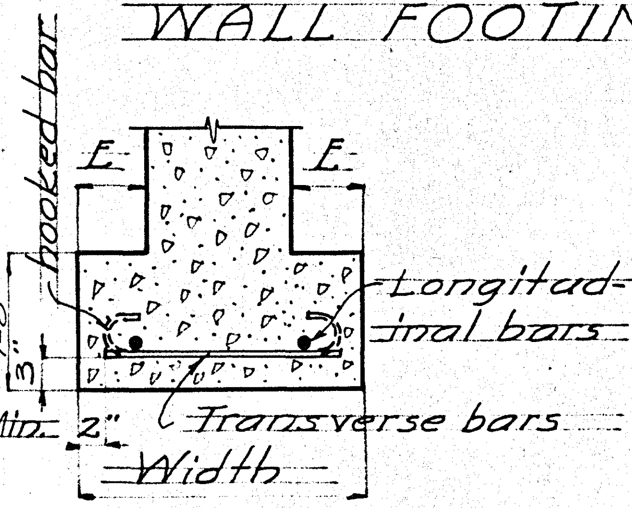
Square footings with square or round columns

$M = CP$ (in.-lb.)

wall footing is designed as a cantilever slab and the results can readily be tabulated. The table on page sixty-one gives the reinforcing required for various projections and various unit soil loads. The longitudinal reinforcing serve as spaces for the transverse bars and also as temperature and shrinkage reinforcing. The concrete protection on footing reinforcing should be three inches or the bars may be damaged by excessive corrosion.

The masses of concrete in the usual building are too large to expand or contract freely resulting from temperature or moisture changes. The tendency to expand and contract develop stresses in the concrete and unless the concrete is reinforced to resist these stresses cracks will occur. The co-efficient of expansion of steel and concrete are approximately equal, which is very desirable. The purpose of reinforcing for temperature and shrinkage is to prevent cracks from localizing, but rather to prevent them all together or render them numerous and very small. The reinforcing bars also tend to distribute the stresses evenly along the mass of concrete. The Joint Committee specifies the ratios of steel required to resist expansion and shrinkage stress to be placed at right angle to the principal reinforcing. The tables on pages sixty-two and sixty-three are based on these ratios and are very convenient to use. The horizontal bars in the walls are spaced uniformly twelve inches so the bars can be lapped regardless

WALL FOOTING REINFORCING



1. f_s = 18000.# or less
2. f_c = 800.# or less
3. v = 40.# or less above heavy zigzag line and ends of transverse bars are not hooked.
4. v = 41# to 60.# between heavy zigzag lines and ends of transverse bars are hooked.

TRANSVERSE BARS

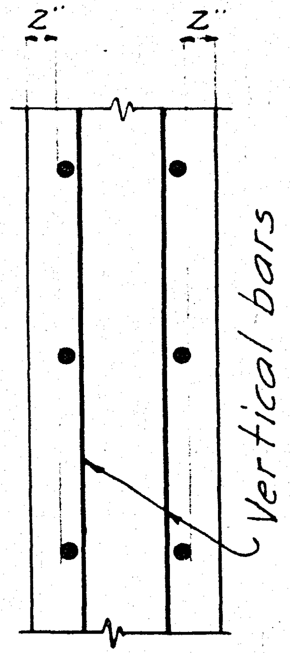
"E" in inches	Unit soil load in pounds a square foot								"E" in inches
	2000.		3000.		4000.		6000.		
	Size	Spacing	Size	Spacing	Size	Spacing	Size	Spacing	
6	3/8" φ	24" o.c.	3/8" φ	24" o.c.	3/8" φ	24" o.c.	3/8" φ	24" o.c.	6
8	3/8" φ	24" o.c.	3/8" φ	24" o.c.	3/8" φ	24" o.c.	1/2" □	4" o.c.	8
10	3/8" φ	24" o.c.	3/8" φ	4" o.c.	1/2" φ	4" o.c.	3/4" φ	4" o.c.	10
12	3/8 φ	5" o.c.	1/2" φ	5" o.c.	1/2" □	4" o.c.			12
14	1/2" φ	6" o.c.	1/2" □	5" o.c.	1/2" □	4" o.c.			14
16	1/2" φ	5" o.c.	1/2" □	4" o.c.	3/4" φ	4" o.c.			16
18	1/2" φ	5" o.c.	1/2" □	4" o.c.					18
20	1/2" □	5" o.c.	1/2" □	3" o.c.					20
22	1/2" □	5" o.c.							22
24	1/2" □	4" o.c.							24
26	1/2" □	4" o.c.							26
28	1/2" □	4" o.c.							28

LONGITUDINAL BARS

Width	Bars
2'-0" or less	2 - 5/8" φ
2'-1" to 3'-0"	3 - 5/8" φ
3'-1" to 4'-0"	4 - 5/8" φ
4'-1" to 5'-0"	5 - 5/8" φ
5'-1" to 6'-0"	6 - 5/8" φ

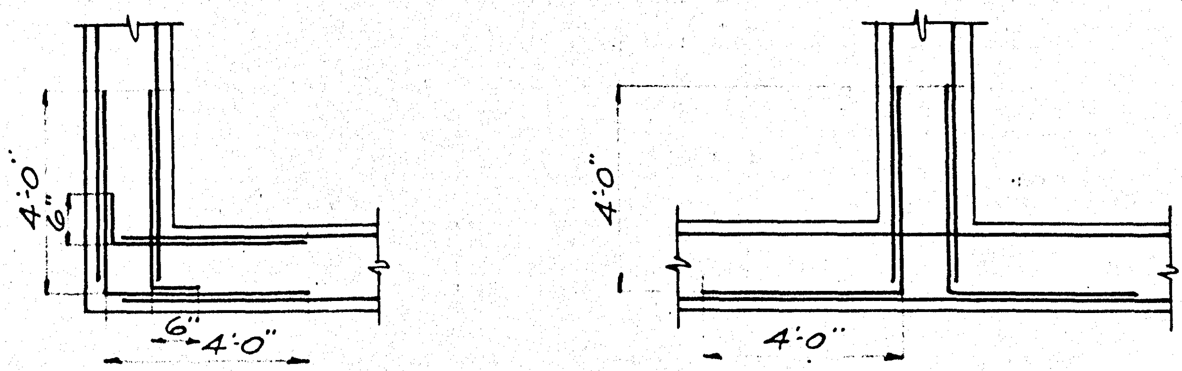
5. The transverse bars above the heavy dotted zigzag line are for longitudinal bar spacers since 40# tension a square inch is allowed in the concrete.
6. u = 100# or less.

REINFORCING FOR SHRINKAGE AND TEMPERATURE IN CONCRETE WALLS				
Thickness of wall	Bars at each face of wall			
	Horizontal bars		Vertical bars	
	Size	Spacing	Size	Spacing
8" or less	3/8 φ	12" o.c.	3/8 φ	24" o.c.
9" to 12"	1/2 φ	12" o.c.	3/8 φ	24" o.c.
13" to 16"	1/2 □	12" o.c.	3/8 φ	18" o.c.
17" to 20"	5/8 φ	12" o.c.	3/8 φ	18" o.c.
21" to 24"	3/4 φ	12" o.c.	1/2 φ	18" o.c.
25" to 28"	3/4 φ	12" o.c.	1/2 φ	18" o.c.
29" to 32"	7/8 φ	12" o.c.	1/2 φ	18" o.c.



NOTE: $p = .0025$

1. The ends of all horizontal bars shall lap at least 24 diameters but not less than 18 inches.
2. The horizontal bars shall lap 4 feet at corners of walls and intersections of walls as shown below.
3. The horizontal bars shall start 6 inches above top of footing and stop 2 inches below top of wall.
4. The horizontal and vertical bars shall be securely wired together at every point of intersection.



CORNER

INTERSECTION

LAPPING OF HORIZONTAL BARS

REINFORCING FOR SHRINKAGE AND TEMPERATURE IN SLABS

1. Reinforcing for shrinkage and temperature stresses normal to the principal reinforcing shall be provided in concrete construction where the principal reinforcing extends in one direction only. The spacing of the bars shall not exceed 18 inches nor 5 times the slab thickness.
2. "T" Slab thickness in inches.
3. "A_s" Area of steel in square inches a foot width of slab.

FLOOR SLABS ($p = 0.002$)				ROOF SLABS ($p = 0.0025$)			
T	A_s	Reinforcing		T	A_s	Reinforcing	
		Size	Spacing			Size	Spacing
2.0	.048	¼"φ	10" o.c.	2.0	.060	¼"φ	10" o.c.
2.5	.060	¼"φ	10" o.c.	2.5	.075	⅜"φ	12" o.c.
3.0	.072	⅜"φ	15" o.c.	3.0	.090	⅜"φ	12" o.c.
3.5	.084	⅜"φ	14" o.c.	3.5	.105	⅜"φ	12" o.c.
4.0	.096	⅜"φ	12" o.c.	4.0	.120	⅜"φ	11" o.c.
4.5	.108	⅜"φ	12" o.c.	4.5	.135	⅜"φ	10" o.c.
5.0	.120	⅜"φ	11" o.c.	5.0	.150	⅜"φ	9" o.c.
5.5	.132	⅜"φ	10" o.c.	5.5	.165	⅜"φ	8" o.c.
6.0	.144	⅜"φ	9" o.c.	6.0	.180	⅜"φ	7" o.c.
6.5	.156	⅜"φ	8" o.c.	6.5	.195	½"φ	12" o.c.
7.0	.168	⅜"φ	8" o.c.	7.0	.210	½"φ	11" o.c.
7.5	.180	⅜"φ	7" o.c.	7.5	.225	½"φ	10" o.c.
8.0	.192	⅜"φ	7" o.c.	8.0	.240	½"φ	10" o.c.
8.5	.204	⅜"φ	6" o.c.	8.5	.255	½"φ	9" o.c.
9.0	.216	⅜"φ	6" o.c.	9.0	.270	½"φ	9" o.c.
10.0	.240	½"φ	10" o.c.	10.0	.300	½"□	10" o.c.
11.0	.264	½"φ	9" o.c.	11.0	.330	½"□	9" o.c.
12.0	.288	½"φ	8" o.c.	12.0	.360	½"□	8" o.c.

o.c. - on centers

of the adjacent wall thickness. The vertical bars in walls serve as spaces and supports for the horizontal bars.

The figure sixteen on page sixty-five shows the correct manner of placing reinforcing bars in a beam over a support. The bars in no case should be tied together over the support, but separated like at the center of the beam. The reinforcing in all cases must be securely held the specified distance above the forms and tied together at every point of intersection. The reinforcing bars should be shop fabricated rather than field fabricated since the operation can be performed with greater accuracy. Reinforcing should never be placed at angles so that any stress included in the bar will spall the concrete. Figure seventeen illustrates a typical core of improper arrangement of reinforcing bars for a stair. The bars shown in figure eighteen will not spall the concrete when the stair is loaded. The figures nineteen and twenty on page sixty-six show the incorrect and correct of splicing vertical column bars at the floor line.

The complete set of architectural construction drawings usually include the general structural plans prepared by some engineer employed by the firm, or by a firm of engineers. The structural information should be on separate plans and not drawn on architectural floor plans. General structural plans include foundation plan, floor and roof framing plans, the necessary details and the

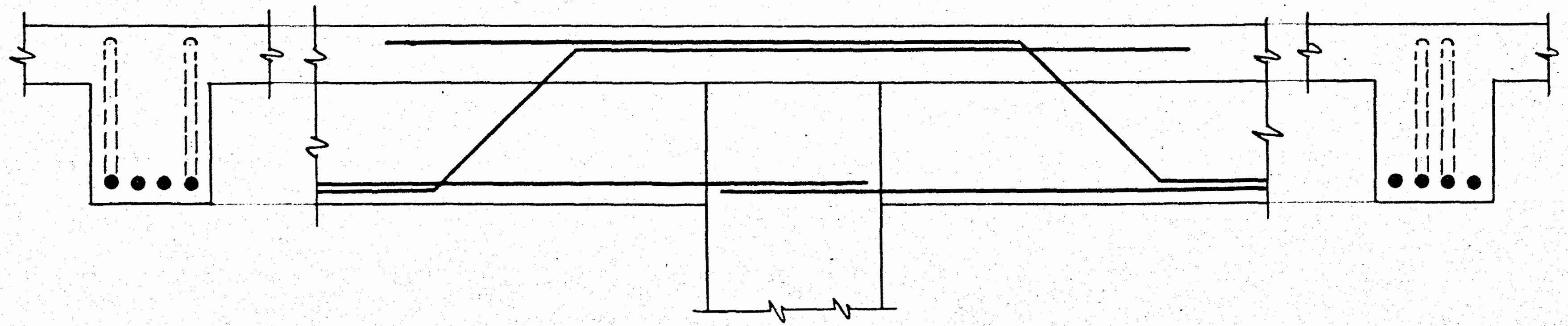


FIGURE 16

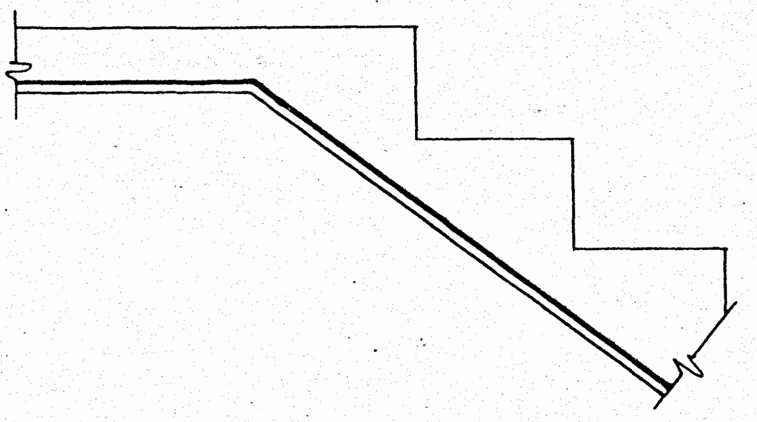


FIGURE 17

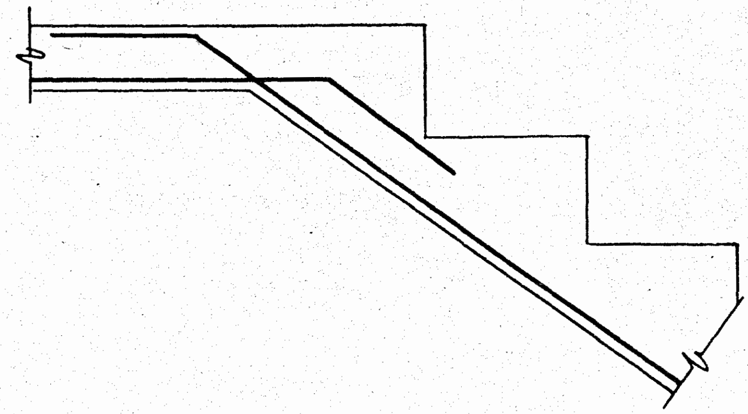


FIGURE 18

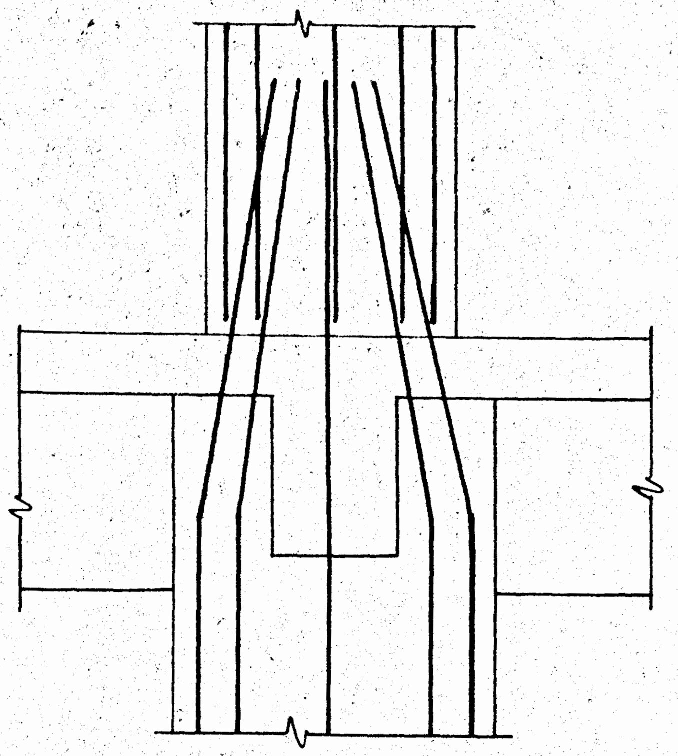


FIGURE 19

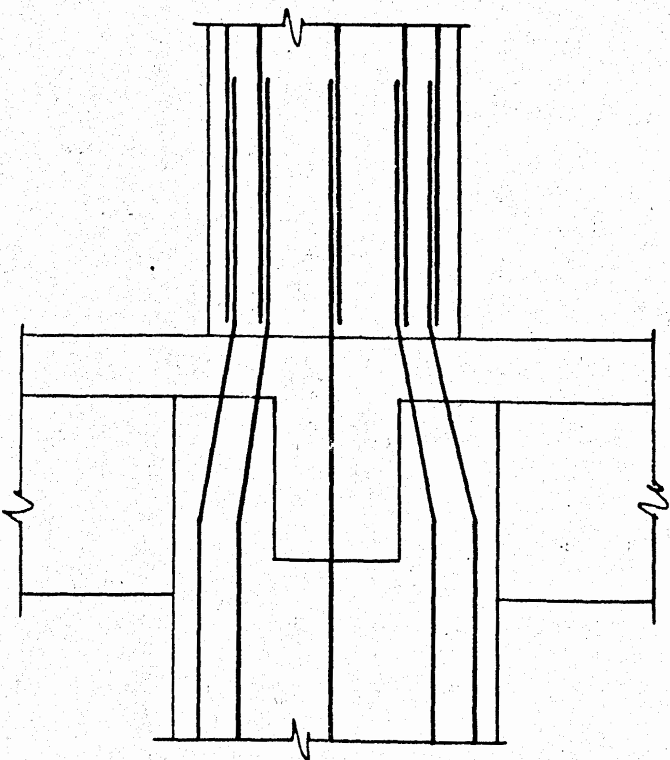


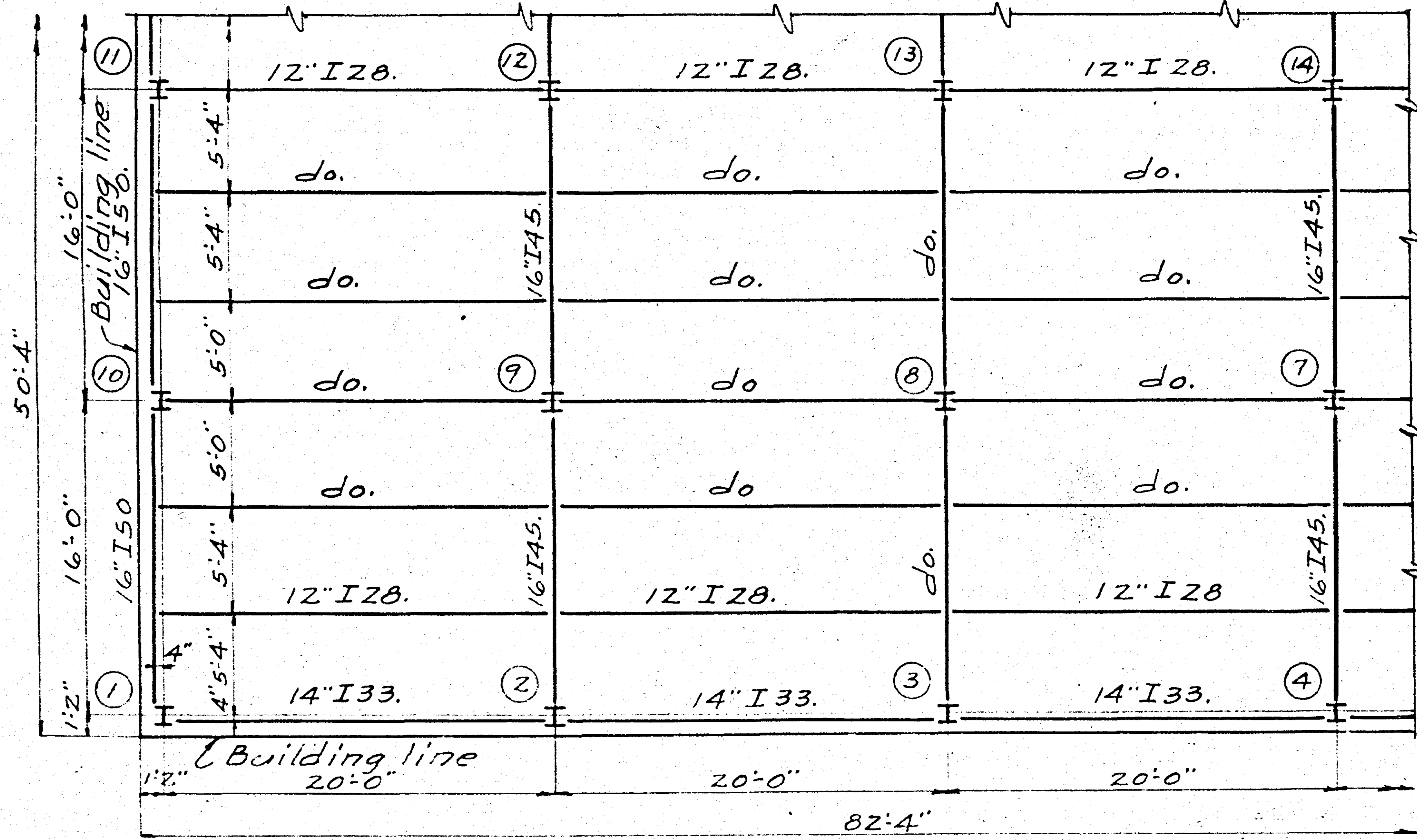
FIGURE 20

(Spiral hooping not shown)

schedules. Architectural features are omitted from these plans including the non-load bearing partitions.

The partial structural steel framing plan shown on page sixty-eight is quite typical. The concrete floor system is omitted on these plans, but are covered on other plans. The columns are each given a separate identification number and is the same on all floors. The structural plans are preferable drawn at the same scale as the architectural floor plans which is usually one-eight inch to a foot. The center lines of all columns and the location of all beams must be dimensioned. The elevations of the tops of all beams should be given in reference to the finished floor line either by a general note or separately on each beam. The plans cover the horizontal framing and the column schedule shown on page sixty-nine the vertical members. The column schedules shown locate the floor lines and other information necessary for the contractor to properly prepare working drawings. The structural plans should include sufficient details to cover special cores.

The partial concrete framing plan on page seventy illustrates the accepted conventions and as previously stated the column center lines of all columns and location of all beams should be dimensioned. The columns are given individual identification numbers and the beams a number prefixed by a letter. The slabs are



PARTIAL STRUCTURAL STEEL FLOOR FRAMING PLAN
(Concrete not shown)
Scale 1/8" = 1'-0"

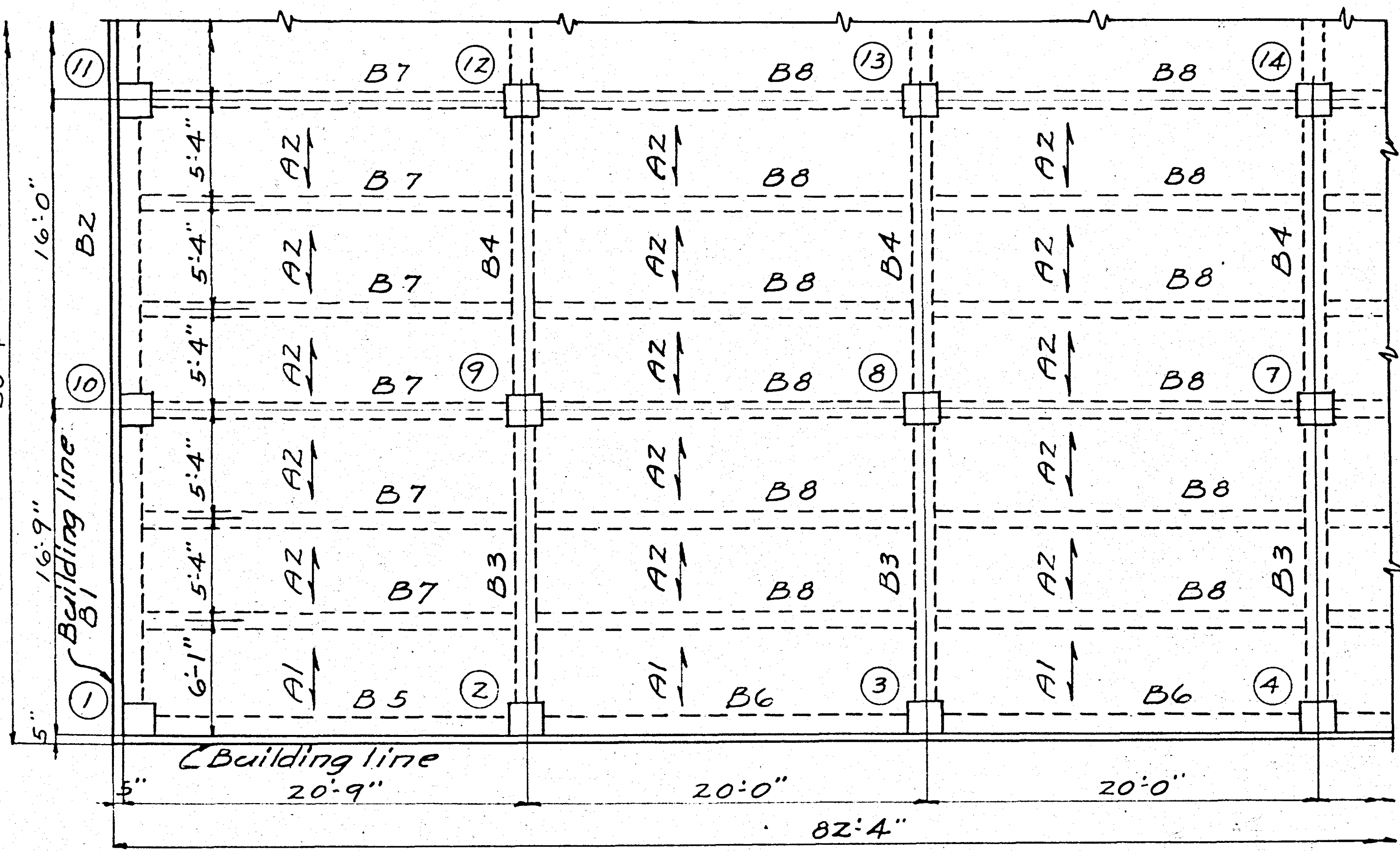
STEEL COLUMN AND FOOTING SCHEDULE

	Column numbers			
	1	2	3	4
Roof				
Elev. 125.50 11'-0" Splice 1'-6" First floor	8" H 24.			
Elev. 114.50 12'-0" Basement	8" H 35.			
Elev. 102.50				
Elev. of top of base plate	102.29			
Size of base plate	16" x 1½" x 1'-4"			
Size of footing	6'-0 x 6'-0" x 1'-9"			
Footing reinforcing	13-½" x 5'-6" two ways			
Elev. of bottom of footing	99.50			

(Anchor bolts and clip angles are detailed)

END BEARING SCHEDULE

Mark	Material		
	Plate	Angles	Anchor bolts
P1	8 x ½ x 0'-8"	2 ∟ 3x3x⅜x0'-8"	2-¾ϕ x 1'-4"
P2	10 x ⅝ x 1'-0"		



PARTIAL REINFORCED CONCRETE FLOOR

FRAMING PLAN

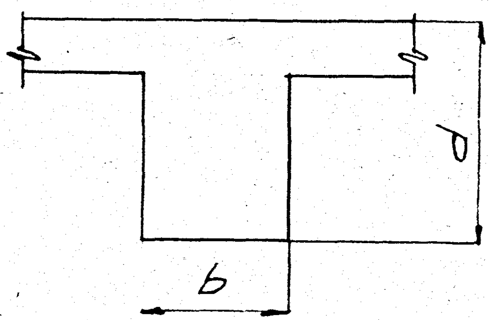
Scale 1/8" = 1'0"

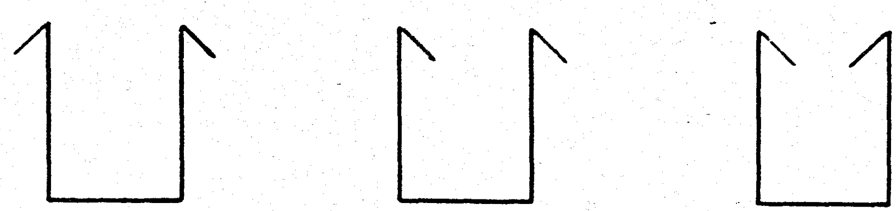
CONCRETE COLUMN AND FOOTING SCHEDULE

	Column numbers				
	1	2	3	4	
Roof					
Elev. 125.50 — 11'-0" — First floor	12"x12" Col. 9" x 9" core, 4-3/4φ & 3/8φ ties 8" o.c.				
Elev. 114.50 — 12'-0" — Basement	12"x12" Col., 9"φ core, 6-3/4φ & 1/4φ spiral 1½" pitch				
Elev. 102.50					
Column dowels	6-3/4φ x 3'-3"				
Size of footing	6'-0"x6'-0" x 1'-9"				
Footing reinforcing	13-½"□x5'-6" two ways				
Elev. of bottom of footing	99.50				

CONCRETE BEAM SCHEDULE

Mark	b	d	"A" bars	"B" bars	Reinforcing bars		Stirrups										
					Bent	Straight	No.	Type	Size	Spacing							
B1	12"	18"	2-1"φ		2-1"φ	2-3/4 φ	10	A	3/8 φ	4"	6"	6"	7"	7"			





A B C

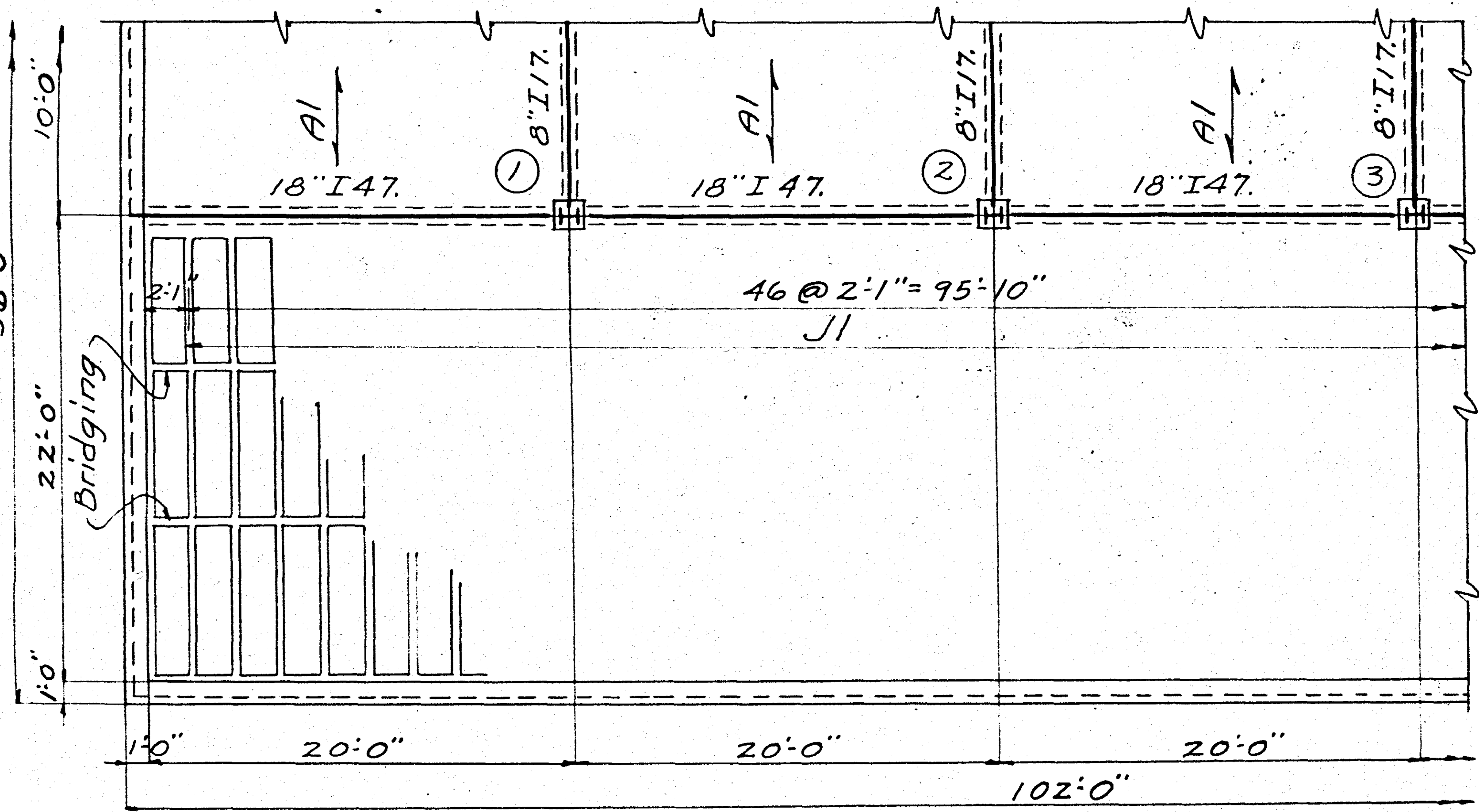
STIRRUP TYPES

"A" bars are additional bars for bond at wall end of beam
"B" bars are additional bars over intermediate support

also identified by a letter and a number and the arrow points the direction of the principal reinforcing. The column, beam and slab data is covered by schedules. The information must be sufficiently complete so the contractor can estimate accurately and prepare working drawings. The points of bending reinforcing bars for slabs and beams need not be shown on the general structural drawings since this information will be shown on the working drawings. The designer will have an opportunity to check all the working drawings submitted by the contractor.

The framing plans just discussed are supported entirely by columns but those shown on pages seventy-four and seventy-five are partially supported by walls and is known as "wall bearing construction". The internal columns may be replaced by brick walls. The concrete and steel beams supported on the exterior walls should extend to within four and one half inches of the exterior face. The wall bearing plates for the steel beams are designated by P1, P2, and are described in the wall bearing schedule. The concrete joists need only be partially drawn if the repetition is unbroken, but all joists should be shown if any irregularity occurs. The end flare of the joists should be drawn as shown on page seventy-five. The ink lines of all members should be drawn boldly and the framing plans should be drawn to scale.

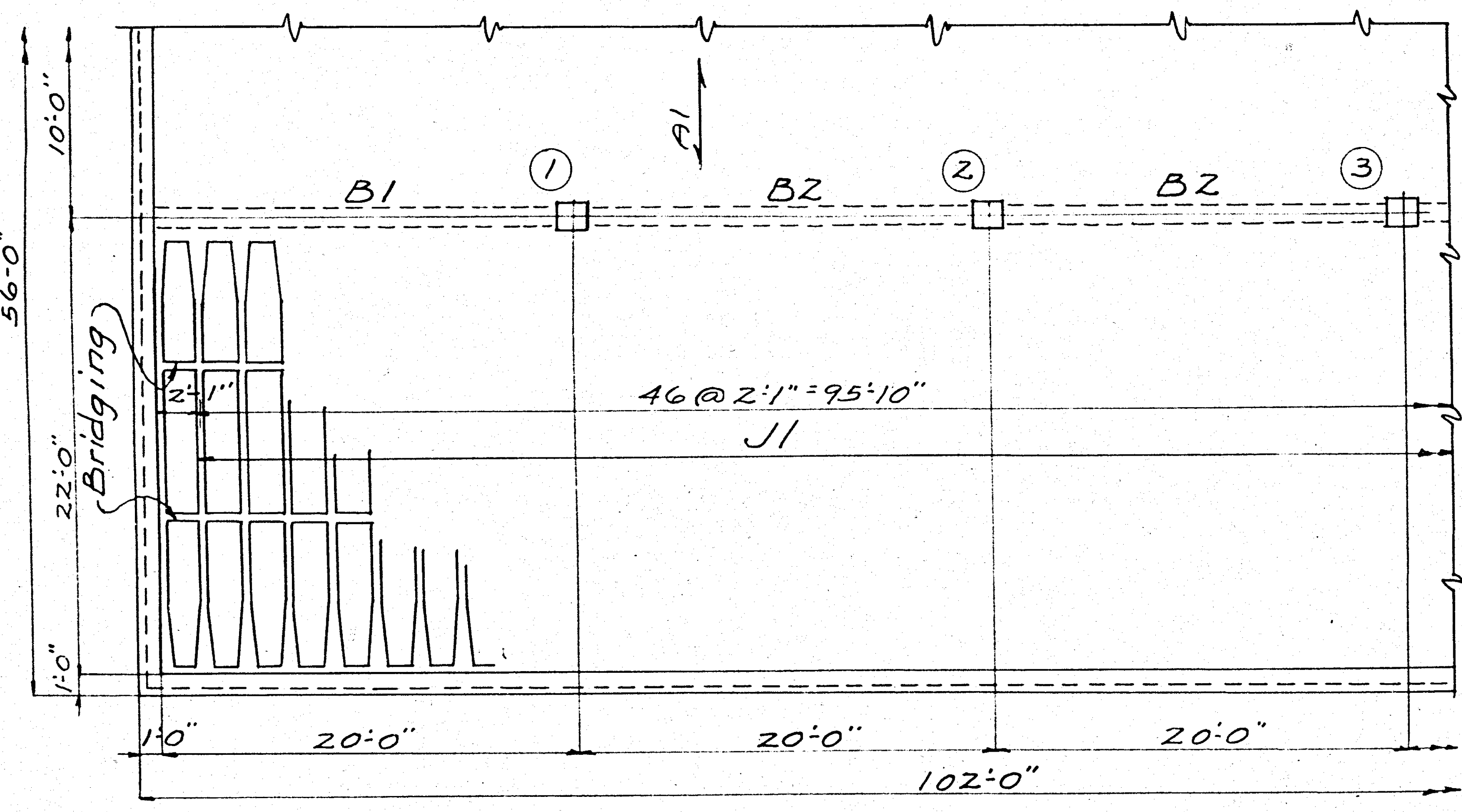
The relative merits of steel and reinforced concrete has been a subject of considerable discussion



PARTIAL FLOOR FRAMING PLAN

(Wall bearing, steel beams with concrete)

Scale $\frac{1}{8}$" = 1'-0"



PARTIAL FLOOR FRAMING PLAN
(Wall bearing, concrete beams and floor)
Scale 1/8" = 1'-0"

CONCRETE JOIST SCHEDULE

Mark	b	d	t	s	Reinforcing bars		Temperature bars		End taper
					Bent	Straight	Size	Spacing	
J1	5"	8"	2"	2'-1"	1-¾ φ	1-⅝ φ	¼ φ	10" o.c.	36"
J2	6"	12"	2"	2'-2"	1-1" □	1-1" φ	¼ φ	10" o.c.	

CONCRETE SLAB SCHEDULE

Mark	Slab thickness	Reinforcing bars		Temperature bars	
		Size	Spacing	Size	Spacing
A1	4"	½ φ	7½" o.c.	⅜ φ	12" o.c.

and difference of opinion. The sponsors of each product feature the merits of their product and in several instances have pointed out the faults of the other. Experience has proven that both products are satisfactory and to the grace of "safety factor" failures are comparatively rare. The contentions however are worthy of consideration since they affect the safety of the completed structure.

The sponsors of structural steel have been active in removing many alleged hinderances. They have succeeded in increasing the working stress to eighteen thousand pounds a square inch and reducing the dead weight of the fireproofing covering. These hinderances formally gave the concrete interests a great economical advantage. Structural steel members are a shop fabricated product and can readily be inspected and even tested before erection. The principals of mechanics can be applied to a greater degree of certainty to a structural steel design. The internal stresses in steel members before erection are very low and perhaps have been entirely eliminated since the introduction of improved rolling.

Reinforced concrete in comparison is a rather uncertain product and is made in the field under unfavorable conditions. The accepted design of beams treats the concrete above the neutral axis in compression and the reinforcing steel as resisting all tensile stress. The concrete obviously will be in tension and is actually permitted to crack or fail before the stresses in steel bars are developed. The tension in the concrete is neglected and the concrete is allowed to crack. The shear and bond stresses are computed on the assumption that the concrete below the neutral axis has not failed. The assumptions are rather contradictory and actually the internal behavior of a concrete beam is not definitely known. The beam may function as a catenary or perhaps as an arch. The shrinkage of concrete due to moisture change induces high stresses in the steel bars which are neglected in the usual computations. The present trend is to confine reinforced concrete to floor systems and secondary members and to construct the columns and primary beams of structural steel. Reinforced conc rete occupies an important place in building construction but its properties should be considered.

BIBLIOGRAPHY

1. Lee H. Miller, 1932 Steel Construction, New York City, Pages 222, 410, 411

2. Lally Column Company, 1926, Brooklyn, New York, Pages 32, 33, 37, 38, 48, 49

3. M. E. Thomas and C. E. Nichols, 1917, Reinforeed Concrete Design Tables, McGraw Hill Book Company, New York City, Pages 15, 16, 44, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104

4. G. A. Hall and C. S. Whitney, 1926, Concrete Designers Manual, McGraw Hill Book Company, New York City, Pages 98, 189

5. W. C. Wells 1924, Design of Concrete Columns with Spiral Hooping, The American Architect, New York City, Pages 475 to 877

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