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/ A CRITICAL EXAMINATION
OF
HOOGE'S VOLUME I+O. /

Thesis for Degree of C.E.

George Perry Springer.

1921. //

THESIS

137

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THS

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Introduction.

The work undertaken in this thesis is the examination of each and every Chapter; the development of such equations as may seem necessary to a clear understanding of the subject matter; the discovery of errors, if such exist; and the solution of all problems and designs, accompanied by complete drawings for each case.

The notation used throughout this thesis, in the solutions and upon the drawings, is the notation recommended in the revised, or second report of the Joint Committee of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers. The public presentation of this report being made to the American Society of Civil Engineers on January 15, 1913.

The standard notation and the formulas for design, as recommended by the Joint Committee follow:

(a) Standard Notation.

1. Rectangular beams

f_s = tensile unit stress in steel

f_c = compressive unit stress in concrete.

E_s = modulus of elasticity of steel,

E_c = modulus of elasticity of concrete.

$$n = \frac{E_s}{E_c}$$

M = moment of resistance, or bending moment in general.

A = steel area (denoted in this thesis by a_s)

b = breadth of beam

d = depth of beam to center of steel.

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

z = depth of resultant compression below top.

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

jd = $d - z$ = arm of resisting couple.

p = steel ratio (not percentage).

2. T-beams.

b = width of flange.

b' = width of stem.

t = thickness of flange.

3. Beams reinforced for Compression.

A' = area of compressive steel (denoted in this thesis by a'_s).

p' = steel ratio for compressive steel.

f'_s = compressive unit stress in steel.

C = total compressive stress in concrete.

C' = total compressive stress in steel.

d' = depth of center of compressive steel.

z = depth of resultant of C and C' .

4. Shear and Bond.

V = total shear.

v = shearing unit stress.

u = bond stress per unit area of bar.

o = circumference or perimeter of bar.

Σo = sum of the perimeters of all bars.

5. Columns.

A = total net area.

A_s = area of longitudinal steel.

A_c = area of concrete.

P = total safe load.

(b) Formulas.

1. Rectangular Beams.

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn \quad (1)$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}k \quad (2)$$

(For $f_s = 15,000$ to $16,000$, and $f_c = 600$ to 650 , k may be taken at $\frac{3}{8}$.)

Fiber stresses,

$$f_s = \frac{M}{Ajd} = \frac{M}{pjbd^2} \quad (3)$$

$$f_c = \frac{2M}{jkb d^2} = \frac{2pf_s}{k} \quad (4)$$

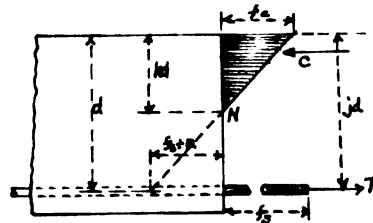


Figure 1.

Steel, ratio for balanced reinforcement,

$$\rho = \frac{1}{2} \cdot \frac{1}{\frac{f_s}{f_c} \left(\frac{f_s}{nf_c} + 1 \right)} \quad (5)$$

2. T-Beams.

Case I. When the neutral axis lies in the flange.

Use the formulas for rectangular beams.

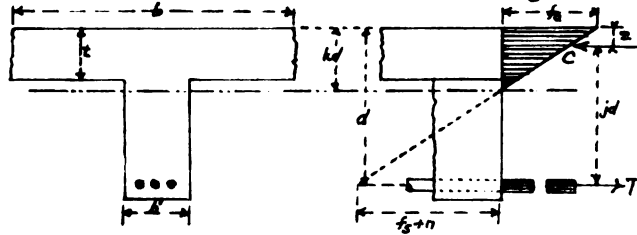


Figure 2

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem:

Position of neutral axis,

$$kd = \frac{2ndA + bt^2}{2nA + 2bt} \quad (6)$$

Position of resultant compression,

$$z = \frac{3kd - 2t}{2kd - t} \cdot \frac{t}{3} \quad (7)$$

Arm of resisting couple,

$$jd = d - z \quad (8)$$

Fiber stresses,

$$f_s = \frac{M}{Ajd} \quad (9)$$

$$f_c = \frac{3kd}{bt \left\{ kd - \frac{1}{2}t \right\} jd} = \frac{f_s}{n} \cdot \frac{k}{1 - k} \quad (10)$$

(For approximate results, the formulas for rectangular beams are used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2ndA + (b-b')t^2}{b'}} + \left\{ \frac{nA + (b-b')t}{b'} \right\}^2 - \frac{nA + (b-b')t}{b'} \quad (11)$$

Position of resultant compression,

$$z = \frac{\left\{ kdt^2 - \frac{2}{3}t^3 \right\} b + \left\{ (kd-t)^2 \left(t + \frac{2}{3}(kd-t) \right) \right\} b'}{t(2kd-t)b + (kd-t)^2 b'} \quad (12)$$

Arm of resisting couple,

$$jd = d - z \quad (13)$$

Fiber stresses,

$$f_s = \frac{M}{A_j d} \quad (14)$$

$$f_c = \frac{2Mkd}{[(2kd-t)bt + (kd-t)^2 b'] j d} \quad (15)$$

3. Beams Reinforced for Compression.

Position of neutral axis,

$$k = \sqrt{2n\left\{p+p'\frac{d'}{d}\right\} + n^2(p+p')^2} - n(p+p') \quad (16)$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}k^3 d + 2p'nd'\left[k - \frac{d'}{d}\right]}{k^2 + 2p'n\left[k - \frac{d'}{d}\right]} \quad (17)$$

Arm of resisting couple,

$$jd = d - z \quad (18)$$

Fiber stresses,

$$f_c = \frac{6M}{bd^2 \left\{ 3k - k^3 + \frac{6p'n}{k} \left(k - \frac{d'}{d} \right) \left(1 - \frac{d'}{d} \right) \right\}} \quad (19)$$

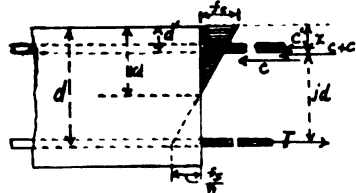


Figure 3.

$$f_s = \frac{M}{p_j b d^2} = n f_c \frac{1-k}{k} \quad (20)$$

$$f'_s = n f_c \frac{k - \frac{d'}{d}}{k} \quad (21)$$

4. Shear, Bond, and Web Reinforcement.

In the following formulas, λ_0 refers to the bars constituting the tension reinforcement at the section in question, and jd is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{bjd} \quad (22)$$

$$u = \frac{V}{jd^2} \quad (23)$$

(For approximate results, j may be taken as $\frac{7}{8}$.)

The stresses in web reinforcement may be estimated by the following formulas:

Vertical web reinforcement,

$$p = \frac{Vs}{jd} \quad (24)$$

Web reinforcement inclined at 45° (not bent-up bars),

$$P = 0.7 \frac{Vs}{jd} \quad (25)$$

in which P = stress in single reinforcing member, V = amount of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v = \frac{V}{b'jd} \quad (26)$$

$$u = \frac{V}{jd^2} \quad (27)$$

(For approximate results, j may be taken as $\frac{7}{8}$.)

c. Columns.

Total safe load,

$$P = f_c(A_c + nA_s) = f_c A(1 + [n-1]\rho) \quad (28)$$

Unit stresses,

$$f_c = \frac{P}{A(1 + [n-1]\rho)} \quad (29)$$

$$f_s = nf_c \quad (30)$$

The working stresses as recommended by the Joint Committee, which will be used throughout this thesis, follow:

WORKING STRESSES

1. General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be

added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may be approximately of the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is to be developed in cylinders 8 inches in diameter and 16 inches long, of the consistency described in Section C, Part 2 (a), made and stored under laboratory conditions, at the age of 28 days. In the absence of definite knowledge, in advance of construction, as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

TABLE OF STRENGTHS OF DIFFERENT MIXTURES
OF CONCRETE.

(In pounds per square inch)

Aggregate	1:1:2	1:1½:3	1:2:4	1:2½:5	1:3:6
Granite, trap rock.....	3300	2800	2200	1800	1400
Gravel, hard limestone and hard sandstone.....	3000	2500	2000	1600	1200
Soft limestone and hard sand- stone.....	2200	1800	1500	1200	1000
Cinders.....	800	700	600	500	400

N.B. For variations in the moduli of elasticity
see Section C, Part 3.

2. Bearing.

When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 22.5 per cent of the compressive strength may be allowed.

3. Axial Compression.

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.5 per

cent of the compressive strength may be allowed.

For other forms of columns the stresses obtained from the ratios given in Section 8, Part 9, may govern.

4. Compression in Extreme Fiber.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5 per cent of the compressive strength. Adjacent to the support of continuous beams stresses 15 per cent higher may be used.

5. Shear and Diagonal Tension.

In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress are recommended:

(a) For beams with horizontal bars only and without web reinforcement calculated by the method given in Formula (22): 2 per cent of the compressive strength.

(b) For beams thoroughly reinforced with web reinforcement: the value of the shearing stress calculated as for a (that is, using the total external vertical shear in Formula (22) for unit shearing stress), must not exceed 5 per cent of the compressive strength. The web reinforcement, exclusive of bent-up bars, in this case, shall be proportioned to resist two-thirds of the external vertical shear in the formulas (24) or (25).

(c) For beams in which part of the longitudinal reinforcement is used in the form of bent-up bars distributed over a portion of the beam in a way covering the requirements for this type of web reinforcement: the limit of maximum vertical shearing stress (the stress calculated as for a), 5 per cent of the compressive strength.

(d) Where punching shear occurs, that is, shearing stress uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for by reinforcement: a shearing stress of 5 per cent of the compressive strength may be allowed.

6. Bond.

The bond stress between concrete and plain reinforcing bars may be assumed at 4 per cent of the compressive strength, or 2

per cent in the case of drawn wire.

7. Reinforcement.

The tensile or compressive strength in steel should not exceed 16,000 pounds per square inch.

In structural steel members, the working stresses adopted by the American Railway Engineering Association are recommended.

8. Modulus of Elasticity.

The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in computations for the position of the neutral axis and for the resisting moment of beams and for the compression of concrete in columns, it be assumed as:

- (a) One-fifteenth of that of steel, when the strength of the concrete is taken as 2200 pounds per square inch or less.
- (b) One-twelfth that of steel, when the strength of the concrete is taken as greater than 2200 pounds per square inch, or less than 2800 pounds per square inch, and,
- (c) One-tenth of that of steel, when the strength of the concrete is taken as greater than 2800 pounds per square inch.

Although not rigorously accurate, these assumptions will give safe results. For the deflections of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus one-eighth of that of steel is recommended.

The report of the Joint Committee, with the exception of working stresses, notation and formulæ, which have already been stated, is attached as Appendix II of this thesis.

In the solution of problems and in the design of structures reference will be made to various tables numbered from 1 to 13. These tables are given in Eool's Volume 1 and appear in this thesis as Appendix 1.

ORDER OF PROCEDURE.

The order in which the various points will be taken up for consideration are:

- 1st. Remarks on the Chapter and discussion of errors, if any, discovered.
- 2nd. Development of equations, where deemed to be necessary.
- 3rd. Statement of each problem or design followed by the solution and necessary drawings.

CHAPTER I.

A discussion of earth pressures, stability and equivalent fluid pressure, in which Rankine's Theory is stated and methods for arriving at equivalent pressures as used in designing reinforced concrete walls are developed.

The assumptions and the general equation of Rankine's Theory are;

Let P = resultant earth pressure in pounds on a vertical surface of wall equal to length 1.

h = total height of surface in feet.

w = weight of earth per foot.

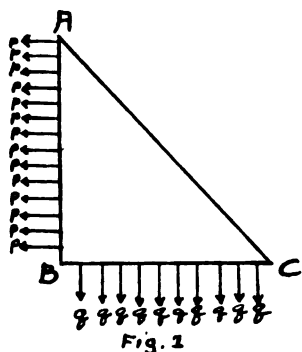
θ = angle of surcharge.

ϕ = angle of internal friction of the earth.

Then the total pressure P upon the wall is given by

$$P = \frac{1}{2} w h^2 \cos \theta \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

This theory assumes the filling to consist of an incompressible homogeneous, granular mass, without cohesion, the particles being held in position by internal friction on each other: that the mass is of indefinite extent, having a plane top: that on no plane passing thru a given point does the obliquity of stress exceed the angle of internal friction: on one plane it is just equal to it: that the resultant pressure on a vertical wall is parallel to the top surface.



Let the stresses caused by forces acting upon the plane AB, and BC, have the intensity p and q . Consider these intensities of stress broken up into parts so that

$$p = \frac{p + q}{2} + \frac{p - q}{2}, \text{ an identity}$$

$$q = \frac{p + q}{2} - \frac{p - q}{2}, \text{ an identity}$$

These parts of p and q may be considered as two stresses of the same kind; and two similar stresses of unlike sign.

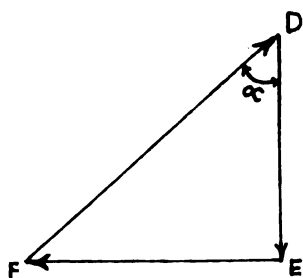
Group the similar stresses of same sign together and the

similar stresses of unlike sign together.

$$\text{draw } LE = \frac{p+q}{2} \times CB,$$

$$\text{draw } EF = \frac{p+q}{2} \times AB,$$

$$\tan \alpha = \frac{EF}{LE} = \frac{\frac{p+q}{2} AB}{\frac{p+q}{2} CB} = \frac{AB}{CB}$$



11.

Therefore $\alpha = \text{angle } AOB$ and LF is $\perp$ to AC

$$\begin{aligned} LF^2 &= LE^2 + FE^2 \\ &= \left(\frac{\sigma + \sigma}{2} \times CB\right)^2 + \left(\frac{\sigma + \sigma}{2} \times AB\right)^2 \\ &= \left(\frac{\sigma + \sigma}{2}\right)^2 (CB^2 + AB^2) \\ &= \left(\frac{\sigma + \sigma}{2}\right)^2 AC^2 \end{aligned}$$

$$LF = \frac{\sigma + \sigma}{2} \times AC$$

Therefore the intensity of stress on AC , due to the equal stresses acting on AB and BC , is equal to the intensity of the stress on these planes and is normal to AC .

$$\text{Now draw } GH = \frac{\sigma - \sigma}{2} \times CB$$

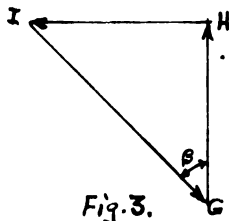


Fig. 3.

$$\text{draw } HI = \frac{\sigma - \sigma}{2} \times AB$$

$$\tan \beta = \frac{HI}{HG} = \frac{\frac{\sigma - \sigma}{2} \times AB}{\frac{\sigma - \sigma}{2} \times CB} = - \frac{AB}{CB}$$

$\therefore \beta = \alpha = \text{angle } AOB$ and GI makes an angle $= \alpha$ to a out on the opposite side

of the vertical through the point of application.

$$\begin{aligned} GI^2 &= HG^2 + HI^2 \\ &= \left(-\frac{\sigma - \sigma}{2} \times CB\right)^2 + \left(\frac{\sigma - \sigma}{2} \times AB\right)^2 \\ &= \left(-\frac{\sigma - \sigma}{2}\right)^2 (CB^2 + AB^2) \\ &= \left(-\frac{\sigma - \sigma}{2}\right)^2 AC^2 \end{aligned}$$

$$GI = - \frac{\sigma - \sigma}{2} \times AC$$

$\therefore$ the intensity of stress on AC due to the unlike similar stresses on AB and $BC = - \frac{\sigma - \sigma}{2}$

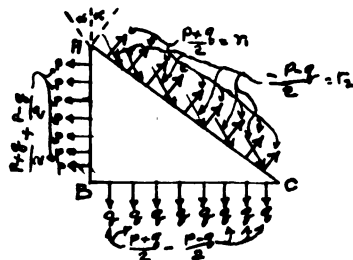


Fig. 4.

To find the resultant of $\frac{\sigma + \sigma}{2} = r_1$ and $-\frac{\sigma - \sigma}{2} = r_2$

Draw AD in fig. 4 parallel to AC in fig. 4. Lay off $OD = r_1 = \frac{\sigma + \sigma}{2}$ normal to AD . From D draw $DE = r_2 = -\frac{\sigma - \sigma}{2}$

parallel to $-\frac{\sigma - \sigma}{2}$ in fig. 4.

Draw $OC = r = \text{resultant of } r_1 \text{ and } r_2$.

With D as a center describe an arc.

$$\text{Then } DO = DS = OD = \frac{\sigma + \sigma}{2} = r_1$$

$$OD = \frac{\sigma + \sigma}{2} + \frac{\sigma - \sigma}{2} = \sigma$$

$$OD = \frac{\sigma + \sigma}{2} - \frac{\sigma - \sigma}{2} = \sigma$$

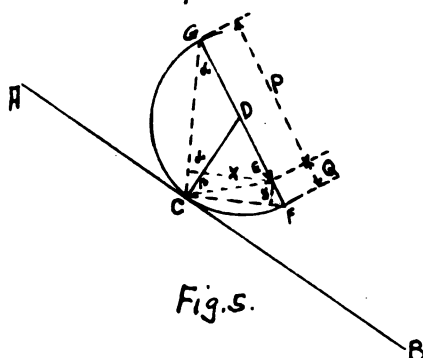


Fig. 5.

Angle DCE = obliquity of $r = \beta$

As the plane AB moves through all the angles about the point C, the point will describe a circle about C with radius $\frac{p+q}{2}$. The locus of E will be an ellipse.

Let x and y be the coordinates of E, referred to C as origin in Fig. 5.

$$\frac{x}{CE} = \sin \alpha, \quad CE = P, \quad \therefore x/P = \sin \alpha \quad (a)$$

$$y/EF = \cos \alpha, \quad EF = Q, \quad \therefore y/Q = \cos \alpha \quad (b)$$

square (a) and (b) and add

$(x/P)^2 + (y/Q)^2 = \sin^2 \alpha + \cos^2 \alpha = 1$, the equation of the path of the point E.

The maximum value of β will be when DE is perpendicular to CE. At this instant angle DEC is a right angle.

$$\text{Then } EC^2 = CD^2 - DE^2$$

$$r^2 = \left(\frac{p+q}{2}\right)^2 - \left(-\frac{p-q}{2}\right)^2$$

$$r^4 = pq$$

$$r = \sqrt{pq}$$

$$\sin \beta = -\frac{p-q}{2} + \frac{p+q}{2} = -\frac{q-q}{p+q}$$

$$\sin \beta (p+q) = -p+q$$

$$p \sin \beta + q \sin \beta = -p+q$$

$$p \sin \beta + p = -q \sin \beta + q$$

$$\frac{p}{q} = \frac{1 - \sin \beta}{1 + \sin \beta}$$

Thus where p represents the horizontal intensity of stress and q the vertical intensity of stress in an unlimited mass of homogeneous, granular nature; the ratio of the horizontal intensity of stress to the vertical will be $\frac{1 - \sin \beta}{1 + \sin \beta}$

For equilibrium the angle β can never be greater than the angle of repose, or angle of internal friction; this is our angle ϕ

$$\text{tho } \frac{p}{q} = \frac{1 - \sin \phi}{1 + \sin \phi}$$

where ϕ is the angle of internal friction of the material under consideration.

Consider a retaining wall, vertical back, earth surface horizontal. The pressure, acting similar to fluid pressure, will be applied along the vertical surface AB, with value from 0 at A to the maximum wh at B shown graphically as a triangle. The total pressure P will be the area of the triangle, or $\frac{1}{2}wh$ where

$$P = \frac{wh^2}{2}$$

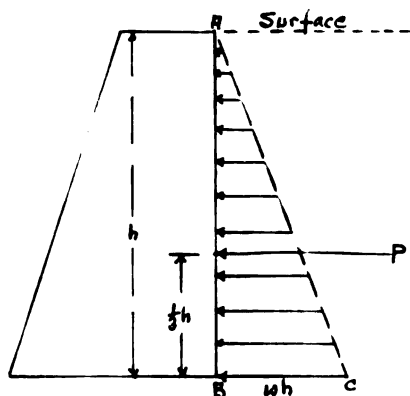


Fig. 6.

and is assumed to act through the center of gravity of the triangle at $\frac{1}{3}h$ above the base.

If ϕ is the angle of internal friction for the earth behind the wall, and the ratio of the horizontal intensity of stress to the vertical intensity of stress is given by

$$\frac{1 - \sin \phi}{1 + \sin \phi}$$

then the total active pressure on the wall will be

$$P = \frac{1}{2}wh^2 \frac{1 - \sin \phi}{1 + \sin \phi}$$

Now consider the same type of wall with a surcharge of earth at an angle θ . The line of action of P is now assumed parallel to the surface of the surcharge.

All the units of material behind the wall are held in equilibrium by three forces; the vertical; the normal to the plane of rupture, and the active pressure P.

Let v , be intensity of vertical stress,
 r , intensity of stress P

The ratio must be found.

In any line AB draw AC making the angle θ with AB.

Lay off $AE = v$

$AD = r$

Take F the middle point of ED and erect the $\perp$ FC.

Draw CD, CE and arc EHLI

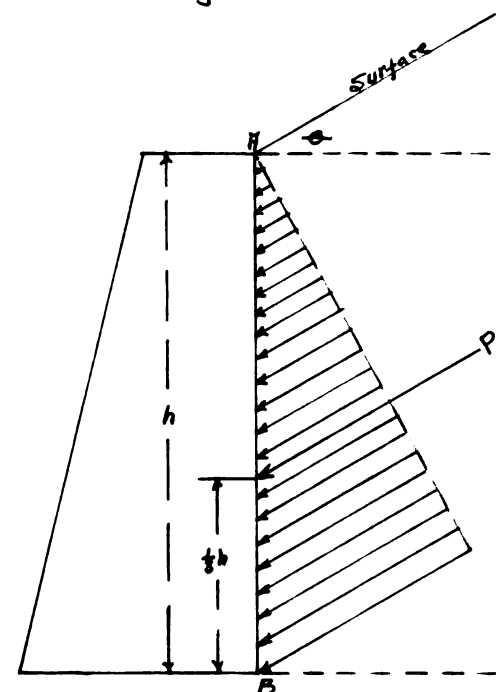


Fig. 7.

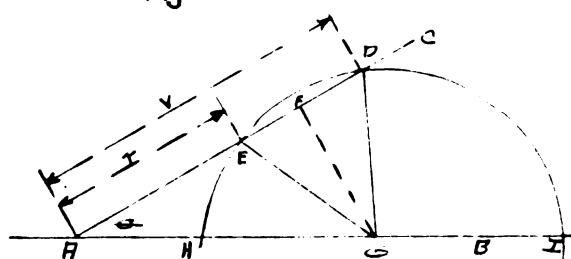


Fig. 8

$$\frac{AE}{AG} = \cos \theta$$

$$AG = \frac{AE}{\cos \theta} = \frac{\frac{1}{2}(AE + DE)}{\cos \theta} = \frac{v + r}{2 \cos \theta}$$

$$\text{but } AG = \frac{v + r}{2}, \text{ from Fig. 8.}$$

$$\therefore \frac{v + r}{2} = \frac{v + r}{2 \cos \theta} \quad (1)$$

$$CD^2 = CE^2 + ED^2$$

$$= AC^2 - AE^2 + DE^2$$

$$\begin{aligned}
 GD^2 &= \left(\frac{v+r}{2\cos\theta}\right)^2 - \left(\frac{v+r}{2}\right)^2 + \left(\frac{v-r}{2}\right)^2 \\
 &= \left(\frac{v+r}{2\cos\theta}\right)^2 - vr \\
 GD &= \sqrt{\left(\frac{v+r}{2\cos\theta}\right)^2 - vr}
 \end{aligned} \tag{2}$$

but $GD = \frac{p-q}{2}$ from Fig. 8.

$$\therefore \frac{p-q}{2} = \sqrt{\left(\frac{v+r}{2\cos\theta}\right)^2 - vr} \tag{3}$$

$$\text{square (1)} \quad \left(\frac{p+q}{2}\right)^2 = \left(\frac{v+r}{2\cos\theta}\right)^2$$

$$\text{square (3)} \quad \left(\frac{p-q}{2}\right)^2 = \left(\frac{v+r}{2\cos\theta}\right)^2 - vr$$

$$(3)^2 + (1)^2 \quad \frac{(p-q)^2}{(p+q)^2} = 1 - \frac{4vr\cos^2\theta}{(v+r)^2}$$

when the earth is just in equilibrium

$$\sin\phi = \frac{p-q}{p+q}$$

$$\therefore \sin^2\phi = 1 - \frac{4vr\cos^2\theta}{(v+r)^2}$$

$$1 - \sin^2\phi = \frac{4vr\cos^2\theta}{(v+r)^2}$$

$$\cos^2\phi = \frac{4vr\cos^2\theta}{(v+r)^2} \tag{4}$$

$$(v+r)^2 = \frac{4vr\cos^2\theta}{\cos^2\phi} \tag{5}$$

subtract $4vr$ from both sides of (5)

$$(v-r)^2 = 4vr\left(\frac{\cos^2\theta}{\cos^2\phi} - 1\right)$$

$$(v-r)^2 = 4vr\left(\frac{\cos^2\theta - \cos^2\phi}{\cos^2\phi}\right) \tag{6}$$

divide (6) by (4)

$$\left(\frac{v-r}{v+r}\right)^2 = \frac{4vr(\cos^2\theta - \cos^2\phi)}{\cos^2\phi} \times \frac{\cos^2\phi}{4vr\cos^2\theta}$$

$$\left(\frac{v-r}{v+r}\right)^2 = \frac{\cos^2\theta - \cos^2\phi}{\cos^2\theta}$$

$$\frac{v-r}{v+r} = \frac{\pm\sqrt{\cos^2\theta - \cos^2\phi}}{\pm\cos\theta}$$

reduce by composition and division,

$$2v:2r::\pm\cos\theta\pm\sqrt{\cos^2\theta - \cos^2\phi}:\pm\cos\theta\mp\sqrt{\cos^2\theta - \cos^2\phi}$$

$$\frac{v}{r} = \frac{\pm\cos\theta\pm\sqrt{\cos^2\theta - \cos^2\phi}}{\pm\cos\theta\mp\sqrt{\cos^2\theta - \cos^2\phi}} \tag{7}$$

Equation (7) represents both the active thrust of the mass of earth and the passive resistance of the retaining wall. Since r is less than v for the active forces, equilibrium will take place when the terms are given the upper signs.

Invert the proportion,

$$\frac{r}{v} = \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

$$r = v \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

$$v = wh \cos \theta. \text{ for } wh \text{ acts } \perp \text{ to } BC$$

$$\therefore r = wh \cos \theta \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

the maximum

value of the intensity of θ

$$P = \frac{1}{2} w h^2 \cos \theta \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

Chapter II.

A chapter discussing the principal types of reinforced retaining walls, cantilever, counterfort and cellular, under conditions of horizontal and surcharged filling.

In determining the maximum and minimum foundation pressures, it seems to me that the pressure formulas recommended by the Committee of the American Railway Engineering Association are simpler of application than the equations proposed by Prof. Hool.

The recommendation of the Committee is:

"When P equals the vertical component of the resultant pressure on the base; B is the full width of the base in feet, and Q is the distance from the toe to where P cuts the base; then if Q is equal to or greater than $B/3$

$$\text{Pressure at toe} = (4B - 6Q) \frac{P}{B^2}$$

$$\text{Pressure at heel} = (6Q - 2B) \frac{P}{B^2}$$

when Q is less than $B/3$

$$\text{Pressure at toe} = -\frac{2P}{3Q}$$

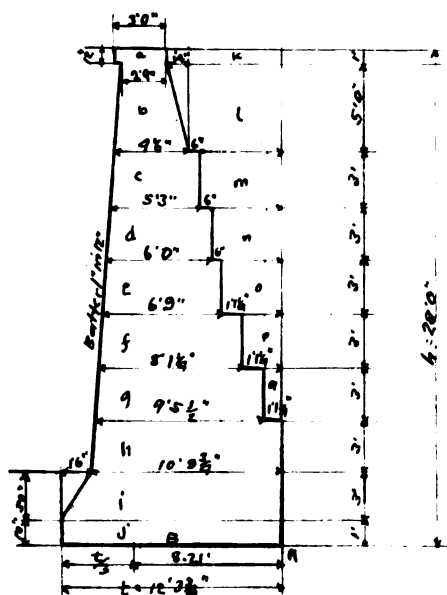
On page 33 of this volume of Hool's appears an error in reading of proof. The equation is given

$$\frac{M}{bE} \equiv \sqrt{\frac{51,000}{(12)(107.4)}} = 6 \text{ inches}$$

this should be $\frac{M}{bE} = \sqrt{\frac{51,000}{(12)(107.4)}} = 3.6" \text{ or } 6" \text{ inches}$

Design 1, Page 36.

A reinforced concrete cantilever wall is to be designed to have the same stability as the standard plain concrete New York, Central and Hudson River Railroad retaining wall, 28 feet high, 12 feet 3 $\frac{3}{4}$ inches wide. The cantilever wall is to be 16 feet high above the ground. Ends of cantilever 12 inches thick, vertical front face, uniform batter at back.



Standard
N.Y.C.
and
H.R.R.

Data for center gravity wall

Section	Area Sq. Ft.	Moment Arm	Moment Area
1	1.00	7.00	7.00
2	1.00	6.50	6.50
3	1.00	6.00	6.00
4	1.00	5.50	5.50
5	1.00	5.00	5.00
6	1.00	4.50	4.50
7	1.00	4.00	4.00
8	1.00	3.50	3.50
9	1.00	3.00	3.00
10	1.00	2.50	2.50
11	1.00	2.00	2.00
12	1.00	1.50	1.50
13	1.00	1.00	1.00
14	1.00	0.50	0.50
15	1.00	0.00	0.00
16	1.00	0.00	0.00
17	1.00	0.00	0.00
18	1.00	0.00	0.00
19	1.00	0.00	0.00
20	1.00	0.00	0.00
21	1.00	0.00	0.00
22	1.00	0.00	0.00
23	1.00	0.00	0.00
24	1.00	0.00	0.00
25	1.00	0.00	0.00
26	1.00	0.00	0.00
27	1.00	0.00	0.00
28	1.00	0.00	0.00
29	1.00	0.00	0.00
30	1.00	0.00	0.00
31	1.00	0.00	0.00
32	1.00	0.00	0.00
33	1.00	0.00	0.00
34	1.00	0.00	0.00
35	1.00	0.00	0.00
36	1.00	0.00	0.00
37	1.00	0.00	0.00
38	1.00	0.00	0.00
39	1.00	0.00	0.00
40	1.00	0.00	0.00
41	1.00	0.00	0.00
42	1.00	0.00	0.00
43	1.00	0.00	0.00
44	1.00	0.00	0.00
45	1.00	0.00	0.00
46	1.00	0.00	0.00
47	1.00	0.00	0.00
48	1.00	0.00	0.00
49	1.00	0.00	0.00
50	1.00	0.00	0.00
51	1.00	0.00	0.00
52	1.00	0.00	0.00
53	1.00	0.00	0.00
54	1.00	0.00	0.00
55	1.00	0.00	0.00
56	1.00	0.00	0.00
57	1.00	0.00	0.00
58	1.00	0.00	0.00
59	1.00	0.00	0.00
60	1.00	0.00	0.00
61	1.00	0.00	0.00
62	1.00	0.00	0.00
63	1.00	0.00	0.00
64	1.00	0.00	0.00
65	1.00	0.00	0.00
66	1.00	0.00	0.00
67	1.00	0.00	0.00
68	1.00	0.00	0.00
69	1.00	0.00	0.00
70	1.00	0.00	0.00
71	1.00	0.00	0.00
72	1.00	0.00	0.00
73	1.00	0.00	0.00
74	1.00	0.00	0.00
75	1.00	0.00	0.00
76	1.00	0.00	0.00
77	1.00	0.00	0.00
78	1.00	0.00	0.00
79	1.00	0.00	0.00
80	1.00	0.00	0.00
81	1.00	0.00	0.00
82	1.00	0.00	0.00
83	1.00	0.00	0.00
84	1.00	0.00	0.00
85	1.00	0.00	0.00
86	1.00	0.00	0.00
87	1.00	0.00	0.00
88	1.00	0.00	0.00
89	1.00	0.00	0.00
90	1.00	0.00	0.00
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96	1.00	0.00	0.00
97	1.00	0.00	0.00
98	1.00	0.00	0.00
99	1.00	0.00	0.00
100	1.00	0.00	0.00
101	1.00	0.00	0.00
102	1.00	0.00	0.00
103	1.00	0.00	0.00
104	1.00	0.00	0.00
105	1.00	0.00	0.00
106	1.00	0.00	0.00
107	1.00	0.00	0.00
108	1.00	0.00	0.00
109	1.00	0.00	0.00
110	1.00	0.00	0.00
111	1.00	0.00	0.00
112	1.00	0.00	0.00
113	1.00	0.00	0.00
114	1.00	0.00	0.00
115	1.00	0.00	0.00
116	1.00	0.00	0.00
117	1.00	0.00	0.00
118	1.00	0.00	0.00
119	1.00	0.00	0.00
120	1.00	0.00	0.00
121	1.00	0.00	0.00
122	1.00	0.00	0.00
123	1.00	0.00	0.00
124	1.00	0.00	0.00
125	1.00	0.00	0.00
126	1.00	0.00	0.00
127	1.00	0.00	0.00
128	1.00	0.00	0.00
129	1.00	0.00	0.00
130	1.00	0.00	0.00
131	1.00	0.00	0.00
132	1.00	0.00	0.00
133	1.00	0.00	0.00
134	1.00	0.00	0.00
135	1.00	0.00	0.00
136	1.00	0.00	0.00
137	1.00	0.00	0.00
138	1.00	0.00	0.00
139	1.00	0.00	0.00
140	1.00	0.00	0.00
141	1.00	0.00	0.00
142	1.00	0.00	0.00
143	1.00	0.00	0.00
144	1.00	0.00	0.00
145	1.00	0.00	0.00
146	1.00	0.00	0.00
147	1.00	0.00	0.00
148	1.00	0.00	0.00
149	1.00	0.00	0.00
150	1.00	0.00	0.00
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161	1.00	0.00	0.00
162	1.00	0.00	0.00
163	1.00	0.00	0.00
164	1.00	0.00	0.00
165	1.00	0.00	0.00
166	1.00	0.00	0.00
167	1.00	0.00	0.00
168	1.00	0.00	0.00
169	1.00	0.00	0.00
170	1.00	0.00	0.00
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172	1.00	0.00	0.00
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187	1.00	0.00	0.00
188	1.00	0.00	0.00
189	1.00	0.00	0.00
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195	1.00	0.00	0.00
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246	1.00	0.00	0.00
247	1.00	0.00	0.00
248	1.00	0.00	0.00
249	1.00	0.00	0.00
250	1.00	0.00	0.00
251	1.00	0.00	0.00
252	1.00	0.00	0.00
253	1.00	0.00	0.00
254	1.00	0.00	0.00
255	1.00	0.00	0.00
256	1.00	0.00	0.00
257	1.00	0.00	0.00
258	1.00	0.00	0.00
259	1.00	0.00	0.00
260	1.00	0.00	0.00
261	1.00	0.00	0.00
262	1.00	0.00	0.00
263	1.00	0.00	0.00
264	1.00	0.00	0.00
265	1.00	0.00	0.00
266	1.00	0.00	0.00
267	1.00	0.00	0.00
268	1.00	0.00	0.00
269	1.00	0.00	0.00
270	1.00	0.00	0.00
271	1.00	0.00	0.00
272	1.00	0.00	0.00
273	1.00	0.00	0.00
274	1.00	0.00	0.00
275	1.00	0.00	0.00
276	1.00	0.00	0.00
277	1.00	0.00	0.00
278	1.00	0.00	0.00
279	1.00	0.00	0.00
280	1.00	0.00	0.00
281	1.00	0.00	0.00
282	1.00	0.00	0.00
283	1.00	0.00	0.00
284	1.00	0.00	0.00
285	1.00	0.00	0.00
286	1.00	0.00	0.00
287	1.00	0.00	0.00
288	1.00	0.00	0.00
289	1.00	0.00	0.00
290	1.00	0.00	0.00

c.g. of earth from A, $\frac{162.661}{77.78} = 2.15$ feet

weight of earth per linear foot 7778 lbs.

Take moments about 1/3 point of base,

$$\frac{1}{2}wh^2\left(\frac{h}{3}\right) = W_2A_2K_2 + W_1A_1K_1$$

w = equivalent weight of fluid,

h = height of wall,

W_2 = weight of earth, 100 lbs. per foot cubic,

W_1 = weight of concrete, 150 lbs. per cu. ft.

A_2 = area cross section of earth, 77.78 sq. ft.

A_1 = area cross section of wall, 202.96 sq. ft.

K_2 = lever arm of earth mass about B.

K_1 = lever arm of concrete mass about B.

$$W_2A_2 = 7778.0 \text{ lbs.}$$

$$W_1A_1 = 30444.0 \text{ lbs.}$$

$$\frac{w(28)^3}{2} = (7778.0)(6.03) + (30444)(2)$$

$$3658.66w = 46903 + 60888$$

$$w = \frac{107791}{3658.66} = 29.4 \text{ lbs per cu. ft., the equivalent weight, use 30 lbs.}$$

Using the equation given on page 19 Pool;

$$\begin{aligned} w &= 150 \left(\frac{73t^2}{108h^2} + \frac{5t^2}{54h} - \frac{1}{216} \right) \\ &= 150 \left(\frac{73(12.31)^2}{108(28)^2} + \frac{5(12.31)}{(54)(28)} - \frac{1}{216} \right) \\ &= 150(0.1307 + 0.0406 - 0.0046) \\ &= 28.00 \text{ lbs equivalent fluid pressure} \end{aligned}$$

ASSUMPTIONS:

w = 30 lbs. equivalent fluid pressure,

cantilever ends 12 inches thick,

allowable soil pressure 4000 lbs. sq. ft.

coefficient friction earth on concrete 0.40

angle of friction, 22° approximate

weight of earth, 100 lbs. cu. ft.

weight of concrete, 150 lbs. per cu. ft.

concrete 2000 lb, 1:2:4

Working stresses. Bearing 32.5 % compressive strength,
Extreme fiber stress, 32.5 % compressive
Shear 6 per cent compressive, 120 lbs.
Bond, 4 per cent compressive, 80 lbs.
Steel stress, 16000 lbs. sq. in.
Concrete, 600 lbs. sq. in.

n = 15

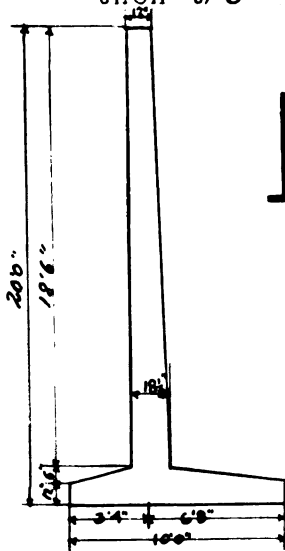
t = width of base,

h = height = 20' high.

for w = 30, t/h = 0.485, Trautwine's empirical ratio of width to height.

t = 0.485h = (0.485)(20) = 9.70 feet, use 10',

then t/8 = 1.25, middle third point under stem.



Assuming an 18" footing at stem gives a wall 18'6" high above footing.

Total pressure on vertical stem

$$P = \frac{1}{2}wh^2 = \frac{1}{2} \cdot 30 \cdot (18.5)^2 = 5133.75$$

applied at $\frac{1}{3} \cdot 18.5 = 6.16$ feet above footing.

Bending moment at top of footing,

$$M = 5133.75 \cdot 6.16 \cdot 12 = 379,486.8 \text{ in.pds.}$$

$f_c = 650$, allowable compression

$f_s = 16000$, allowable tension in steel

p = steel ratio.

$$p = \frac{\frac{1}{2}}{\frac{f_s}{f_c} \left(\frac{f_c}{nf_c} + 1 \right)} = \frac{0.5}{\frac{16000}{650} \left(\frac{650}{15 \cdot 650} + 1 \right)} = 0.00769$$

$$k = \sqrt{2pn + (pn)^2} - pn = \sqrt{2(0.00769)(15) + (0.00769 \cdot 15)^2} - (0.00769 \cdot 15) = 0.3786$$

$$j = 1 - \frac{k}{3} = 1 - 0.1262 = 0.8738$$

$$R = pf_s j = 0.00769 \cdot 16000 \cdot 0.8738 = 107.5$$

$$M = pf_s j (bd^2)$$

$$bd^2 = \frac{379,486.8}{107.5} = 3530 \text{ sq. in.}$$

let b = 12", unit length of wall.

$d = \sqrt{\frac{3530}{12}} = 17.1$ " thickness required at base.
and $1\frac{1}{2}$ " for steel covering, use 18.5"

a_s = area steel required for vertical reinforcement.

$$a_s = pbd$$

$$a_s = 0.00769 \cdot 12 \cdot 17.1 = 1.58 \text{ sq. in. required per linear foot of wall.}$$

4, 3/4 " Ø rods give 1.77 sq.in. per linear foot, space at 8" c-c.

M at 15' from top,

$$M = \frac{1}{2} \cdot 30 \cdot (15)^2 \cdot \frac{15}{3} \cdot 12 = 202500 \text{ in. pas.}$$

at 10' from top,

$$M = \frac{1}{2} \cdot 30 \cdot (10)^2 \cdot \frac{10}{3} \cdot 12 = 60000 \text{ in.pds.}$$

at 5 feet from top,

$$M = \frac{1}{2} \cdot 30 \cdot (5)^2 \cdot \frac{5}{3} \cdot 12 = 7500 \text{ in.pds.}$$

Consider a section at 15 feet, Table 3, gives

$p = 0.00384$, which is one half of p at foot.

at 10 foot point, $p = 0.00133$, which is a little

less than one fifth of p at the bottom. Therefore

for the vertical reinforcing, carry four rods up

to 15 feet from top, 2 rods to 10 feet from top and

one rods top.

Maximum shear on vertical stem is at top of footing

$$V = 5133.75 \text{ pds.}$$

$$\text{unit shear } v = \frac{V}{jbd} = \frac{5133.75}{12 \cdot 0.8738 \cdot 17.1} = 28.68 \text{ pds. (safe)}$$

$$\text{bond stress } u = \frac{V}{\sum ojd} = \frac{5133.75}{4 \cdot 2.356 \cdot 0.8738 \cdot 17.1} = 36.45 \text{ pds.}$$

$$\text{embedment } \frac{f_s l}{4u} = \frac{16000 \cdot 0.75}{4 \cdot 80} = 37.5", \text{ the committee}$$

recommends the use of more length

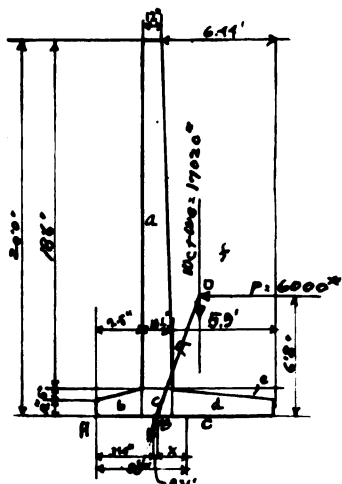
to take care of strain and temperature cracks use

0.4 per cent of steel

$$a_s = \frac{0.004 \cdot 12 \cdot 12 \cdot 4 \cdot 12}{2} = 0.586 \text{ sq. in.}$$

use $\frac{1}{2}"$ rods 4 "c-c on outer face

Footing.



section	weight	moment arm	moment weight
a	3524.25	3.04	10713.72
b	480.00	1.37	657.60
c	346.80	3.33	1153.85
d	1106.25	6.85	7577.81
e	147.80	8.03	1184.43
f	11414.50	6.92	78988.34
	17019.00		100275.75

$$\text{Lever arm of } W = \frac{100275.75}{17019.0} = 5.89' \text{ from A}$$

$$\tan \phi = \frac{P}{W} = \frac{0.5 \cdot 30 \cdot (20)^2}{17019.0} = 0.38254$$

$$\tan \phi = \frac{BC}{DE}$$

$$BC = 6.2 \cdot 0.38254 = 2.36 \text{ ft.}$$

$$5.89 - 2.36 = 3.54' \text{ P cuts base from A.}$$

• P cuts base $3.54 - 3.33 = 0.21'$ inside middle third point of base.

$$\text{Overturning factor } \frac{5.89}{2.36} = 2.5$$

$$\text{Average pressure per foot, } \frac{17019}{10} = 1701.9 \text{ pas.}$$

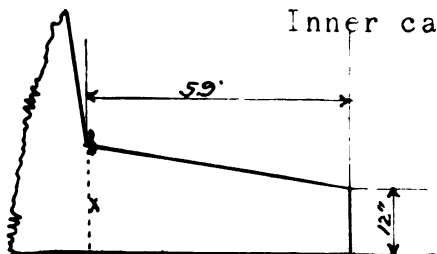
$$\text{Pressure due to bending } P_b = \frac{6Wx}{t^2}$$

$$\frac{6 \cdot 17019 \cdot 1.46}{100} = 1490.86$$

Maximum Pressure $1702 + 1480. = 3192$ lbs. at toe,

Minimum Pressure $1702 - 1480 = 210$ lbs. at heel

$$\text{Factor against sliding } \frac{0.4 \cdot 1702}{3192} = 2.13$$



Inner cantilever.

shear along x,

$$V = 11662.00 + 1106.25 - \left(\frac{1969.4 + 211}{2} \right) 5.9$$

$$= 6336.07 \text{ lbs.}$$

$$M = [(11662.0 \cdot 2.79) + (1106.25 \cdot 2.75) - (6432.18 \cdot 2.14)] 12$$

$$= 261771.6 \text{ in. lbs.}$$

Min. depth to steel

$$\sqrt{\frac{261771.6}{12 \cdot 107.5}} = 14.2"$$

depth to satisfy bond.

$$a = \frac{6336.07}{(12+4.5)2.356 \cdot 80 \cdot 0.875} = 14.4", \text{ use } 14.5"$$

3/4" rods at 4.5" c-c

$$\text{total depth } 14.5 + 1.5 = 16"$$

$$p = \frac{0.442}{(12+4.5)14.5} = 0.0114$$

From Table 3, $j = 0.854$

$$u = \frac{6336.07}{(12+4.5)2.356 \cdot 0.854 \cdot 14.5} = 77.8 \text{ lbs. sq. in. allowable}$$

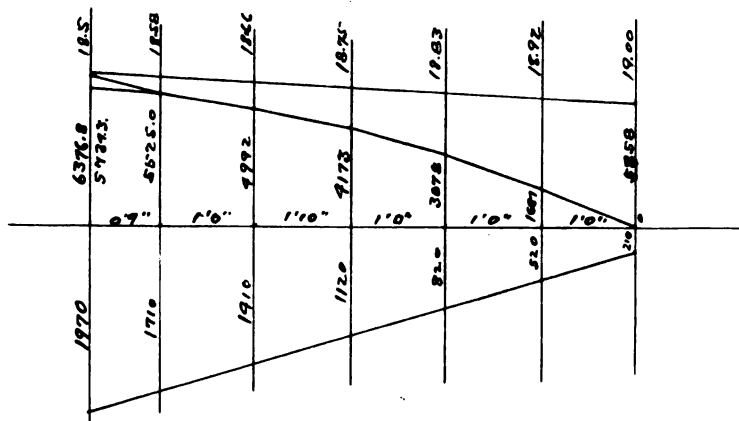
$$v = \frac{6336.07}{12 \cdot 0.854 \cdot 14.5} = 42.6 \text{ lbs. (80 lbs. allowable according to final report of Committee)}$$

$$f_s = \frac{261771.6}{0.0114 \cdot 0.854 \cdot 12 \cdot (14.5)^2} = 10688.8 \text{ lbs. sq. in.}$$

embedment necessary

$$\frac{f_s}{f_u} = \frac{10688.8 \cdot 0.75}{4 \cdot 60} = 24.8"$$

Shear and pressure diagram.



	1	2	3	4	5	6
Average height section	18.96	18,875	18.79	18.795	18.62	18.54
Weight earth	1896.00	1887.500	1879.00	1876.500	1862.00	1668.60
Average depth concrete	1.04	1.126	1.21	1.294	1.38	1.46
Weight concrete	156.30	168.900	181.50	194.100	206.70	197.10
Z loads	2052.30	4108.700	6189.20	8235.800	10302.50	12168.20
Z pressure	565.50	1931.000	1996.50	3242.000	4777.50	6433.90
Shear	1686.80	3977.700	4172.70	4991.80	5525.00	5734.30*

*Does not = maximum shear as the triangle of earth along back of wall is neglected.

OUTER Cantilever.

weight of concrete 480.00 lbs.

Lever arm about point E.

$$\frac{2.56(2.1 + 1.5)}{2} = 1.19$$

$$M = 480 \cdot 1.19 \cdot 12 = 6854.4 \text{ in. lbs.}$$

Upward pressure at E,

$$\frac{2.17019 \cdot (10 - 2.56)}{10} = 2532.4 \text{ lbs.}$$

M of upward force

$$\frac{1}{2}((2532.4 + 3192.8) \cdot 2.56 \cdot 0.89)12 = 51884 \text{ in. lbs.}$$

Total moment, 51884 - 6854.4 = 45029.6 in. lbs.

depth necessary for moment,

$$\sqrt{\frac{45029.6}{15 \cdot 107.4}} = 3.9 \text{ inches}$$

$$a_s = 0.0077 \cdot 12 \cdot 3.9 = 0.44 \text{ sq. in.}$$

In the vertical wall there is 1.77 sq. in. per foot so that bending all the rods into the outer cantilever gives more than the required area of steel. Bend each alternate rod.

$$v = \frac{(1122.8 + 1122.4)}{2} \cdot 2.56 = 6648.28$$

$$j = 0.875$$

$$u = \frac{6648.28}{(15+6)2.56 \cdot 0.875} = 20.4$$

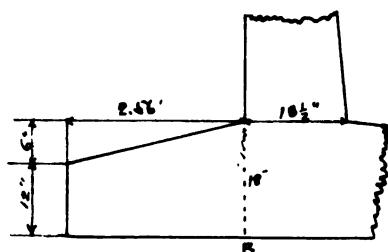
using a 20" depth

INNER Cantilever

$$p = \frac{0.44}{4.5 \cdot 20} = 0.0049$$

from Table 3, $j = 0.894$

$$u = \frac{6648.07}{2 \cdot 2.56 \cdot 0.894 \cdot 20} = 70 \text{ lbs.}$$



$$v = \frac{6336.07}{12 \cdot 0.894 \cdot 20} = 29.5 \text{ lbs. sq. in.}$$

$$f_s = \frac{261.771.6}{0.0049 \cdot 0.894 \cdot 12(20)^2} = 12450 \text{ lbs. sq. in.}$$

$$\text{embed } \frac{12450 \cdot 0.75}{4 \cdot 80} = 29.1 \text{ inches.}$$

CYCLER CANTILEVER.

$$p = \frac{1.77}{12 \cdot 20} = 0.0074$$

From Table 3, $j = 0.876$

$$u = \frac{6848.25}{4 \cdot 2.356 \cdot 0.876 \cdot 20} = 41.4 \text{ lbs. sq. in.}$$

$$v = \frac{6848.25}{12 \cdot 0.876 \cdot 20} = 32.5 \text{ lbs. sq. in.}$$

$$f_s = \frac{45039.6}{0.0074 \cdot 0.876 \cdot 12(20)^2} = 1447 \text{ lbs.}$$

$$\text{embed } \frac{1447}{1 \cdot 40} = 4.6 \text{ inches}$$

Chapter III.

A chapter dealing with construction, is very short, in fact gives very little information or discussion of the work necessary to construction. The Chapter XIX referred to is complete in regard to erecting and pouring forms.

Design No. 2.

Design a cantilever retaining wall 18 feet high above ground supporting an earth bank whose surface has an upward $1\frac{1}{2}$ to 1 slope from the top of wall. The face of the wall is to be placed on a property line and the foundation is to be 4 feet below the ground surface.

Assumptions;

Earth 100 lbs. per sq. ft.

Concrete 150 lbs. per sq. ft.

Allowable soil pressure 6000 lbs. per sq. ft.

Coefficient of friction, concrete on earth, 0.4

Equivalent fluid pressure, 25 lbs. per ft.

2000 lb. concrete

Thickness of footing 20 inches.

Working stresses;

Bearing, 25.0 per cent compressive strength.

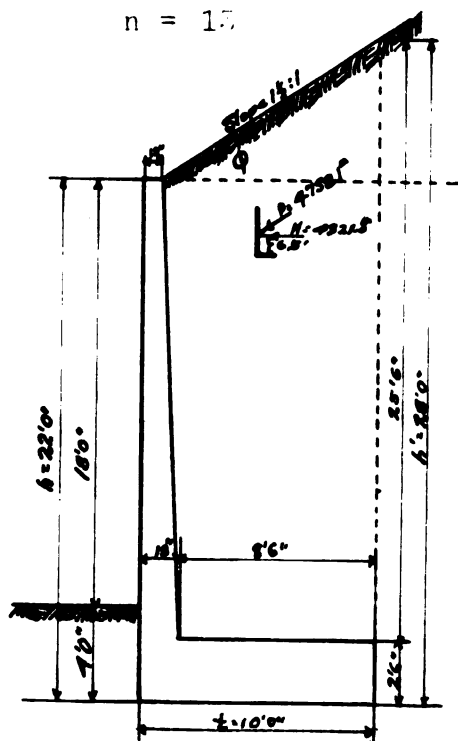
Incar, 6 per cent compressive strength.

Bond, 4 per cent compressive strength.

Allowable steel stress, 18000 lbs. sq. in.

Allowable concrete stress, 600 lbs. sq. in.

$n = 15$



$$\frac{E}{n} = 3.446 \text{ for } w = 25 \text{ lbs.}$$

$$t = 25 \cdot 0.446 = 9.8', \text{ use } 10' \text{ base top of wall } 18'$$

$$P = \frac{wh^2}{2} = \frac{25(18.5)^2}{2} = 4766.1 \text{ lbs.}$$

$$H = P \cos \phi = \frac{4766.1 \cdot 8}{9} = 4321.0 \text{ lbs.}$$

$$M = \frac{P(h - \frac{2}{3}t)}{3} = \frac{4321 \cdot 18.5}{3} = 36036.5'$$

$$K = 107.4 \text{ from table 2}$$

$$a = \sqrt{\frac{36036.5}{107.4}} = 16.3', \text{ use } 16.5''$$

Total depth 18", using $1\frac{1}{2}$ " concrete over steel, taper to 12" at top.

$$p = 0.0077 \text{ from Table 2.}$$

$$a_s = pbs = 0.0077 \cdot 18 \cdot 16.5 = 1.524 \text{ sq. in.}$$

Diagram 3 gives $\frac{11}{8}$ " rods spaced $2\frac{7}{8}$ ", area 1.55 sq. in.

At 10' from the top.

$$M = \frac{1}{2} w h^2 \frac{h}{3} = \frac{(25)(19)^3}{2 \cdot 3} = 4166.66 \text{ ft. pds.}$$

$$K = \frac{M}{b d^2} = \frac{4166.66 \cdot 12}{12(15.7)^2} = 16.9$$

At 5' from the top.

$$M = \frac{(25)(5)^3}{2 \cdot 3} = 520.8 \text{ ft. pds.}$$

$$K = \frac{520.8 \cdot 12}{12(14.85)^2} = 2.4$$

Tables cannot be used to find p .

Taking $j = \frac{7}{8}$

$$p = \frac{K}{f_s j} = \frac{16.90}{16000 \cdot 0.88} = 0.0012 \text{ at 10' from top}$$

$$p = \frac{2.4}{16000 \cdot 0.88} = 0.00017 \text{ at 5' from top}$$

$$\frac{1}{7} \text{ of steel} = \frac{0.0077}{7} = 0.0011$$

$$\frac{1}{14} \text{ of steel} = 0.0005$$

Run every seventh rod to 5' from top and every 14th rod to top, stop six rods at 10' from top

Shear on vertical stem is horizontal component of earth pressure, is maximum at top of footing = 4321 lbs.

From diagram 5, $j = 0.883$

$$v = \frac{V}{b j d} = \frac{4321}{12 \cdot 0.883 \cdot 16.5} = 24.71 \text{ lbs. per sq. inch.}$$

$$u = \frac{V}{\phi b j d} = \frac{4321}{\frac{12}{0.875} \cdot 0.883 \cdot 0.875 \cdot 16.5} = 28.13 \text{ lbs. per sq. in.}$$

each rod must be embedded at least

$$\frac{f_s l}{4u} = \frac{60,11}{16} = 37.57" \text{ extent into footing}$$

Use 0.4 per cent steel for shrinkage reinforcement.

$$a_s = \frac{0.004(12+12+17)}{2} 12 = 2.28 \text{ sq. in.}$$

Use 3 - $\frac{1}{2}$ " rods per foot, round rods give 0.5869 sq. inches. space 6" c-c on front and 12" c-c on back.

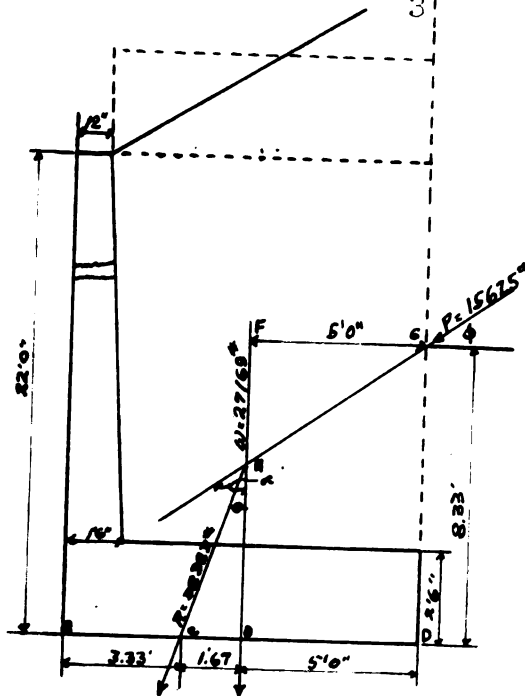
Section	Height	Mon. Arm	Mon. weight
+25.0	3356.25	2.367	34242.094
	1776.00	5.000	18750.000
	17062.50	1.375	74631.375
	2700.00	5.000	6100.000
	27168.75		135729.469

$$\text{Lever arm about steel} = \frac{135729.469}{27168.75} = 4.997'$$

Total earth pressure on plane thru heel of footing considering slope of earth above top of wall equivalent to a surcharge equal to one-half of six, 6 being the increase of h' over h .

$P = \frac{wh^2}{2} = \frac{1}{2} \cdot 25 \cdot (25)^2 = 15625$ lbs, acts parallel to slope of earth at $h/3 = \frac{25}{3} = 8.33'$ from bottom of footing.

$$\tan \phi = \frac{1}{3}, \quad \phi = 33^\circ 41' 23''$$



$$AF = \tan \phi \cdot 5 = 0.66666 \cdot 5.0 = 3.33'$$

$$\alpha = 90^\circ - \phi = 56^\circ 18' 37''$$

$$R^2 = P^2 + W^2 + 2PW \cos \alpha$$

$$R = \sqrt{P^2 + W^2 + 2PW \cos \alpha}$$

$$R = 36383.89 \text{ lbs.}$$

$$\frac{R}{\sin \alpha} = \frac{P}{\sin \phi}$$

$$\sin \phi = \frac{P \sin \alpha}{R} = \frac{15625 \sin \alpha}{36383.89}$$

$$\log P = 4.1938200$$

$$\log \sin \alpha = 9.9201513 - 10$$

$$\text{cglog } R = 5.4156510 - 10$$

$$\log \sin \phi = 19.5298223 - 20$$

$$\phi = 19^\circ 47' 53''$$

$$AB = 8.33 - 3.33 = 5.0'$$

$$CB = AB \tan \phi$$

$$= 5.0 \cdot 0.35389 = 1.8'$$

$$EC = 10.0 - 1.8 - 5.0 = 3.2', \text{ distance } R \text{ acts from toe}$$

$$\text{One-third base} = 3.33'$$

$$\text{Factor safety against overturning } \frac{2 \cdot 36115.0}{13000} = 1.11$$

$$\text{Ver. Comp. of } R = R \cos \phi = 36383.89 \cdot 0.94089 = 36115.0 \text{ lbs.}$$

$$\text{Hor. Comp. of } R = R \sin \phi = 36383.89 \cdot 0.33871 = 13000.0 \text{ lbs}$$

$$\text{Ver. Comp. of } P = 36115 - 27168.75 = 8946.25$$

$$\text{Factor safety against sliding } \frac{2 \cdot 36115.0}{13000} = 1.11$$

$$\text{Tendency to slide } 13000 \text{ lbs.}$$

$$\text{Frictional resistance } 14446.0 \text{ lbs.}$$

$$\text{Add to heel as precaution } 12" \times 1'6" \text{ cross wall}$$

$$\text{Maximum pressure on base } \frac{2 \cdot 36115}{2(5-1.8)} = 7523.56 \text{ lbs.}$$

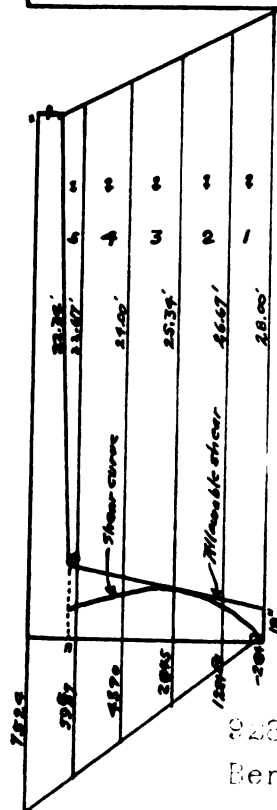
$$\text{Safe bearing of the soil is } 6000 \text{ lbs. per sq. ft.}$$

$$\text{Use piles along front to increase bearing to proper value, one pile every 4 feet, under center of stem.}$$

$$\text{Pressure on heel } \frac{36115}{10} (1 - \frac{2 \cdot 1.8}{10}) = - 363.9 \text{ lbs.}$$

Load and shears.

	1	2	3	4	5
Average height of section	27.34	26.01	24.67	23.34	22.51
Weight of earth	5468.0	5202.0	4934.0	4668.0	1125.5
Depth Concrete	2.5	2.5	2.5	2.5	2.5
Weight Concrete	750.0	750.0	750.0	750.0	187.5
Vert. Comp P	2105.5	2105.5	2105.5	2105.5	526.3
Σ Loads	8323.5	16381.0	24170.5	31694.0	33533.3
Σ Upward Pressure	1254.0	4970.0	13167.0	23826.0	26980.7
Shears	7069.5	11411.0	11003.5	7868.0	6552.6



$$x = 3(0.3) - 3.3 = 3 \cdot 1.8 - 5.0 = 0.4'$$

$$V = vbjd = 40 \cdot 12 \cdot 0.875 \cdot 12 = 5040 \text{ at end of cantilever.}$$

at maximum part of curve,

$$11411 = 40 \cdot 12 \cdot 0.875 \cdot d$$

$$d = 37.1"$$

Depth required at A 36"

$$V = 40 \cdot 12 \cdot 0.875 \cdot 36 = 15120 \text{ lbs.}$$

Bending moment at A = bending moment of earth + bending moment of concrete - moment of uplifting pressure.

$$M_e = \left[\left(\frac{12.5 + 26}{2} \right) 8.5 \cdot 100 \cdot 4.6 \right] = 92862.5$$

$$M_c = (8.5 \cdot 2.5 \cdot 150 \cdot 4.25) = 13546.88$$

$$M_p = \left(\frac{1}{2} \cdot 6945.4 \cdot 8.5 \cdot \frac{8.1}{3} \right) = 69419.75$$

$$92862.5 + 13546.88 - 69419.75 = 36989.63 \text{ ft. lbs.}$$

Bending moment due to vertical component,

$$6946.25 \cdot 8.5 = 70043.125 \text{ ft. lbs.}$$

Total moment 113032.76 ft. lbs.

Depth of steel required for moment

$$d = \sqrt{\frac{113032.76}{107.4}} = 32.4"$$

to provide for shear requires $36 + 1\frac{1}{2} = 37.5"$

$$K = \frac{M}{d^2} = \frac{113032.76}{(37.5)^2} = 87.22$$

diagram I gives $p = 0.0062$

$$A_s = pbd = 0.0062 \cdot 12 \cdot 36 = 2.678 \text{ sq. in. steel}$$

use 6 - $\frac{3}{4}$ " round rods at 3" c-c

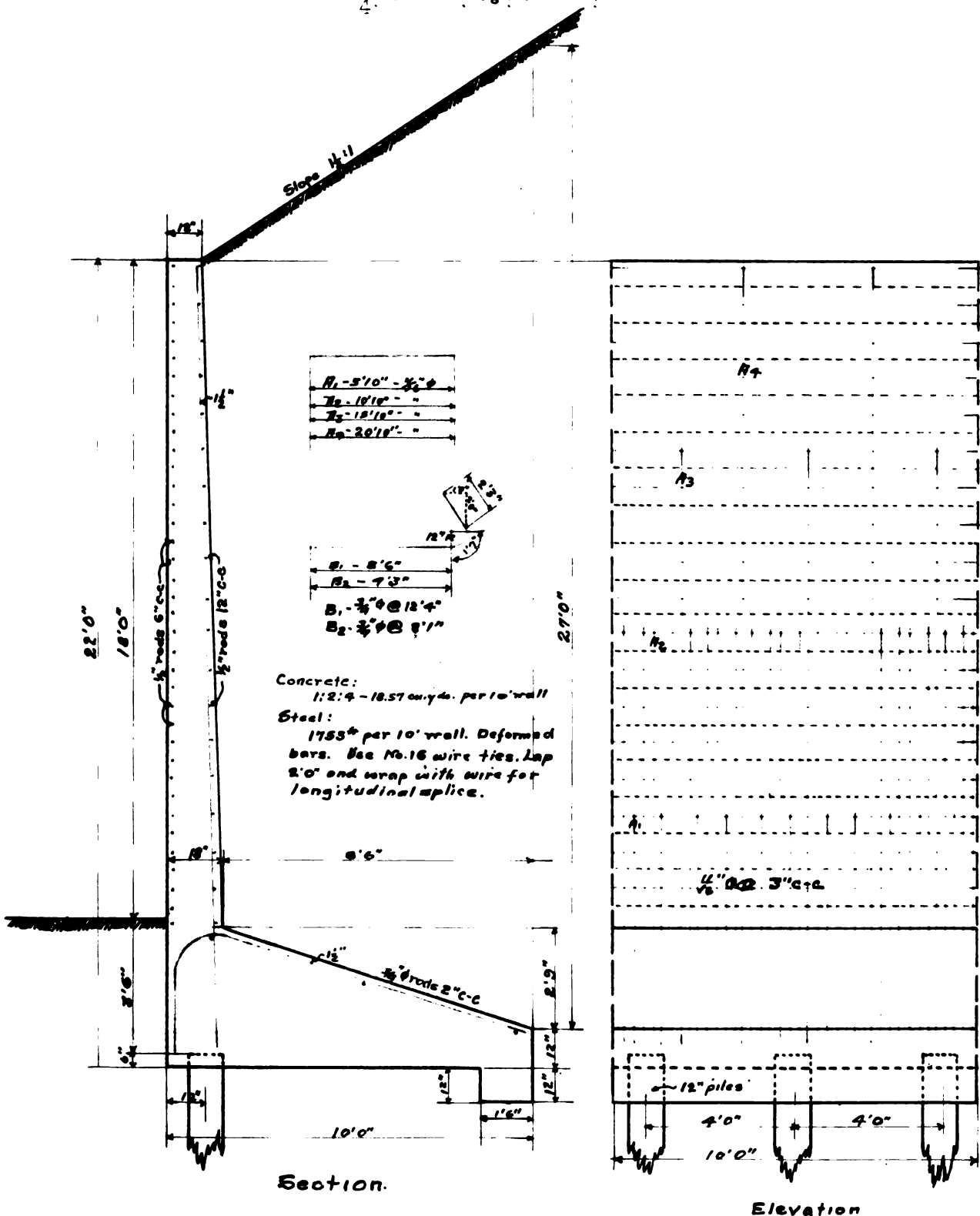
$$\text{embed } \frac{16000 \cdot 0.75}{4 \cdot 20} = 37.5"$$

At maximum point of curve shear is 12263.75 lbs.

$$\text{Bond at this point, } \frac{V}{\sum ojd} = \frac{12263.75}{\left(\frac{12}{2}\right)(2.36)\left(\frac{7}{8}\right)(30.5)} = 32.4 \text{ lbs.}$$

If every alternate rod is stopped at middle point of base

$$u = \frac{12263.75}{\left(\frac{12}{4}\right)(2.36)\left(\frac{7}{8}\right)(30.5)} = 64.8 \text{ lbs., allowable.}$$



Design No. 3.

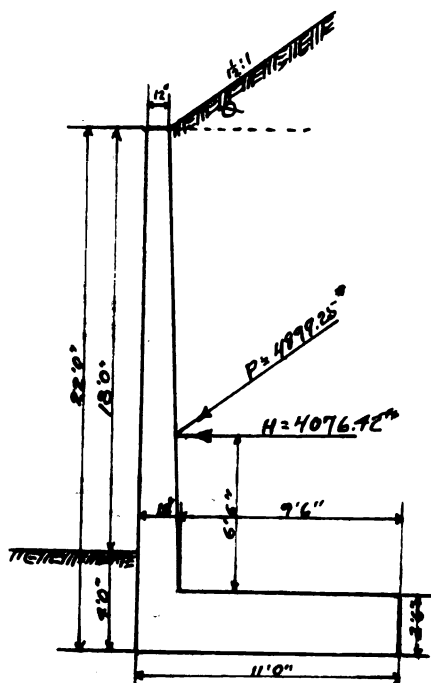
Cantilever retaining wall 18' high above ground supporting an earth bank with surface sloping upward at $1\frac{1}{2}$ to 1 from top of wall. Face of wall on property line. Foundation 4' feet below ground surface.

Assumptions;

- Earth 100 lbs. per cu. foot.
- Concrete 150 lbs. per cu. ft.
- Bearing pressure of soil 6000 lbs. per sq. ft.
- Coefficient of friction, concrete on earth, 0.4.
- ϕ , angle of internal friction of earth, 45° C'
- Rankine's formula for pressure to be used.
- Concrete 2000 lb.
- Footing 30"

Working stresses.

- Bearing, 32.5 per cent compressive strength.
- Shear, 6 per cent compressive strength.
- Bond, 4 per cent compressive strength.
- Steel, 16000 lbs. per sq. inch.
- Concrete, 650 lbs.
- $n = 15$



$$P = \frac{1}{2} \gamma h^2 \cos \theta \frac{\cos \phi - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

$$\phi = 45^\circ \text{ C'}$$

$$\theta = 11.25^\circ = 1' 52''$$

$$\cos \phi = 0.70711 \quad \cos^2 \phi = 0.5000$$

$$\cos \theta = 0.98005 \quad \cos^2 \theta = 0.9605$$

$$\sqrt{\cos^2 \theta - \cos^2 \phi} = 0.4335$$

$$P = \frac{100 \times (18)^2 \times 0.98005 (0.98005 - 0.4335)}{(0.98005 + 0.4335)}$$

$$P = 4076.42 \text{ lbs.}$$

$$H = 4076.42 \times 0.61205 = 4076.42 \text{ lbs.}$$

$$\text{lever arm } l_2 = \frac{18 \times 6}{8} = 6.5'$$

$$\text{Bending moment } 4076.42 \times 6.5 = 26496.73 \text{ ft. lbs.}$$

$$\text{from Diagram 1, Appen. 2. } K = 107.4$$

$$a = \frac{M}{Z} = \frac{26496.73}{107.4} = 15.7", \text{ use } 16"$$

$$\text{with total depth at } 18"$$

$$\text{Diagram 1 gives } p = 0.0077$$

$$a_g = p \times b = 0.0077 \times 18 \times 12 = 1.43 \text{ sq. in. steel}$$

$$4\# - \frac{11}{16}" \text{ reb. rods give } 1.43 \text{ sq. in. } 3" \text{ c-c.}$$

at 15 feet from the top

$$M = \frac{26496.7(15)^3}{(19.5)^3} = 11923.5 \text{ ft. pds.}$$

at 10 feet from the top

$$M = \frac{26496.7(10)^3}{(19.5)^3} = 3573.5 \text{ ft. pds.}$$

$$K = \frac{M}{d^2} = \frac{11923.5}{(16.5)^2} = 43.8 \text{ for 15 feet.}$$

$$K = \frac{3573.5}{(15)^2} = 15.8 \text{ for 10 feet.}$$

$$p = \frac{K}{f_{sj}} = \frac{43.8}{14000} = 0.0031 \text{ for 15 feet}$$

$$p = \frac{15.8}{14000} = 0.0011 \text{ for 10 feet.}$$

$\frac{1}{2}$ of 0.0077 = 0.0038, at 15 feet from top drop 2 bars.

$\frac{1}{4}$ of 0.0077 = 0.0011, at 10 feet from top drop 1 bar.

carry each sixth bar to top of wall.

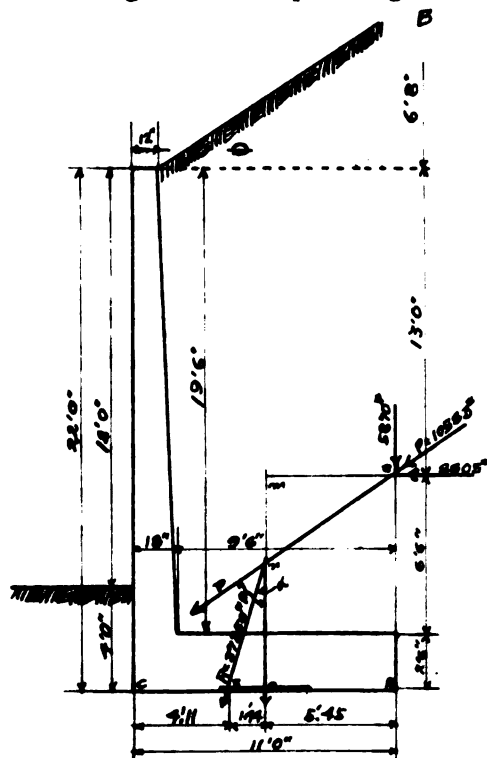
Shear at top of footing = 4076.4 lbs.

$$v = \frac{V}{b_j d} = \frac{4076.4}{12 \cdot 0.875 \cdot 16} = 24.26 \text{ lbs. per sq. in.}$$

$$u = \frac{V}{b_o d} = \frac{4076.4}{8 \cdot 16 \cdot 2.18 \cdot 0.875} = 44.0 \text{ lbs. sq. in.}$$

$$\text{embedment } \frac{f_{ei}}{4u} = \frac{10000 \cdot 11/16}{4 \cdot 80} = 34.4, \text{ use 36 inches.}$$

Use 2 - $\frac{1}{2}$ " rods per foot horizontal to provide for shrinkage. 12" spacing front and rear, alternating.



Width of base assumed at 0.5h at front face.

Height	M.F.	M.W.
1	10.8.25	10.37
2	4115.00	1.70
3	16112.00	1.87
4	8111.00	1.88
	11111.75	16112.68

Center of gravity about A =

$$\frac{11111.75 \cdot 1.70}{16112.68} = 1.16 \text{ feet}$$

Pressure on DE

$$= \frac{1}{2} (10.8.25 + 10.37) (0.332(5 - 0.4385) + 0.332(6 + 0.4385))$$

$$P = 10101.41 \text{ lbs.}$$

$$P_{comp} = 10.8.25 \cdot 0.332(5 - 0.4385) = 2804.6 \text{ lbs.}$$

$$V_{comp} = 10.37 \cdot 0.332(6 + 0.4385) = 2270.0 \text{ lbs.}$$

$$u_n = 1.48 \cdot 0.6666 = 3.62 \text{ feet}$$

$$R = \sqrt{(10101.41)^2 + (2804.6)^2} = 10400.19 \text{ lbs.}$$

$$F = 37249.39 \text{ lbs.}$$

$$\sin \alpha = \frac{10400.19}{37249.39} \sin 11.01' = 11.01'$$

$$\begin{aligned}
 \log 10,582.45 &= 2.0245863 \\
 \log \sin 56^{\circ}18'37'' &= 9.9201513 - 10 \\
 \operatorname{colog} 37,249.39 &= 5.4288808 - 10 \\
 \log \sin \alpha &= 19.3736184 - 20 \\
 \alpha &= 13^{\circ} 40' 24''
 \end{aligned}$$

os = on $\tan \alpha$

$$= 5.93 \times 9.24328 = 1.44$$

R cuts the base at $11.00 - 5.45 - 1.44 = 4.11'$ from toe

one-third base = $11 \div 3 = 3.67'$

Therefore R cuts base at $4.11 - 3.67 = 0.44'$ inside middle third

Factor of safety against sliding.

Horizontal component of R = 8804.6 lbs.

Vertical component of R = $30,323.75 + 5879.9 = 36,193.75$ lbs.

$$\frac{V_{\text{comp. R}} \times \text{Coef. F}}{H_{\text{comp. R}}} = \frac{36193.75 \times 0.4}{8804.6} = 1.64, \text{ safe.}$$

Tendency to slide is 8804.6 lbs.

Resistance to sliding 14477.5 lbs.

Factor safety against overturning.

$$Co + so = 5.55 \div 1.44 = 3.85, \text{ safe}$$

Pressure on base,

$$\begin{aligned}
 [4(AC) - 6(Cs)] \frac{V_{\text{Comp. R}}}{(AC)^2} &= \text{pressure on toe} \\
 [4 \times 11 - 6 \times 4.11] \frac{36193.75}{121} &= 5784.98 \text{ lbs, allowable.} \\
 [6(Cs) - 2(Ac)] \frac{V_{\text{comp. R}}}{(AC)^2} &= \text{pressure on heel} \\
 [6 \times 4.11 - 2 \times 11] \frac{36193.75}{121} &= 795.66 \text{ lbs.}
 \end{aligned}$$

	1	2	3	4	5	6	7	8
Ave. Hgt.	25.5	24.5	23.8	23.2	22.5	21.8	21.2	20.0
Sec. Depth	5100.0	2450.0	2383.0	2317.0	2250.0	2183.0	1117.09	3000.0
Dpt. Conc.	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Wgt. Conc.	750.0	375.0	375.0	375.0	375.0	375.0	375.0	562.5
V.C. Pres.	1235.6	617.8	617.8	617.8	617.8	617.8	617.8	926.7
Σ Lpads	7085.6	1528.4	13904.2	17214.0	20456.8	23632.6	26742.4	30735.2
Σ U.P.	2346.0	4269.0	6572.0	9427.5	12588.0	16437.0	20424.0	27436.0
Shears	4739.6	6259.4	7332.2	7786.5	7868.8	7196.6	6318.4	3299.2

See diagram drawn from this table on Page 33.

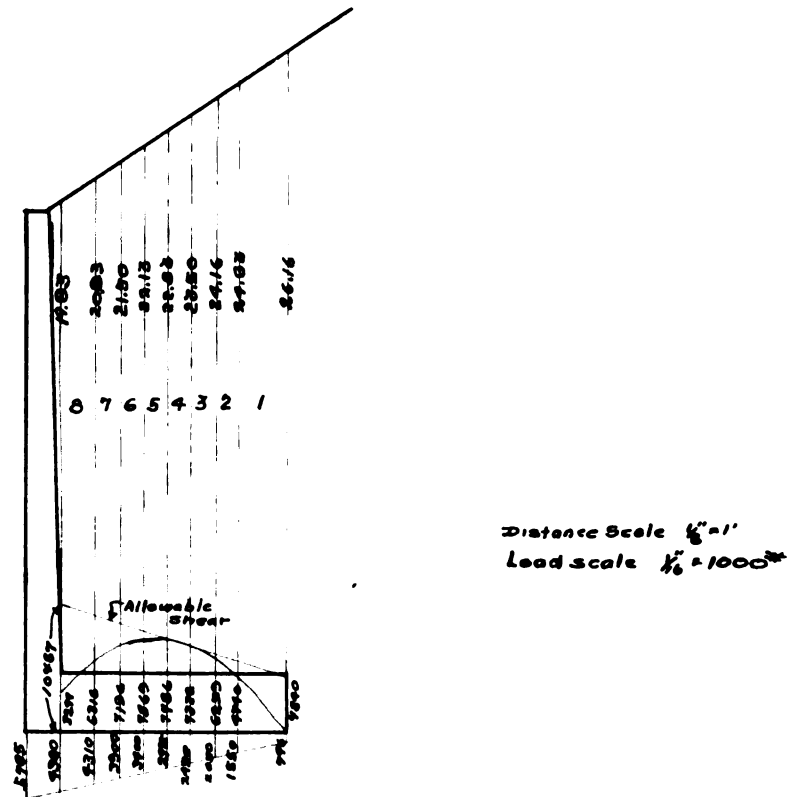
$$V = vbjd = (40)(12)(0.875)(12) = 4840$$

at the maximum point

$$7869 = (40)(12)(0.875)(d)$$

$$d = 7869 \div 403.33 = 19.5'', \text{ use } 20''$$

at inner edge of cantilever foot, ab, the depth will have to be



$$(8.5 \times \frac{8}{6}) + 12 = 23.3", \text{ use } 24" \text{ at inner edge of cantilever.}$$

$$V = 40 \times 12 \times 0.875 \times 24 = 9680 \text{ lbs.}$$

At section number 3

$$40 \times 12 \times 0.875 \times d = 7832$$

$$d = 18.1", \text{ use } 18.5"$$

At ab depth with be, due to depth at section 3,

$$(8.5 \times \frac{6.5}{4}) + 12 = 25.8", \text{ use } 26"$$

$$V = 40 \times 12 \times 0.875 \times 26 = 10486.58 \text{ lbs.}$$

Taking moments at ab;

$$\begin{aligned} M &= (21800 \times 4.97) + (3562.5 \times 4.75) - (2838 \times 9.5 \times 3.6) \\ &= 108346 + 16921.88 = 98769.6 \\ &= 26498.28 \end{aligned}$$

$$K = \frac{M}{d^2} = \frac{26498.28}{(26)^2} = 39.2$$

$$p = \frac{K}{f_{sJ}} = \frac{39.2}{16000 \times 7/8} = 0.0028$$

$$a_s = pbd = 0.0028 \times 12 \times 26 = 0.8736 \text{ sq. inches.}$$

use 5/8 inch rods at 3" c-c gives 0.9204 sq. in.

$$\text{embed } \frac{f_{ei}}{4u} = \frac{16000 \times 5/8}{4 \times 30} = 31.25", \text{ use } 33"$$

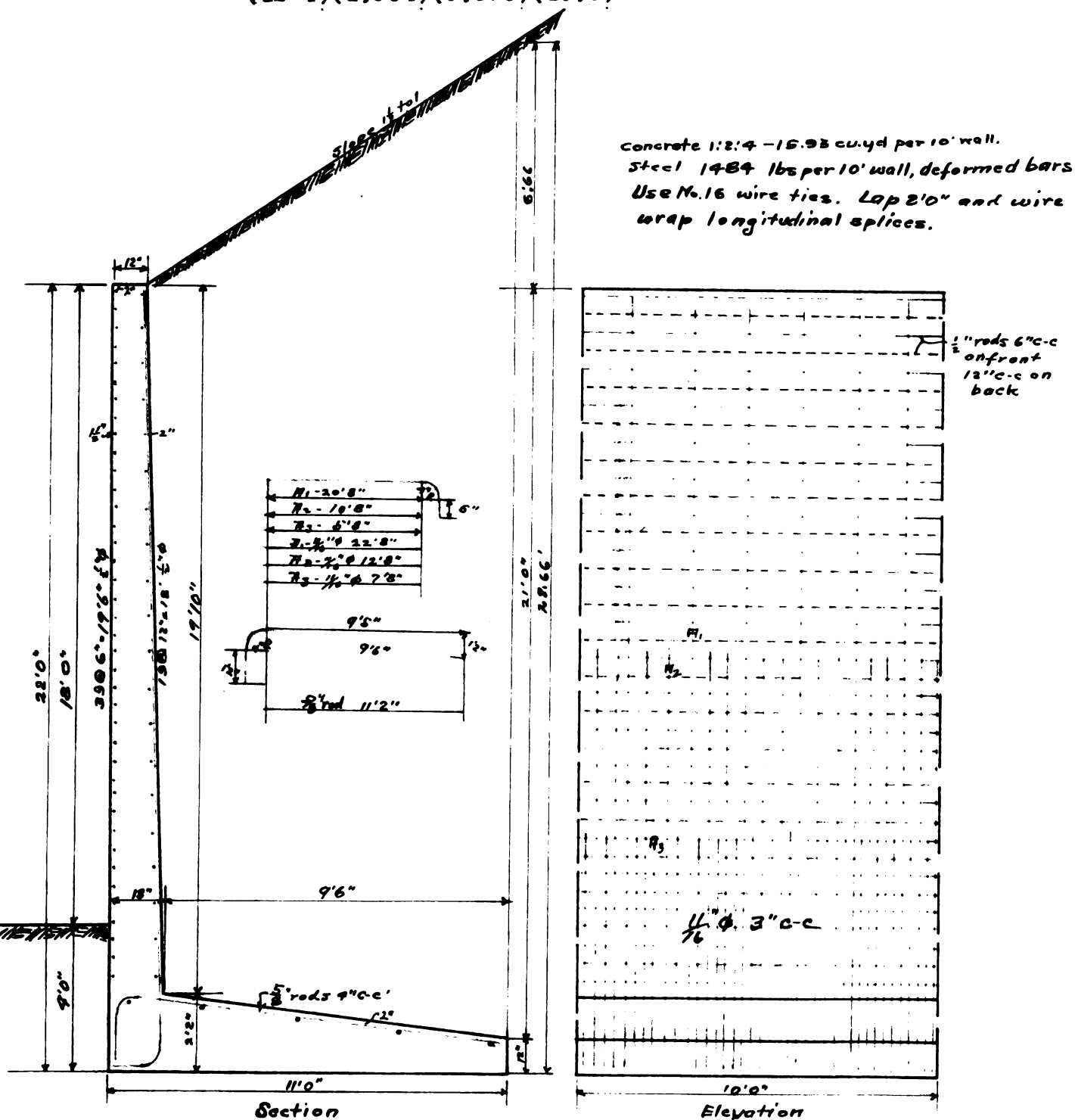
Stop one rod at 5 feet from end of cantilever.

$$v = \frac{V}{b_j d} = \frac{7332}{12 \times 9.875 \times 18.5} = 37.2 \text{ lbs., safe.}$$

$$u = \frac{V}{\Sigma o_j d} = \frac{7332}{(12+8)(1.964)(0.875)(18.5)} = 853.8 \text{ lbs. to high}$$

must carry all rods thru

$$u = \frac{7332}{(12+4)(1.964)(0.875)(18.5)} = 76.8 \text{ lbs, allowable.}$$



Design No. 4.

Design a counterforted retaining wall 30 feet high above the ground which is to support a level bank of earth. The wall is also to sustain an additional 5 foot surcharge. The vertical slab is to be placed so as to give a minimum amount of material in the wall. Counterforts to be spaced 8 feet center to center. The face of the wall is to be placed on the property line with foundation 4 feet below the ground surface.

Assumptions:

Concrete 150 lbs. per cu. ft.
 Earth 100 lbs. per cu. ft.
 Bearing pressure of soil, 6000 lbs. per sq. ft.
 Coefficient of friction, earth on concrete 0.4.
 Equivalent fluid pressure 25 lbs. cu. ft.
 Concrete 1:2:4, 2000 lbs.
 Footing assumed for preliminary 30" deep.

Working stresses:

Bearing 32.5 percent of compressive strength.
 Shear 6 percent of compressive strength.
 Bond 4 per cent of compressive strength.
 Steel, 16000 lbs. per sq. in.
 Concrete, 650 lbs. per sq. in.
 $n = 25$.

From table 1, ratios and equivalents,

$$\frac{t}{h} \text{ for } w = 25 \text{ is } 0.44$$

$$t = 0.44 \times 35$$

$$t = 15.4, \text{ use } 15' 6"$$

$t = 15' 6"$ was on trial found to be too small and it was necessary to increase the base width to 17' for overturning.

Depth of concrete in footing being taken at 28.5"

height of wall above footing $30 - 2.395 = 27.625$ feet.

Pressure on footing = wh , h being height of wall plus surcharge height.

$$P = 25 \times 32.625 = 815.6 \text{ lbs at top footing}$$

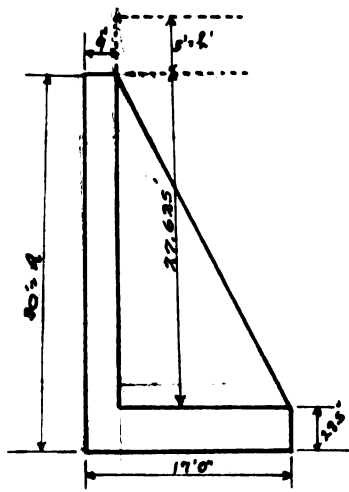
$$P' = 5 \times 25 = 125 \text{ lbs at top of wall.}$$

Take a one foot horizontal strip of curtain at top of footing

$$M = \frac{wl^2}{10}, \text{ continuous slab}$$

$$M = \frac{815.6 \times 8^2}{10} = 5419.8 \text{ ft. lbs. per foot.}$$

$$d = \sqrt{\frac{M}{K}}$$



From table 2, $K = 107.4$ $j = 0.874$
 $p = 0.0077$ $k = 0.378$

$$d = \sqrt{\frac{5419.8}{107.4}} = 7.1", \text{ use } 9"$$

$$a_s = pbd = 0.0077 \times 12 \times 7.1 = 0.656 \text{ sq. in. steel per ft. of width.}$$

use $\frac{1}{2}"$ rods spaced 3" c-c

use $\frac{1}{2}"$ straight rods to take negative moment at the counterforts.

$$\text{embedment } \frac{f_s l}{4u} = \frac{16000 \times \frac{1}{2}}{320} = 25"$$

embedment must be not less than $\frac{1}{4}$ of span

$$\frac{1}{4} \times 8 = 1.6', \text{ embedment safe.}$$

Total shear on span, counterfort assumed at 1'4"

$$V = \frac{Pl}{2} = 815.6 (4.00 - 0.67) = 2715.9 \text{ lbs.}$$

using the same size rods and spacing over supports

$$u = \frac{V}{\sum j d} = \frac{2716 \times 3}{12 \times 1.571 \times 0.874 \times 7.1} = 69.6 \text{ lbs per in.}$$

$$v = \frac{V}{jdb} = \frac{2716}{12 \times 0.874 \times 7.1} = 36.5 \text{ lbs. per sq. in.}$$

Making the curtain wall the same thickness throughout increase spacing of reinforcement as pressure decreases.

$$\text{at } \frac{1}{4} \text{ height above footing space } 3 \times \frac{32.5}{20.7} = 4.7", 4\frac{1}{2}" \text{ c-c}$$

$$\text{at } \frac{1}{4} \text{ height of wall space } 3 \times \frac{32.5}{13.8} = 7" \text{ c-c}$$

use $\frac{1}{2}"$ rods spaced 2'0" c-c for vertical reinforcing
 Footing:

$$P = \frac{1}{2}wh(h+2h') = \frac{1}{2} \times 25 \times 30(30+10) = 15000 \text{ lbs.}$$

$$\text{acts at } \frac{h^2+3hh'}{3(h+2h')} = \frac{900+450}{120} = 11.24 \text{ ft. above base}$$

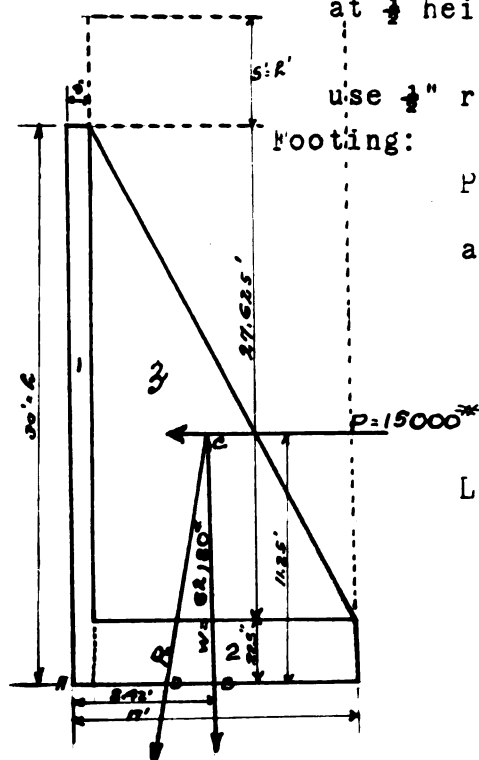
Section	Wgt.	M. Arm	M. Wgt.
1	3375.00	0.375	1265.63
2	5739.06	8.88	51406.85
3	53015.63	8.88	470778.79
	62179.69		523451.27

$$\text{Lever arms about A, } \frac{523451.27}{62179.69} = 8.42 \text{ ft.}$$

$$BD = BC \tan \angle CBD = 11.25 \times \frac{15000}{62179.69} = 2.71'$$

$$AD = 8.42 - 2.71 = 5.71'$$

$$R \text{ cuts base at } 5.71 - \frac{1}{2} = 5.04' \text{ inside mid. third}$$



Factor safety against sliding $\frac{62179.69 \times 0.4}{15000} = 1.66$

Factor safety against overturning $\frac{8.42}{2.71} = 3.1$

Average unit pressure on base $\frac{62179.69}{17} = 3657.6$ lbs. ft.

Pressure at toe $(4 \times 17 - 6 \times 5.71) \frac{62179.69}{17^2} = 7259.16$ lbs. sq. ft.

This is too high, use piling at toe, single row, 4' c-c

Pressure at heel $(6 \times 5.71 - 2 \times 17) \frac{62179.69}{17^2} = 59.04$ lbs. sq. ft.

Floor slab:

Slab assumed at depth of footing 28.5"

Considering a one foot section at the end of floor slab, where difference of upward pressure and load are at maximum.

P at end = 59.94 lbs. upward

P at 12" from end = 483.42 lbs.

Upward pressure $\frac{483.42 + 59.94}{2} = 271.68$ lbs.

Wgt. one foot of footing $1 \times 2.375 \times 150 = 356.25$ lbs.

Wgt. of one foot earth $1 \times 32.625 \times 100 = 3262.5$ lbs.

Uniform load on end foot of slab,

$3262.5 + 356.25 - 271.68 = 3347.07$ lbs.

Slab is partially continuous

$M = \frac{3347.07 \times 8^2}{10} = 21420.8$ ft. lbs.

From Table 2, $K = 107.4$

$d = \sqrt{\frac{21420.8}{107.4}} = 14.1"$, use 16"

Shear at edge of counterfort,

$3347.07(4.00 - 0.67) = 11145.51$ lbs.

$j = 0.875$

d for shear $\frac{11145.51}{12 \times 0.875 \times 40} = 26.5"$, total depth 28.5"

$K = \frac{M}{bd^2} = \frac{21420.8 \times 12}{12(26.5)^2} = 30.5$

From Table 3, $j = 0.9271$

$p = \frac{K}{f_s j} = \frac{30.36}{16000 \times 0.9271} = 0.002$

$a_s = pbd = 0.002 \times 12 \times 26.5 = 0.636$ sq. in.

$\frac{3}{4}"$ rods spaced 8" c-c sufficient at bottom

Use same size rods over supports.

Spacing over supports $\frac{b u d i}{V} = \frac{80 \times 12 \times 2.36 \times 0.927 \times 26.5}{11145.51} = 5"$
5" c-c over footing at outer foot.

Bond at edge of counterfort $\frac{V}{\sum o j d} = \frac{11145.51}{12 \times 2.36 \times 0.9271 \times 26.5}$

Bond 80.09 lbs per sq. in.

$v = \frac{11145.51}{12 \times 0.9271 \times 26.5} = 37.7$ lbs. sq. in.

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For each strip toward the stem the unit load decreases

$$\text{by } \frac{7259.16}{17} = 427.0 \text{ lbs.}$$

The shear changes sign at $\frac{3347.0}{427.0} = 7.8'$ from inner edge of footing

Place long rods at top here and short ones at bottom.

With thickness of slab uniform, the spacing of the rods increase as load decreases toward the stem.

At two feet from end:

Spacing, $\frac{5 \times 3347.0}{3347.0 - 427.0} = 5.73"$, use 5.5" spacing for top rods

$$\frac{8 \times 3347.0}{2920.0} = 9.17" \text{ use } 9" \text{ for bottom rods.}$$

At four feet from end,

$$\frac{5 \times 3347.0}{3347.0 - 1281.0} = 8.1", \text{ use } 8" \text{ for top rods.}$$

$$\frac{8 \times 3347.0}{2966.0} = 12.96", \text{ use } 12.5" \text{ for bottom rods}$$

At six feet from end,

$$\frac{5 \times 3347.0}{3347.0 - 2135.0} = 13.8", \text{ use } 13.5" \text{ for top rods}$$

$$\frac{8 \times 3347.0}{1212.0} = 22.0", \text{ use } 20" \text{ for bottom rods}$$

Use this spacing for the rest of the slab.

$$p = \frac{a_s}{S_d} = \frac{0.44}{5 \times 26.5} = 0.00332$$

From Table 3, $j = 0.91$

$$f_s = \frac{16000 \times 12}{0.00332 \times 0.91 \times 12 \times 26.5} = 7544 \text{ lbs.}$$

Depth of embedment each side of counterfort,

$$\frac{f_i}{4u} = \frac{7544 \times 0.75}{4 \times 80} = 17.9"$$

One fifth of span = $\frac{8 \times 12}{5} = 19.2"$, use 20"

$$V = \frac{11145.51}{12 \times 0.91 \times 26.5} = 33.5 \text{ lbs per square inch,}$$

shear at edge of counterfort.

COUNTERFORT.

$$\text{Total force } P = \frac{wh^2 l}{2} = \frac{25 \times 32.625^2 \times 8}{2} = 106439 \text{ lbs.}$$

$$M = 106439 \times 10.41 = 1108029.99 \text{ lbs.ft.}$$

Thickness of counterfort 1' 4",

Allowance 2" for steel,

y = lever arm of maximum stress about A,

$$y + 2: 16.625 \neq 27.625 : \sqrt{16.625^2 + 27.625^2}$$

$$y + 2 = 14.24'$$

$$y = 14.24 - 0.17 = 14.07'$$

$$\text{Spacing } \frac{12 \times 2}{1.3} = 18.4",$$

Use 18" up to 19' from top, then use 22" to top
VERTICAL RODS.

At end of footing the downward tension caused by shear on both sides, due to floor slab is,

$$3347 \times 6.67 = 22324.5 \text{ lbs. on 12" length,}$$

The $1\frac{1}{4}$ " diagonal rods take the tension in first strip.

The tension decreases by $427 \times 6.67 = 2848$ for each foot from heel to toe.

$$\text{Tension in 2nd foot is } 22324.5 - 2848 = 19476.5 \text{ lbs.}$$

$$\text{Use } \frac{3}{4}" \text{ rods at } \frac{19476.5}{0.196 \times 16000} = 6.2, \text{ use 6 per ft.}$$

Place 3 each side at 4" c-c.

$$\text{Tension in 3rd foot, } 19476.5 - 2848 = 16628.5 \text{ lbs.}$$

$$\text{Use } \frac{3}{4}" \text{ rods, } \frac{16628.5}{0.196 \times 16000} = 5.3, \text{ use 6 per ft.}$$

Use 3 each side at 4" c-c.

$$\text{Tension on 4th foot, } 16628.5 - 2848 = 13780.5 \text{ lbs.}$$

$$\text{Use } \frac{3}{4}" \text{ rods, } \frac{13780.5}{0.196 \times 16000} = 4.4 \text{ needed}$$

Place in pairs at 5" c-c.

$$\text{Tension in 5th foot, } 13780.5 - 2848 = 10932.5 \text{ lbs.}$$

$$\text{Use } \frac{3}{4}" \text{ rods, } \frac{10932.5}{0.196 \times 16000} = 3.4, \text{ use 4}$$

Place in pairs at 6" c-c.

$$\text{Tension in 6th foot, } 10932.5 - 2848 = 8084.5 \text{ lbs.}$$

$$\text{Use } \frac{3}{4}" \text{ rods, } \frac{8084.5}{0.196 \times 16000} = 2.5, \text{ space 9" c-c.}$$

$$\text{Tension in 7th foot, } 8084.5 - 2848 = 5236.5 \text{ lbs.}$$

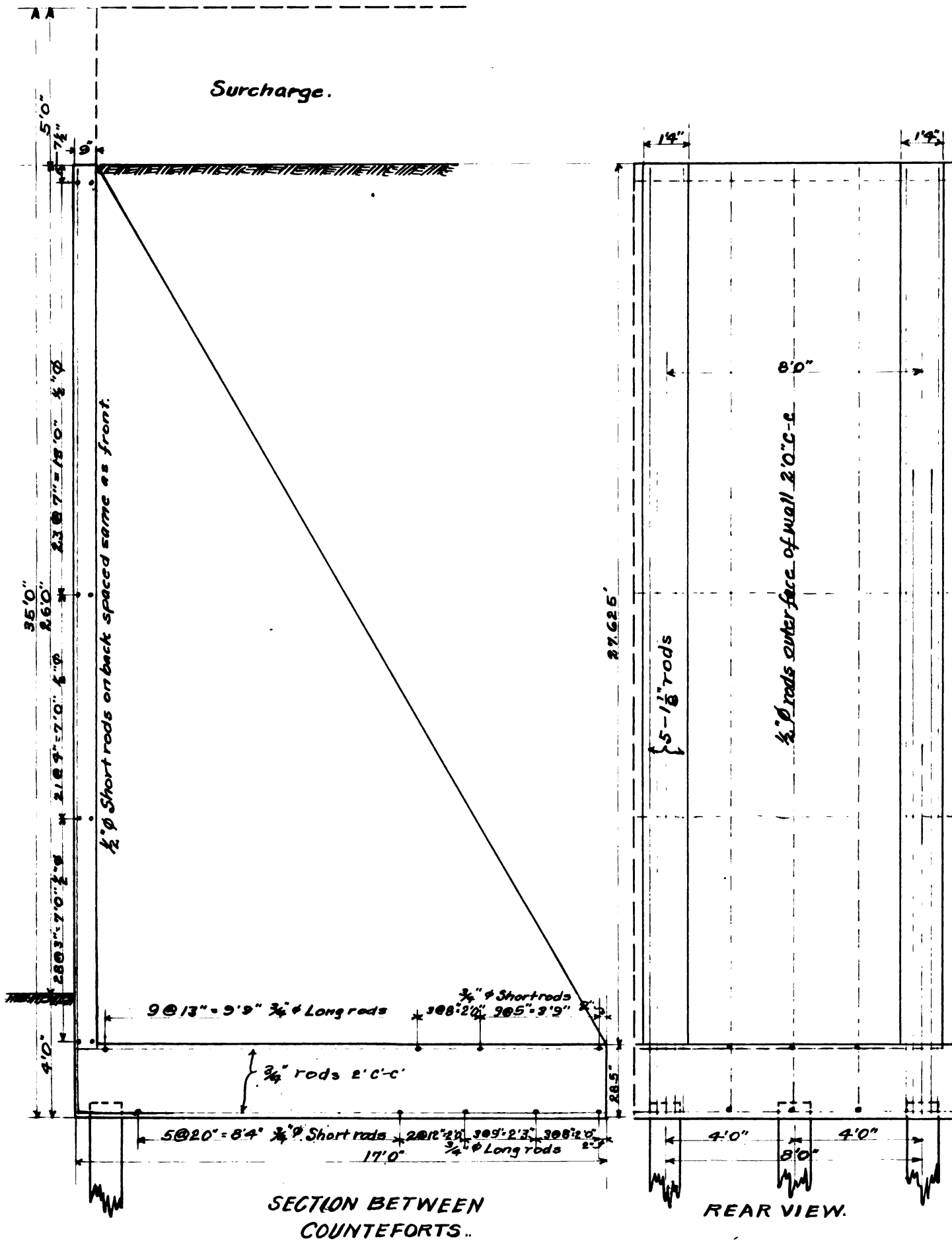
$$\text{Use } \frac{3}{4}" \text{ rods, } \frac{5236.5}{0.196 \times 16000} = 1.6, \text{ space at 15" c-c.}$$

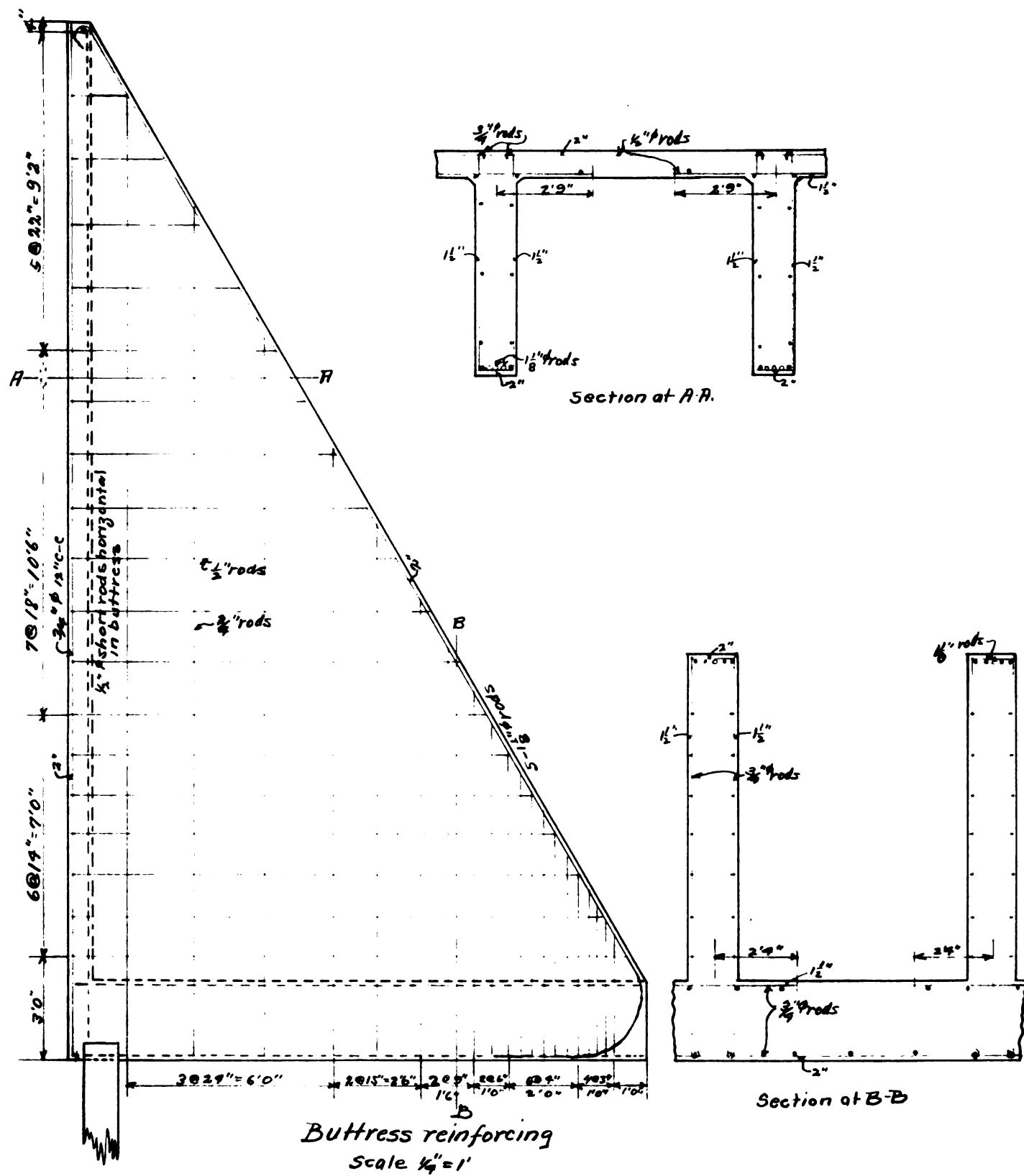
$$\text{Tension in 8th foot, } 5236.5 - 2848 = 2388.5 \text{ lbs.}$$

$$\text{Use } \frac{3}{4}" \text{ rods, } \frac{2388.5}{0.196 \times 16000} = 0.7, \text{ space 24" c-c.}$$

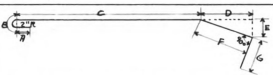
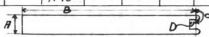
Carry this space thru to the vertical stem.

Extend the diagonal rods 50 diameters into footing,
hook vertical rods around horizontal footing rods.





STEEL SCHEDULE

No.	LOCATION	POSITION	SIZE	L'GT.	FABRICATION.																																																																																																																																																								
17	Footing	Lengthwise	$\frac{3}{4}$ "	8'0"	Straight.																																																																																																																																																								
19	"	"	"	4'8"	"																																																																																																																																																								
5	"	Crosswise	"	16'8"	"																																																																																																																																																								
5	Wall	Vertical	$\frac{1}{2}$ "	298'	"																																																																																																																																																								
74	"	Horizontal	"	8'0"	"																																																																																																																																																								
73	"	"	"	5'6"	"																																																																																																																																																								
4	Buttress	"	$\frac{3}{4}$ "	16'8"	"																																																																																																																																																								
4	"	Vertical	"	298'	"																																																																																																																																																								
																																																																																																																																																													
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On page 79 of Volume 2 is stated the equation for the most economical depth of cross beam. This equation follows

$$d = \sqrt{\frac{rM}{f_s b'}} + \frac{t}{2}$$

Consider the derivation of this equation.

Let c = cost of concrete per unit of volume,

r = ratio of cost of steel to cost of concrete,

d = depth of beam below slab,

C = cost of beam per unit length.

$$\text{now } a_s = \frac{M}{f_s (d - \frac{t}{2})}$$

M = bending moment,

f_s = working stress of steel,

a_s = area of steel,

d = depth of beam and slab,

t = depth of slab,

b' = breadth of stem

$$\frac{rM}{f_s (d' + \frac{t}{2})} = \frac{rM}{f_s (d - \frac{t}{2})} \quad \text{ratio of cost of steel per unit length of beam to cost of concrete.}$$

$b'd'$ = volume of concrete per unit length,

$$c[b'd' + \frac{rM}{f_s (d' + \frac{t}{2})}] = C, \text{ the cost of beam per unit length,}$$

Considering b' as fixed the minimum cost will be found by placing the first derivative of the equation for $C = 0$ and solve for d

$$c[b'd' + \frac{rM}{f_s (d' + \frac{t}{2})}] = C$$

$$cb'd' + \frac{rMc}{f_s (2d' + t)} = C$$

$$b'd'c + \frac{2rcM}{2d'f_s + tf_s} = C$$

$$\frac{dC}{dd'} = b'c + \frac{(2d'f_s + tf_s)0 - 4rcMf_s}{(2d'f_s + tf_s)^2} = 0$$

$$b'cf_s(2d' + t)^2 - 4rcMf_s = 0$$

$$(2d' + t)^2 = \frac{4rcMf_s}{b'cf_s} = \frac{4rM}{b'f_s}$$

$$(2d' + t) = 2\sqrt{\frac{rM}{b'f_s}}$$

$$2d' = 2\sqrt{\frac{rM}{b'f_s}} - t$$

$$d' = \sqrt{\frac{rM}{b'f_s}} - \frac{t}{2}$$

$$\text{but } d = d' + t$$

$$d = \sqrt{\frac{rM}{b'f_s}} - \frac{t}{2} + t$$

$$d = \sqrt{\frac{RM}{b \cdot f_s}} + \frac{t}{2}, \text{ the most economical depth of T section.}$$

Problem 5. Design an interior floor bay with a 1-inch granolithic finish. Live load 200 pounds per square foot. Columns spaced 18' x 20' centers. Girder span 18', cross beam span 6' center to center.

Assumptions, plain round rods,

ratio of unit cost of steel in place to concrete in place = 60,

1" finish not included as effective part of slab thickness, but weight is included in dead load.

Working stresses, bearing, 32.5 percent of compressive strength,

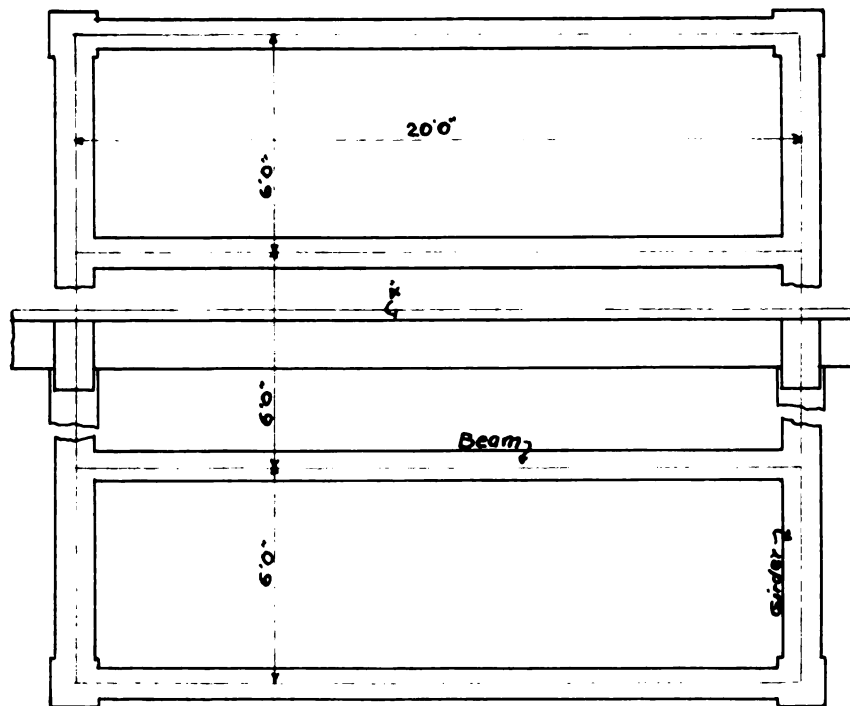
shear, 6 percent compressive strength,

bond, 4 percent compressive strength,

steel, 16000 pound,

concrete, 1:2:4, 2000 lb., 650 lbs compression.

$n = 15$



SLAB:

Table 6 shows that a 4" slab ($d = 3\frac{1}{2}$ ") span 6' will carry $315 \times 1.2 = 378$ lbs. per sq. ft. live and dead.

slab weights 50 lbs. per sq. ft.

total load from slab, $200 + 50 = 250$ lbs.

$a_s = 0.3$ sq. in.

Table 4, find that $\frac{3}{8}$ " rods spaced 4" give 0.33 sq. in.

$$p = \frac{0.11}{4 \times 3.25} = 0.0084$$

From table 7, $w = 324$, this is more than load to be carried and is safe.

place 2 - 3/8" rods transversely in each 6' panel to prevent shrinkage and temperature cracks.

CROSS BEAMS.

Span 20'

Load per ft of length $3 \times 250 = 1500$ lbs.

Assumed load of stem 230 lbs per linear foot.

Total load per foot 1730 lbs.

$$\text{Maximum shear } V = \frac{1730 \times 20}{2} = 17300 \text{ lbs.}$$

$$\text{Maximum moment, } \frac{1730 \times (20)^2 \times 12}{12} = 692000 \text{ lbs.in.}$$

Required cross section determined by shear,

$$\frac{17300}{105} = 164.76 \text{ sq. inches}$$

$$d = \sqrt{\frac{rV}{f_s b'}} + \frac{t}{2}, \text{ for economical depth}$$

$$r = 60$$

$$\text{for } b' = 8, d = \sqrt{\frac{60 \times 692000}{18000 \times 8}} + \frac{4}{2} = 20"$$

$$b' = 8, d = 19.9"$$

$$b' = 10, d = 19.1"$$

$$b' = 11, d = 17.3"$$

using 6 rods in two rows, spaced 2 1/2" c-c a 10" x 19" beam has sufficient area.

weight of stem, to center of reinforcement,

$$\frac{10 \times 18 \times 150}{144} = 187.5 \text{ lbs per foot.}$$

width of flange 8 times thickness of slab + b' ,

$$(8 \times 4) + 10 = 42"$$

$$\frac{M}{bd^2} = \frac{692000}{42 \times 22 \times 22} = 34.0$$

with 34.0 and $t/d = 4/22 = 0.181$, table 9 gives p a value between 0.002 and 0.004

f_c has a value between 301 and 319

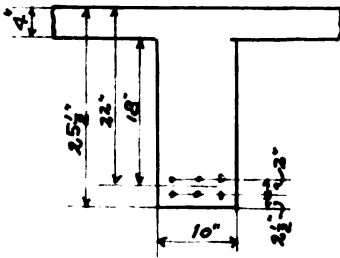
$$p = 0.002 + \frac{4.2}{28.2}(0.002) = 0.0023$$

$$j = 0.924 \quad f_c = 319$$

$$a_s = 42 \times 22 \times 0.0023 = 2.13 \text{ sq. in.}$$

use 2 - 3/4" rods at 0.4418 = 1.3284 sq. in.

use 3 - 5/8" rods at 0.9063 = 0.9204 sq. in.
2.2488 sq. in.



3 - 3/4" in lower row go straight thru,
3 - 5/8" bend up and lap over support.

$$u = \frac{17300}{6 \times 1.964 \times 0.85 \times 22} = 67.1 \text{ lbs. s.q. in}$$

Rods at top of beam over support have same depth as rods at bottom in center of span.

$$\frac{d'}{d} = \frac{3.25}{22} = 0.1477$$

$$p = \frac{2.2458}{10 \times 22} = 0.0102 = p'$$

From table 11 find,

$$L = 0.2353$$

$$K = 0.0090$$

$$f_c = \frac{692000}{10 \times 22^2 \times 0.2353} = 607 \text{ lbs. s.q. in.}$$

$$f_s = \frac{692000}{10 \times 22^2 \times 0.0090} = 15886 \text{ lbs. s.q. in.}$$

no haunch or extra steel required

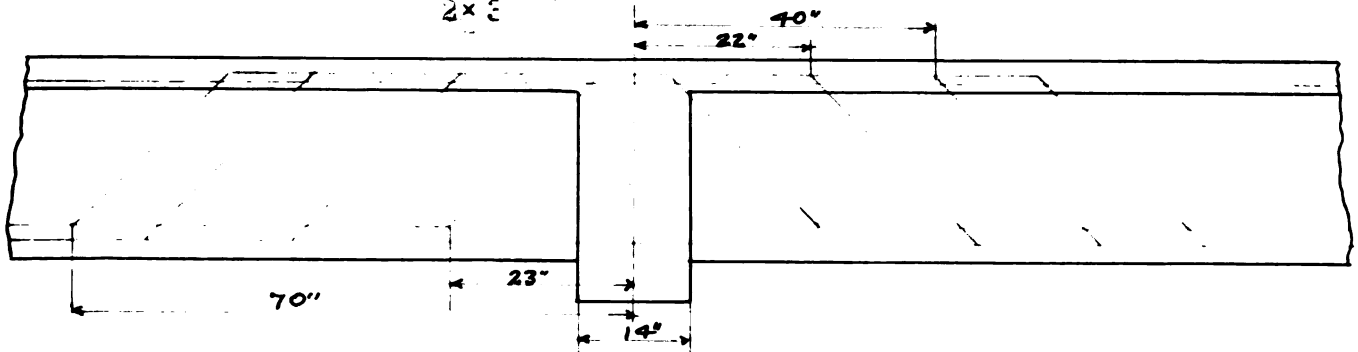
$$\frac{20}{2} \left(1 - \sqrt{\frac{8 \times 2 \times 0.307}{12 \times 2.2458}} \right) 12 = 69.6" \text{ from center of support bend up } 2-5/8" \text{ rods}$$

$$\frac{20}{2} \left(1 - \sqrt{\frac{8 \times 2 \times 3}{12 \times 6}} \right) 12 = 22.8" \text{ bend up } 1-5/8" \text{ rod}$$

Bars over support

$$\frac{2 \times 0.507 \times 20 \times 12}{2.2458 \times 3} = 21.9" \text{ turn down one bar at top}$$

$$\frac{20 \times 12}{2 \times 3} = 40" \text{ turn down two bars}$$



Distance to point where web reinforcing is not needed

$$x = \frac{20}{2} - \frac{40 \times 10 \times 0.924 \times 22}{1730} = 5.3' \text{ or } 63.6"$$

$$AA' = \frac{2 \times 17300}{3 \times 0.85 \times 22} = 616.7 \text{ lbs.}$$

Tensile value of 1 - 5/8" rod, $0.307 \times 16000 = 4912 \text{ lbs.}$

allowable distance between bent up rods,

$$3/4d = \frac{3 \times 22}{4} = 16.5" \quad 70 - 23 = 47", \text{ spacing to great.}$$

$$i = \frac{2.4u_d}{f_s} = \frac{2.4 \times 80 \times 22}{16000} = 0.264, \text{ max. dia. stirrups}$$

use 5/16" round rods.

$$s = \frac{3a_s f_s j d}{2V}, \text{ minimum spacing for stirrups}$$

$$s = \frac{3 \times 0.0767 \times 2 \times 16000 \times 0.85 \times 22}{2 \times 17300} = 3.9"$$

$$\text{at } 1/8, s = 4/3 \times 3.9 = 5.2"$$

$$\text{at } 1/4, s = 2 \times 3.9 = 7.8"$$

place 1st at 2" from edge of girder, the 5 at 4" c-c. 4 at 7" c-c. 1 at 1'0", 2 at 1'6".

The bars over support should extend to the one-third point of span or 6'8" past center of support.

Allowable stress in compressive rods,

$$\frac{8488 \times 0.75}{1 \times 80} = 19.8", \text{ use } 20".$$

GIRDER

Span, 13' c-c. concentrated loads at 3rd points.

Weight of stem, 450 l s. linear foot.

Reaction of concentrated loads $17300 \times 2 = 34600$

Uniform load on beam $\frac{34600 \times 3}{13} + 450 = 8217 \text{ lbs. per ft.}$

$$M = \frac{w l^2}{12} = \frac{8217 \times 13^2 \times 14}{12} = 2,014,308 \text{ in lbs.}$$

less 10 per cent = 1,812,878 in lns.

Total V = $34600 + 3 \times 450 = 35350 \text{ lbs.}$

cross section of stem $\frac{35350}{105} = 336 \text{ sq. in.}$

$$d = \sqrt{\frac{M}{f_s p}} + \frac{t}{2} \text{ for economical depth.}$$

b'	d	b' × d
12"	25.8	309.6
13"	24.8	322.8
14"	24.0	336.0
15"	23.2	348.0
16"	22.6	361.6

If b is taken at 12", use d at 31", $12 \times 31 = 372 \text{ sq. in.}$

If b is taken at 14", use d at 26.5",

Using $d = 26.5"$

$$\text{flange width} = (4 \times 8) + 26.5 = 58.5"$$

$$\frac{M}{bd^2} = \frac{1812878}{58.5 \times (26.5)^2} = 44.1$$

$$\text{for } \frac{M}{bd^2} = 44.1 \text{ and } \frac{t}{d} = \frac{4}{26.5} = 0.151$$

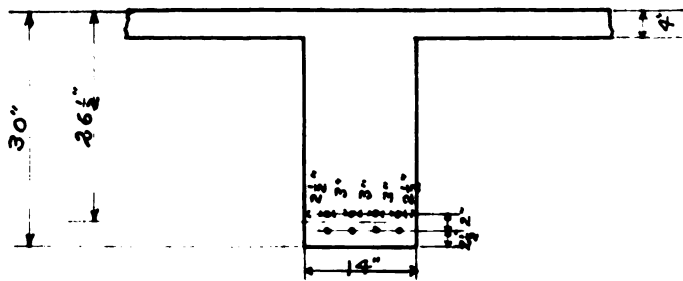
Diagram 8 shows $f_c = 425$ per sq. in. and $j = 0.93$

$$a_s = \frac{1812878}{16000 \times 0.93 \times 26.5} = 4.59 \text{ sq. in.}$$

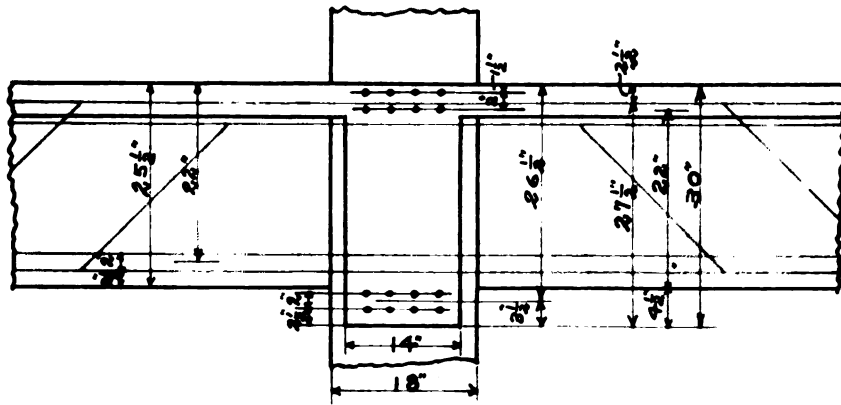
8 - $7/8"$ rods total area 4.81 sq. in. will be used.

Bond stress at top of beam

$$u = \frac{38650}{8 \times 2.749 \times 0.85 \times 26.5} = 73.0 \text{ lbs.}$$



$$\frac{14 \times 26 \times 150}{144} = 379.1 \text{ lbs. per linear foot, assumed value safe side.}$$



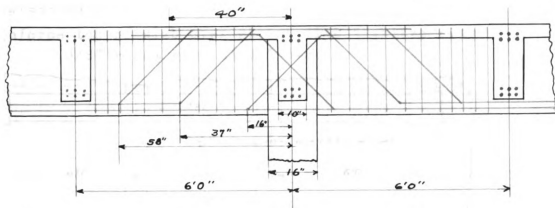
$$\frac{d'}{d} = \frac{2.5}{27.5} = 0.091$$

$$p = \frac{4.81}{14 \times 27.5} = 0.0124, \quad p' = 0.5p$$

Table 11 gives $L = 0.243$ and $K = 0.0110$

$$f_c = \frac{1812878}{15(27.5)^2 \times 0.243} = 657.6 \text{ lbs. per sq. inch.}$$

$$f_s = \frac{1812878}{15(27.5)^2 \times 0.011} = 14530 \text{ lbs. per sq. inch.}$$



Maximum shear = 38650 lbs.

On opposite side at one-third point shear =

$$38650 - 6 \times 450 = 35950 \text{ lbs.}$$

On side toward center, shear = $35950 - 34600 = 1350 \text{ lbs.}$

$$v = \frac{1350}{14 \times 26.5} = 3.6 \text{ lbs. sq. inch.}$$

Use web reinforcing from support out to beam

$$\text{At support, } j = \frac{38650}{0.85 \times 27.5} = 1654 \text{ lbs. linear inch.}$$

$$\text{At one-third point } \frac{35950}{0.83 \times 26.5} = 1459 \text{ lbs. linear inch}$$

The total diagonal tension

$$\left(\frac{1654 + 1459}{2} \right) \times 6 \times 12 = 112,068 \text{ lbs.}$$

$\frac{2}{3} \times 112,068 = 74,712 \text{ lbs}$ to be taken by web reinforcing.

Bend 6 rods.

$$\frac{6 \times 0.16013 \times 16000}{0.7} = 22,178 \text{ lbs. which is in excess of that to be provided for.}$$

Assume a 16" column.

$$l = 13' \quad \frac{l}{3} = 6' \quad \frac{2 \times l}{3} = 16"$$

$$\frac{18 \times 2}{3} = 12", \text{ bend 2 rods up}$$

Allow 4" and bend up at 16" from center of cross beam

Bend next two up at 37" from beam.

Bend next two up at 58" from beam.

Use 1/2 in. stirrups.

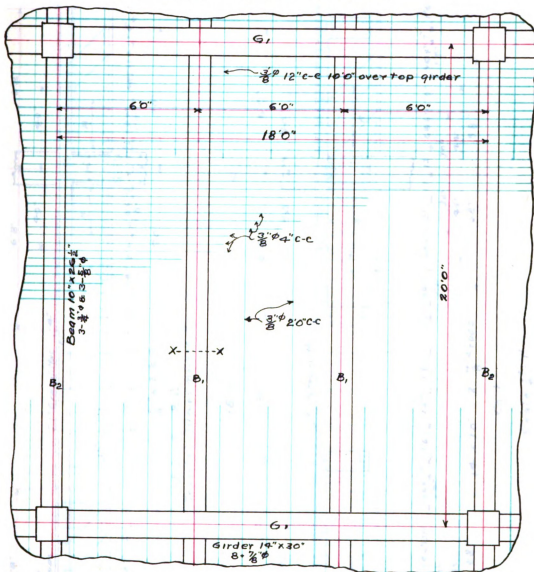
The value of each in tension is $2 \times 0.196 \times 16000 = 6270 \text{ lbs.}$

The shear to be provided for between beam and 1st bent

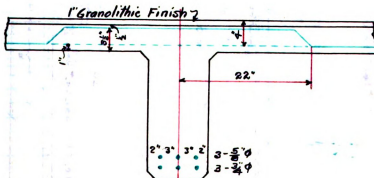
rod is 1459 lbs per linear inch.

$$\text{Space stirrups } \frac{6270}{1459} = 4.3", \text{ use } \frac{4}{8}"$$

Tension rods will extend over $\frac{14530 \times 0.875}{4 \times 80} = 39.69"$, use 40".
 Compression rods will extend thru $\frac{657 \times 6 \times 14}{4 \times 80} = 28.7"$, use 29".
 Reinforce top of slab over girder with traverse rods
 3/8" round rods at 12" c-c.



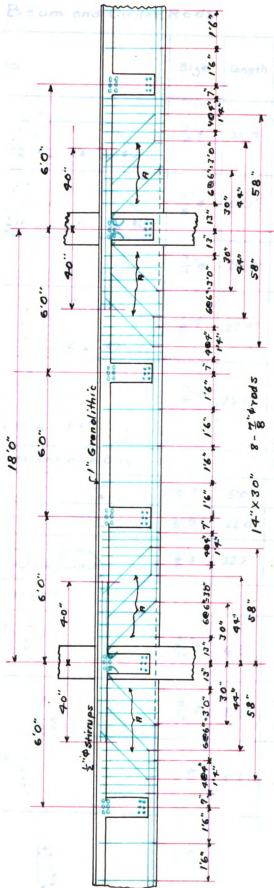
Slab Plan
 $\frac{1}{4}" = 1'$



Cross Section
 of X-X
 $\frac{3}{8}" = 1'$



B1 & B2
Inverted stirrups marked 'H'



Scale $\frac{1}{4} = 1'$

On page 106, Volume 2, Hool, "appears this equation for the economical dept of tile floor beams.

$$d = \sqrt{\frac{c_c r M}{f_s (b' c_c + 144 c_t)}} + \frac{t}{2}$$

Let all letters have same values as previously states.

c_t = cost of tile per 1" depth.

$$C = c_c b' d' + \frac{c_c r M}{f_s (d' + \frac{t}{2})} + 144 d' c_t$$

Place first differential of C with respect to $d' = 0$ and solve for d.

$$\frac{DC}{Dd'} = c_c b' + \frac{(2f_s d' + f_s t) 0 - 2c_c r M (2f_s)}{(2f_s d' + f_s t)^2} + 144 c_t = 0$$

$$c_c b' (2f_s d' + f_s t)^2 - 4c_c r M f_s + 144 c_t (2f_s d' + f_s t)^2 = 0$$

$$(c_c b' + 144 c_t) (2f_s d' + f_s t)^2 = 4c_c r M f_s$$

$$(2f_s d' + f_s t)^2 = \frac{4c_c r M f_s}{c_c b' + 144 c_t}$$

$$2f_s d' + f_s t = 2 \sqrt{\frac{c_c r M f_s}{c_c b' + 144 c_t}}$$

$$2d' + t = 2 \sqrt{\frac{c_c r M}{f_s (c_c b' + 144 c_t)}}$$

$$d' = \sqrt{\frac{c_c r M}{f_s (c_c b' + 144 c_t)}} - \frac{t}{2}$$

$$\text{but } d = d' + t$$

$$d = \sqrt{\frac{c_c r M}{f_s (c_c b' + 144 c_t)}} + \frac{t}{2}$$

On page 112 the following design is desired.

Design an interior panel of a one way floor to carry a live load of 120 lbs per square foot. Girders spaced 19 feet on centers.

WORKING STRESSES:

Bearing 32.5 percent compression.

Shear 6 per cent compression.

Bond 4 per cent compression.

Steel, 16000 lbs per inch.

Concrete, 2000 lb. 1:2:4, 650 lbs.

$n = 15$

4" ribs for small beams, 2" topping, 36" flange for girder, $r = r = 70$, $c_c = 20$ cents. Finished floor 7/8" maple nailed on 2"x3" sleepers. Sleepers rest on concrete slab, 1:3:6, cinder concrete between them.

Weights per sq. ft. floor area:

Wooden floor, 5 lbs.

Concrete, 15 lbs., cinder.

Sleepers, 2 lbs.

Plaster, 5 lbs.

Assume a 9" tile.

Total load per line foot of beam:

Live load $= 120 \times \frac{16}{12} = 160$ lbs. 160 lbs,

Wood floor, $5 \times \frac{4}{3} = 7$ lbs. 7 lbs.

Sleepers, $2 \times \frac{4}{3} = 3$ lbs. 3 lbs.

Concrete filling $15 \times \frac{4}{3} = 20$ lbs

Concrete topping $160 \times \frac{2 \times 16}{2 \times 12 \times 12} = 42$ lbs.

Stem, $\frac{4 \times 9}{144} \times 160 = 38$ lbs.

Tile 33 lbs.

Plaster, $5 \times \frac{4}{3} = 7$ lbs.

Total load per linear foot 310 lbs.

Bending moment;

$$M = \frac{wl^2}{12} \frac{310 \times 19 \times 19 \times 12}{12} = 111,710 \text{ in. lbs.}$$

d for economical depth, when $c_c = 1\frac{1}{2}$ cents

$$d = \sqrt{\frac{c_c r M}{f_s (b' c_c + 144 c_t)}} + \frac{t}{2} = \sqrt{\frac{20 \times 70 \times 111710}{16000 (4 \times 20 + 144 \times 1.25)}} + \frac{5}{4} = 7.35$$

Try d at 7.5", use 2" concrete below steel, this will allow the use of 9" tile.

$$\text{Checking, } \frac{M}{bd^2} = \frac{111710}{16(7.5)^2} = 124.1$$

$$\frac{t}{d} = \frac{2.2}{7.8} = 0.33$$

Diagram 8 shows concrete stress = 725 lbs, this is too high.
If d is taken at 9", use 1 1/2" concrete below steel, 8" tile.

$$\frac{M}{bd^2} = \frac{111710}{16 \times 9^2} = 86.2 \text{ lbs.}$$

$$\frac{t}{d} = \frac{2.5}{9} = 0.277$$

Diagram 8 shows concrete stress = 575 lbs, allowable.

$$j = 0.89$$

$$a_s = \frac{M}{f_s j d} = \frac{111710}{16000 \times 0.89 \times 9} = 0.8717 \text{ sq. in.}$$

use 2 - 3/4" round rods in each rib, lay one straight, bend one at each end, thus having as much steel over support as at center of span.

$$\text{Bend up at, } \frac{19}{2} \left(1 - \sqrt{\frac{8.1}{12 \times 2}} \right) 12 = 49"$$

Project thru at top one third span, 6.5'

$$\text{Shear at support, } V = \frac{310 \times 19}{2} = 2945 \text{ lbs.}$$

Bond of two rods at support

$$u = \frac{V}{\sum o_j d} = \frac{2945}{2 \times 2.36 \times 0.89 \times 9} = 73 \text{ lbs.}$$

The distance to point where stirrups are necessary

$$x = \frac{19}{2} - \frac{40 \times 1 \times 0.89 \times 9}{310} = 5.4' = 65 \text{ inches}$$

Using 1/4" stirrups, bent ends, spacing is

$$s = \frac{2 \times 10,432 \times 2 \times 16000 \times 0.89 \times 9}{2945} = 6.4", \text{ use } 6.25"$$

Beam moment at edge of girder flange

$$M = \frac{310 \times 19 \times 16 \times 12}{12} = 79360 \text{ in. lbs.}$$

$$\frac{a'}{d} = \frac{1.5}{9} = 0.167$$

$$p' = p = \frac{0.8717}{4 \times 9} = 0.0242$$

Table 11 shows $L = 0.380$, $K = 0.0204$

$$f_c = \frac{79360}{4 \times 9^2 \times 0.38} = 645 \text{ lbs. per sq. inch.}$$

$$f_s = \frac{79360}{4 \times 9^2 \times 0.0204} = 12008 \text{ lbs. per sq. in.}$$

Load on floor is $\frac{310 \times 19}{16} = 232.5 \text{ lbs. per sq. ft.}$

Girder load $232.5 \times 19 = 4417.5 \text{ lbs. per linear foot.}$

Girder weight 375 lbs. per foot.

Total weight $4417.5 + 375 = 4792.5$, use 4800 lbs.

$$M = \frac{4800 \times 18 \times 12}{12} = 1732800 \text{ in. lbs.}$$

Limiting depth of girder is 36", effective depth 32.5"
b assumed at 36"

$$\frac{M}{bd^2} = \frac{1732800}{36 \times 32.5^2} = 45.5$$

Table 3 shows $p = 0.0031$, $j = 0.91$

$$a_s = 0.0031 \times 36 \times 32.5 = 3.627 \text{ sq. in.}$$

use 8 - 13/16" round rods, total area 4.15

Bending moment of girder

$$1732800 \text{ less } 10 \text{ per cent} = 1559520 \text{ lbs. ins.}$$

$$\text{Shear } 4800 \times 8.5 = 40800 \text{ lbs.}$$

$$\text{Cross section of web, } b = \frac{V}{av} = \frac{40800}{32.5 \times 120} = 11.7", \text{ use } 12"$$

$$u = \frac{40800}{8 \times 2.553 \times 0.85 \times 32.5} = 80.8 \text{ lbs. sq. in.}$$

$$\text{weight of stem } \frac{12 \times 27 \times 150}{144} = 337.5 \text{ lbs, safe.}$$

$$\frac{d'}{d} = \frac{2.5}{33.5} = 0.075$$

$$p = \frac{4.15}{12 \times 33.5} = 0.0103 \quad p' = 0.5p$$

Table 11 shows $L = 0.222$, $K = 0.0091$

$$f_c = \frac{1559520}{12 \times 33.5^2 \times 0.222} = 522 \text{ lbs. sq. in.}$$

$$f_s = \frac{1559520}{12 \times 33.5^2 \times 0.0091} = 12725 \text{ lbs. sq. in.}$$

$$\text{Bend rods up at } \frac{19}{2} [1 - \sqrt{\frac{3 \times 2}{12 \times 8}}] 12 = 68.4", \text{ bend 2 at } 5'9" \text{ from support.}$$

$$\text{Bend down at } \frac{19}{2 \times 3} = 3.5' \text{ or } 42"$$

Bending at top will control.

Distance to where web reinforcing is not needed

$$\frac{19}{2} - \frac{40 \times 12 \times 0.91 \times 32.5}{4800} = 6.54' \text{ or } 78.48"$$

$$\text{Bend other rods at } 3/4 d = \frac{3 \times 32.5}{4} = 24.4"$$

$$\text{Extend over support } \frac{1}{3} = 6\frac{1}{3}'$$

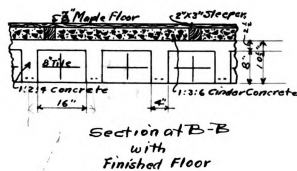
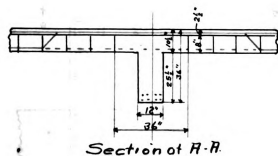
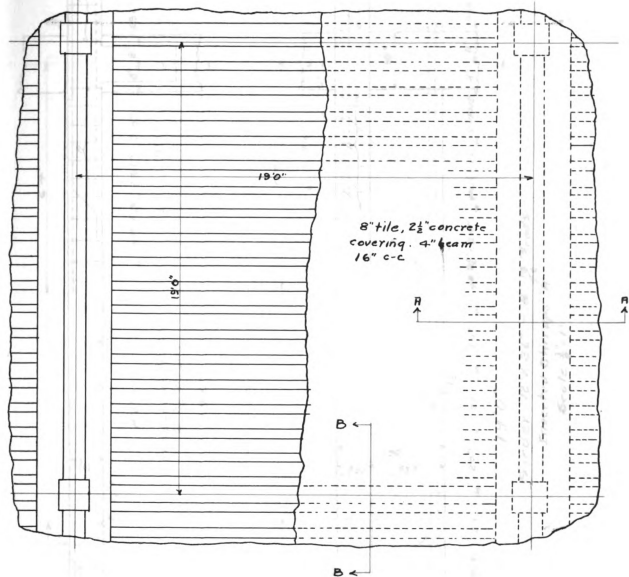
$$\text{Dia. stirrups, } \frac{2.4 u_d}{f_s} = \frac{2.4 \times 80 \times 32.5}{16000} = 0.39, \text{ use } 7/16" \text{ round rod}$$

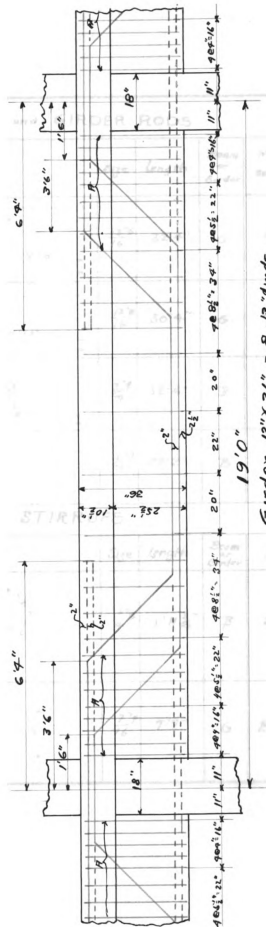
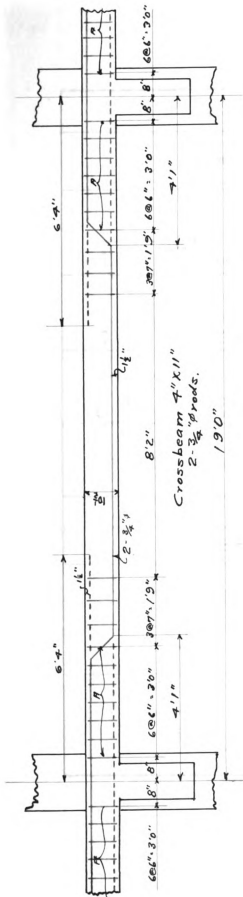
Spacing of stirrups at support,

$$s = \frac{3 \times 0.15 \times 2 \times 16000 \times 0.55 \times 32.5}{2 \times 40800} = 4.3"$$

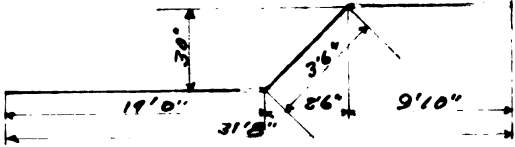
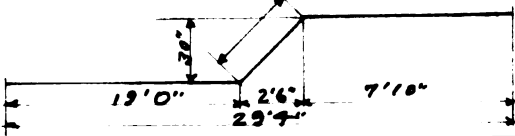
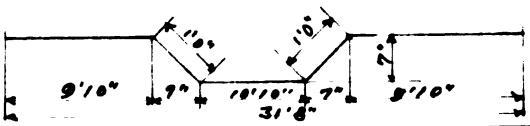
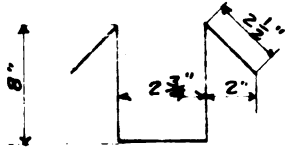
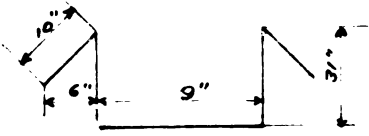
$$\text{at } \frac{1}{3} s = \frac{4 \times 4.3}{3} = 5.7" \quad \text{at } \frac{1}{4} s = 2 \times 4.3 = 8.6"$$

Place first stirrup at 2" from edge of column, at center of span space at 18". Bend 6 rods, 2 go straight thru.





Girder 12" x 36" - 8 - 13" Ø rods
 Inverted stirrups "H"
 Scale $\frac{1}{8}" = 1'$

BEAM and GIRDER RODS					
Bends	Size	Length	Beam or Girder	No. each	Total No
	$\frac{13}{16}$	32'4"	G	4	4
	$\frac{13}{16}$	30'4"	G	4	4
	$\frac{3}{4}$	32'6"	B	1	14
	$\frac{3}{4}$	27'2"	B	1	14
STIRRUPS					
Bends	Size	Length	Beam or Girder	No.	
	$\frac{1}{4}$	1'11 $\frac{3}{4}$ "	B	20	
	$\frac{7}{16}$	7'7"	G	28	

On Page 129, Volume 2, the following design is desired.

Design an interior bay for a flat slab floor carrying a live load of 150 lbs. per sq. ft. Column spacing 18' c-c.

Working stresses, Bearing 32.5 per cent compression

Shear, 6 percent compression,

Bond, 4 per cent compression,

Steel, 16000 lbs. sq. in.

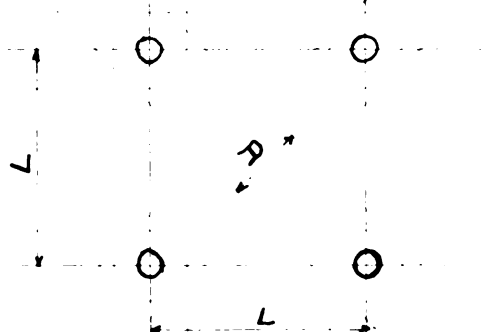
Concrete, 2000 lb. 1:2:4, 750 lbs.

$n = 15$

Assumptions: d at 8",

$w = 100$ lbs. sq. ft.

Total weight per sq. ft. 250 lbs.



$$D = 0.42L = 0.42 \times 18 = 7.56'$$

$$M_1 = \frac{WL}{48(0.42L)} = \frac{W}{20} = \frac{18^2 \times 250}{20} = 4050 \text{ lbs.ft. per ft. width.}$$

From Table 2, $K = 133.8$, $p = 0.0097$ $j = 0.862$

$$d = \sqrt{\frac{M}{bK}} = \sqrt{\frac{4050}{133.8}} = 5.5"$$

$$a_s = 0.0097 \times 18 \times 5.5 = 0.64 \text{ sq. in.}$$

Total steel required for one band

$$7.6 \times 0.64 = 4.864 \text{ sq. inches.}$$

Area rods in one band over support,

$$\frac{4.864}{2} = 2.432 \text{ sq. inches}$$

Use 10 - 9/16" round rods 9" c-c.

Total thickness of slab

$$d + 3 \times \frac{1}{4} + \frac{1}{2} = 7\frac{1}{2}"$$

Assumed diameter of capitol = 50"

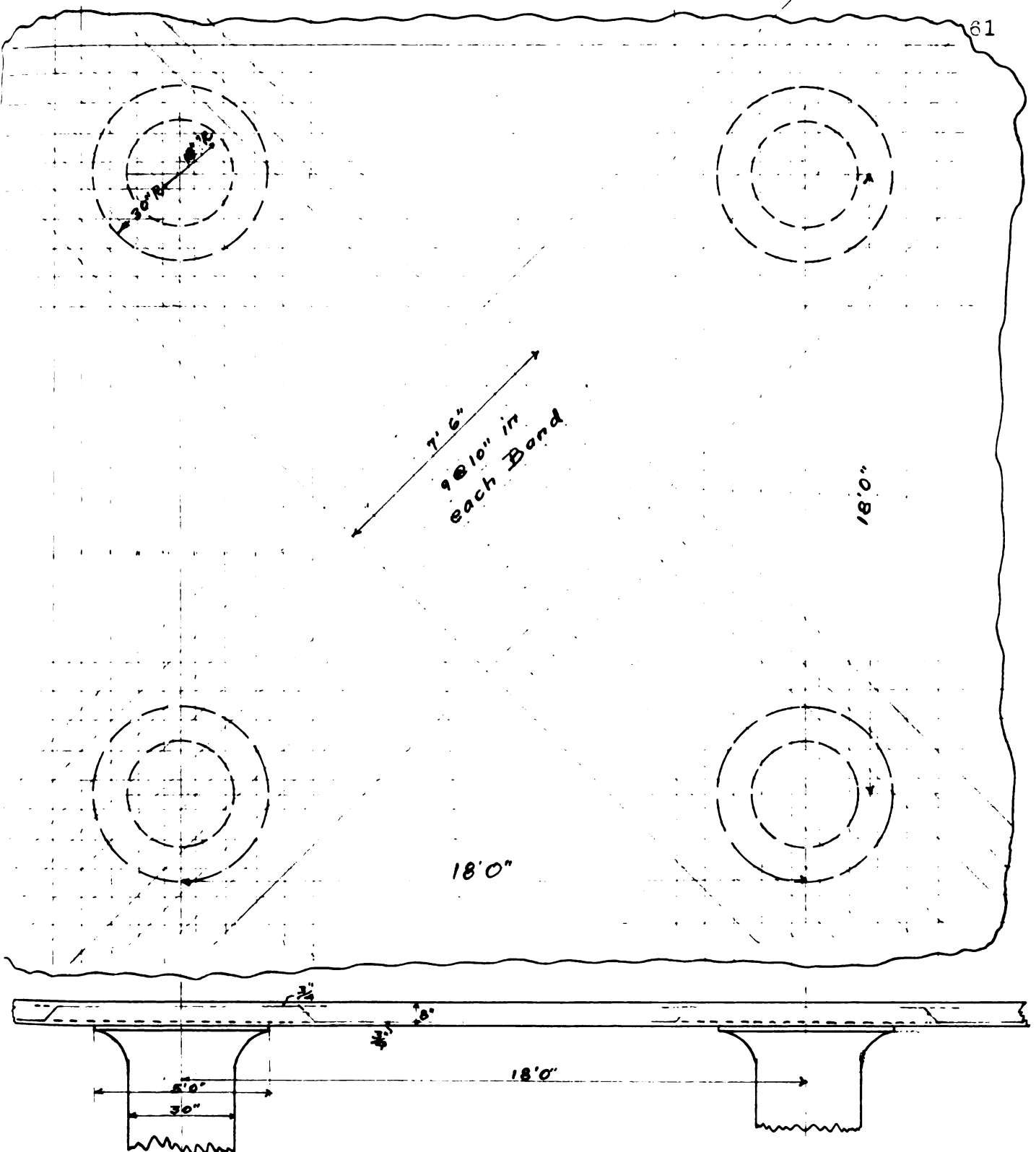
Area of concrete in shear at diameter + thickness of slab to steel $\pi \times 61 \times 5.5 = 1053.17$ sq. inches

Total load on column, $18 \times 18 \times 250 = 81000$ lbs.

Load on capitol $\pi \times (\frac{50}{2})^2 \times 250 = 4863$ lbs.

$81000 - 4863 = 76137$ lbs. to be considered.

$$\frac{76137}{1054} = 72 \text{ lbs. compression sq. in.}$$



STEEL LIST

Bends	Size	Length	No.
	7/16"	25'3"	18
	9/16"	35'9"	18

Scale 1/4" = 1'

Circular Plate method applied to the previous data, loads and stresses as given before.

M_1 = bending moment per unit width of section, causing radial fiber stress at any distance r .

M_2 = bending moment per unit width of section causing circumferential fiber stress at any distance r .

q = load per unit area.

R_1 = ordinate to proper solid curve, Figure 1.

C_1 = ordinate to proper dotted curve, Figure 1.

p = load along circumference per unit length.

R_2 = ordinate to proper solid curve, Figure 2.

C_2 = ordinate to proper dotted curve, Figure 2.

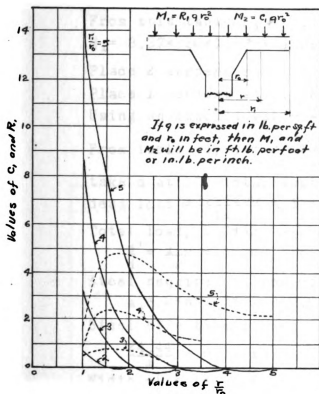


Fig. 1.

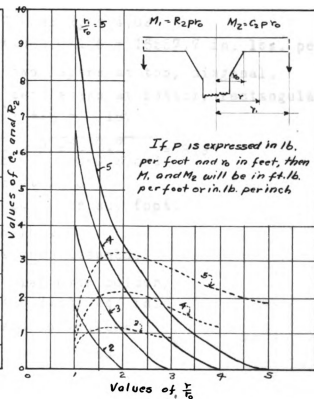


Fig. 2.

$$\text{then } M_1 = R_1 q r_0^2 \quad \text{and } M_1 = R_2 p r_0$$

$$M_2 = C_1 q r_0^2 \quad \text{and } M_2 = C_2 p r_0$$

: Assume diameter of column capitol at 0.2L, $0.2 \times 18 = 3.6'$

$$r_1 = \frac{0.6 \times 18}{2} = 5.4' \quad r_0 = \frac{0.2 \times 18}{2} = 1.8'$$

Total loading 300 lbs per sq. ft.

Area slab, $18 \times 18 = 324$ sq. ft.

Area cir. plate $3.6^2 \pi = 38.8$ sq. ft.

Area outside of assumed plate $324 - 38.8$ sq. ft.

$\frac{232.4 \times 200}{2\pi \times 5.4} = 2065.4$ circumferential unit loading per foot.

$2\pi \times 5.4$

$$\frac{r_1}{r_0} = \frac{3.4}{1.8} = 3.0$$

Value of constants R_1 and R_2 based on Poisson's ratio of 0.1 and with $\frac{r_1}{r_2} = 1$

Values of $\frac{r_1}{r_2}$													
	2.0	2.2	2.4	2.6	2.8	3.0	3.2	3.4	3.6	3.8	4.0	4.5	5.0
R_1	0.72	1.08	1.52	2.04	2.66	3.37	4.17	5.06	6.05	7.15	8.35	11.77	15.93
R_2	1.66	2.10	2.53	3.03	3.51	4.02	4.54	5.07	5.60	6.17	6.73	8.12	9.72

From this table $R_1 = 3.37$ and $R_2 = 4.02$

$$M_1 = 3.37 \times 300 \times 1.8^2 + 4.02 \times 2065.4 \times 1.8 = 16557.7 \text{ in. lbs. per in. cir}$$

Place 2 percent steel in two layers at top, diagonal.

Place 1 percent steel in two layers at bottom, rectangular.

Using an assumed value $\frac{d'}{d} = \frac{1.2}{8} = 0.187$

$$\text{From Table 11, } a = \sqrt{\frac{M}{f_c L b}} = \sqrt{\frac{16557.7}{750 \times 0.275 \times 1}} = 8.9"$$

take d at 9", total depth at 10.5"

$$\text{dead load } \frac{10.5 \times 150}{12} = 132 \text{ lbs. per sq. fpoft.}$$

Total load, $150 + 132 = 282 \text{ lbs.}$

$$\frac{d'}{d} = \frac{1.2}{9} = 0.186, \text{ less than assumed.}$$

Steel required at top of slab over column,

$$a_s = 2 \times \pi \times 1.8 \times 12 \times 9 \times 0.02 = 24.44 \text{ sq. in.}$$

Steel effective on four side of column,

$$\frac{24.44}{4} = 6.11 \text{ sq. in. in rod area.}$$

$$\text{Width of band, } 0.42L = 0.42 \times 18 = 7.56'$$

5/8" round rods spaced at 4" c-c sufficient,

At bottom of slab at column space 6" p-c

Diagonal distance between centers of inflection,

$$25.4 - 10.5 = 14.9 \text{ ft.}$$

$$W = \frac{wl^2}{24} = \frac{300 \times 14.9^2}{24} = 31,974 \text{ in lbs. per ft.}$$

1 1/2" concrete below steel, effective thickness slab 9"

$$K = \frac{W}{bd^2} = \frac{31974}{12 \times 9^2} = 32.89$$

Tables 3 gives, $p = 0.0023$, $k = 0.226$, $j = 0.925$

0.22 percent steel needed in diagonal direction.

$$0.22 \times 6.11 = 1.344, \text{ effective 2 sides, } 2.688 \text{ sq. inches.}$$

Use 2 - 5/8" round rods, spaced 8" c-c

Distance between circles of inflection along rectangular lines, $18 - 10.3 = 7.2'$

$$M = \frac{wl^2}{12} = \frac{300 \times 7.2^2 \times 12}{12} = 15552 \text{ in. lbs. per ft.}$$

$$K = \frac{M}{bd^2} = \frac{15552}{12 \times 9^2} = 16.0$$

$$K = pf_{sj} \quad p = \frac{13.0}{16000 \times 0.85} = 0.0011$$

$$0.11 \times 6.11 = 0.672, \text{ effective 2 sides, } 1.344 \text{ sq. in.}$$

Use 5 - 5/8" round rods spaced 20" c-c

Area of concrete in shear at a distance out from capitol equal to depth of slab

$$\pi \times 60 \times 9 = 1697.1 \text{ sq. in.}$$

$$\text{Load on one column } 18 \times 18 \times 282 = 91368 \text{ lbs.}$$

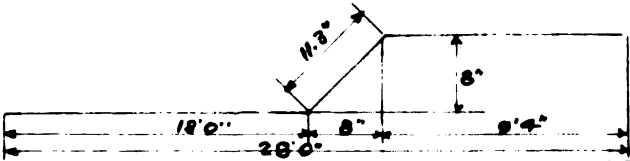
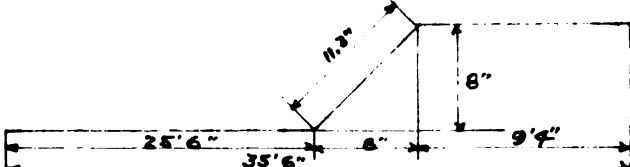
$$\text{Capitol section load, } \pi \times 2.5^2 \times 282 = 553.9 \text{ lbs.}$$

$$\text{Total load causing shear } 85829 \text{ lbs.}$$

$$\frac{85829}{1697} = 50.5 \text{ lbs. per sq. in shear.}$$

Bend the rods and pass over top of capitol.

Steel list.

Bends	Size	Length	No.
	5/8"	28'3 1/2"	10
	5/8"	35'9 1/2"	18

Design No. 8, page 178.

Interior floor bay. Columns 16' x 16' on centers. One intermediate beam. Live load 200 lbs. per sq. ft. 1" granolithic finish of 1:2 mortar.

Working stresses, $r = 60$

Bearing, 32.5 percent compression,

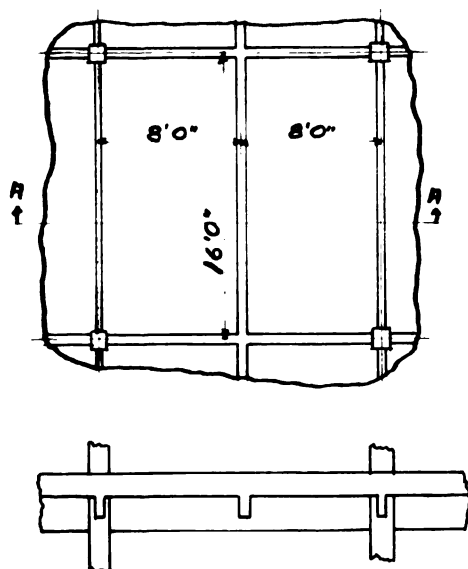
Shear, 6 percent compression,

Bond, 4 percent compression,

Steel, 16000 lbs. sq. in.

Concrete, 650 lbs. compression.

$n = 15$



Sec. at A-A.

Table 6 shows for 8' span, 5" slab, $d = 4$

Safe live and dead load, $269 \times 1.2 = 323$ lbs, slab 62 lbs. per ft. 1" granolithic finish weights 12.5 lbs. sq. ft.

Safe live load, $323 - 75 = 248$ lbs. sq. ft.

For 5" slab, $a_s = 0.370$

Table 4 shows that 3/8" round rods spaced 3.5" c-c will give sufficient area.

$$p = \frac{0.11}{3.5 \times 4} = 0.0078$$

Safe load formula, $w = 1064 \frac{d^2}{l^2} = 1064 \frac{4^2}{7^2} = 343$, this is more than the 323 lbs computed load, therefore safe.

Place 4 - 3/8" round rods transverse in each panel for temperature and shrinkage.

Weight of steel 75 lbs. per ft.

Total weight $200 + 75 = 275$ lbs. sq. ft.

CROSSBEAMS.

Dead and Live Load per ft. of beam, $8 \times 275 = 2200$ lbs.

Assumed weight of beam 250 lbs. per linear foot

Total load per foot of length, $2200 + 250 = 2450$ lbs.

$$V = \frac{2450 \times 16}{2} = 19600 \text{ lbs.}$$

$$M = \frac{2450 \times 16 \times 12}{12} = 627,200 \text{ in lbs.}$$

Cross section determined by shear, $\frac{19600}{105} = 186.6 \text{ sq. in.}$

$$d = \sqrt{\frac{RM}{f_s b'}} + \frac{t}{2}$$

$$b' = 8" \quad d = 19.6" \quad b'd = 156.8 \text{ sq. in.}$$

$$b' = 9" \quad d = 18.6" \quad b'd = 167.4 \text{ sq. in.}$$

$$b' = 10" \quad d = 17.8" \quad b'd = 178.0 \text{ sq. in.}$$

$$b' = 11" \quad d = 17.1" \quad b'd = 188.1 \text{ sq. in.}$$

Using 8 rods, so as to bend up for diagonal tension and 10" spacing make $d = 19"$ as necessary to sufficient area. Gives an economical beam.

Width of flange $(8 \times 5) + 10 = 50"$

$$\frac{M}{bd^2} = \frac{627200}{50 \times 19^2} = 34.74$$

$$\frac{t}{d} = \frac{5}{19} = 0.26$$

Table 9 shows p lies between 0.002 and 0.004.

$$p = 0.002 + \frac{9.06 - 0.002}{14.1} = 0.0032$$

$$j = 0.91$$

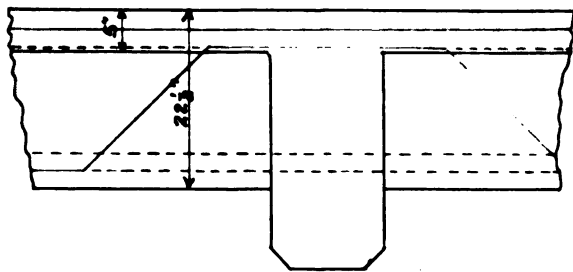
$$a_s = 50 \times 19 \times 0.0032 = 3.04 \text{ sq. in.}$$

Use 4 - $3/4"$ round rods, and 4 - $5/8"$ round rods.

Total area 3.00 sq. in.

Put $3/4"$ on bottom, $5/8"$ on top, bend 4 up over support, 4 lap support on bottom of beam.

$$u = \frac{19600}{[(4 \times 2.356) + (4 \times 1.964)](0.91 \times 19)} = 65.6 \text{ lbs. sq. in.}$$



$$\frac{d'}{d} = \frac{3.5}{19} = 0.184$$

$$p = \frac{3.00}{10 \times 19} = 0.0157 = p'$$

From table 11, $L = 0.297$, $K = 0.0143$

$$f_c = \frac{627200}{10 \times 19^2 \times 0.297} = 535 \text{ lbs. sq. in.}$$

$$f_s = \frac{627200}{10 \times 19^2 \times 0.0143} = 12149 \text{ lbs. sq. in.}$$

Bend up bottom rods at

$$\frac{16}{2} \left[1 - \sqrt{\frac{8 \times 2 \times 0.307}{12 \times 3.00}} \right] 12 = 60.5", \text{ bend two rods.}$$

$$\frac{16}{2} \left[1 - \sqrt{\frac{8 \times 4}{12 \times 8}} \right] 12 = 40.3" \text{ bend two rods.}$$

Bend down from top,

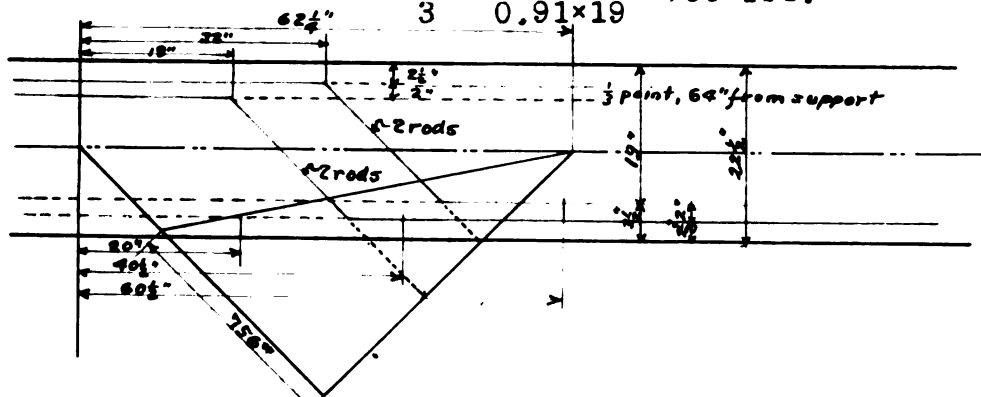
$$\frac{2 \times 0.442 \times 16}{3.00 \times 3} = 1.57' \text{ or } 19" \text{ from support.}$$

$$\frac{1}{3} \times 1 = 2\frac{1}{3}' = 32" \text{ from support}$$

Distance out to where web reinforcement is not needed,

$$\frac{16}{2} - \frac{40 \times 10 \times 0.91 \times 19}{2450} = 5.13' \text{ or } 62.25"$$

$$\text{Diagonal tension } \frac{2}{3} \times \frac{19600}{0.91 \times 19} = 756 \text{ lbs.}$$



Points at top control bending.

$$\text{Tension value of one } 3/4" \text{ rod} = 0.4418 \times 16000 = 7070 \text{ lbs.}$$

Maximum value of diameter of stirrup

$$\text{Table 15, } 0.012 \times 19 = 0.228 \text{ in.}$$

Use 5/8" round, diameter 0.11"

$$\text{spacing, } \frac{3a_s f_c j d}{2V} = \frac{3 \times 0.11 \times 2 \times 16000 \times 0.85 \times 19}{2 \times 19600} = 4.3", \text{ use } 4"$$

$$\text{at } \frac{1}{8} = \frac{4 \times 1.3}{3} = 5.7" \quad \text{at } \frac{1}{4}, s = 2 \times 1.3 = 2.6"$$

Space first stirrup 2" from edge of girder.

Girder has span of 16' with concentrated load at center.

Weight of stem assumed at 500 lbs per linear foot.

$$\text{Reaction of concentrated loads } 2 \times 19600 = 39200 \text{ lbs.}$$

$$\frac{39200 \times 2}{16} + 500 = 5400 \text{ lbs. uniform load per foot.}$$

$$M = \frac{5400 \times 16^2 \times 12}{12} = 1,332,400 \text{ in. lbs.}$$

$$\text{deduct 10 percent, } M = 1,244,160 \text{ in lbs.}$$

$$\text{Maximum shear, } 39200 + 6 \times 500 = 43200 \text{ lbs.}$$

$$\text{Cross section of web, } \frac{43200}{105} = 412 \text{ sq. in.}$$

$d = \sqrt{\frac{M}{f_s b}} + \frac{t}{2}$	b'	d	b'd
	10"	24.1"	241.0 sq. in.
	11"	23.5"	259.3 " "
	12"	22.3"	267.6 " "
	13"	21.4"	278.2 " "

Use 10 rods, 5 in a row

$$\text{girder } 14" \times 30.0", \text{ total } d = 33.5"$$

Width of flange $(8 \times 5) + 14 = 54"$

$$M = \frac{1244160}{64 \times 30} = 25.6$$

Weight of stem $\frac{14 \times 53.5 \times 150}{12 \times 12} = 433.5 \text{ lbs.}$

$$\frac{t}{d} = \frac{5}{30} = 0.16666$$

Diagram 8 shows $f_c = 250$ $j = 0.96$

$$a_s = \frac{1244160}{16000 \times 0.96 \times 30} = 2.48$$

Use 10 - 5/8" round rods, 3.07 total area.

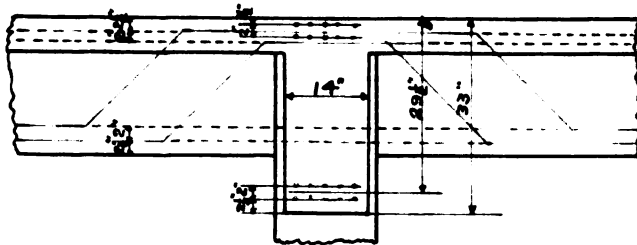
Bond at top of support,

$$u = \frac{43200}{10 \times 1.964 \times 0.96 \times 30} = 75 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{2.2}{30.5} = 0.082$$

$$p = \frac{3.07}{14 \times 30.5} = 0.0072 \quad p' = 0.5p$$

From Table 11, $L = 0.186$ $K = 0.00644$



$$f_c = \frac{1244160}{14 \times 30.5 \times 0.186} = 514 \text{ lbs sq. ft.}$$

$$f_s = \frac{1244160}{14 \times 30.5 \times 0.00644} = 14835 \text{ lbs. sq. in.}$$

At support $\frac{V}{jd} = \frac{43200}{0.88 \times 30.5} = 1666 \text{ lbs linear inch}$

diagonal tension $\frac{1666 \times 5 \times 12}{k} = 79968 \text{ lbs.}$

Two-thirds taken by web reinforcement, 53312 lbs.

8 rods are bent up.

$$\frac{8 \times 0.3068 \times 16000}{0.7} = 56100.5 \text{ lbs taken by rods}$$

Assume an 18" column

Zero moment occurs at $\frac{1}{2}$ of $\frac{1}{2} = 4'$ from center of column

Bend rods at

$$\frac{16}{2} \left[1 - \sqrt{\frac{2 \times 0.3068}{3.07}} \right] = 4.48', \text{ from support, bend 2}$$

$$\frac{16}{2} \left[1 - \sqrt{\frac{4 \times 0.3068}{3.07}} \right] = 2.96', \text{ bend 2}$$

$$\frac{16}{2} \left[1 - \sqrt{\frac{6 \times 0.3068}{3.07}} \right] = 1.84', \text{ bend 2}$$

$$\frac{16}{2} \left[1 - \sqrt{\frac{8 \times 0.3068}{3.07}} \right] = 0.88', \text{ bend 2}$$

Bend down two rods at

$$\frac{2 \times 0.3068 \times 16}{3.07 \times 3} = 1.06' \text{ from support}$$

Bending up will control spacing.

Extend rods for bond,

$$\frac{56100}{8 \times 4 \times 80} = 22"$$

Stirrups will be used near column,

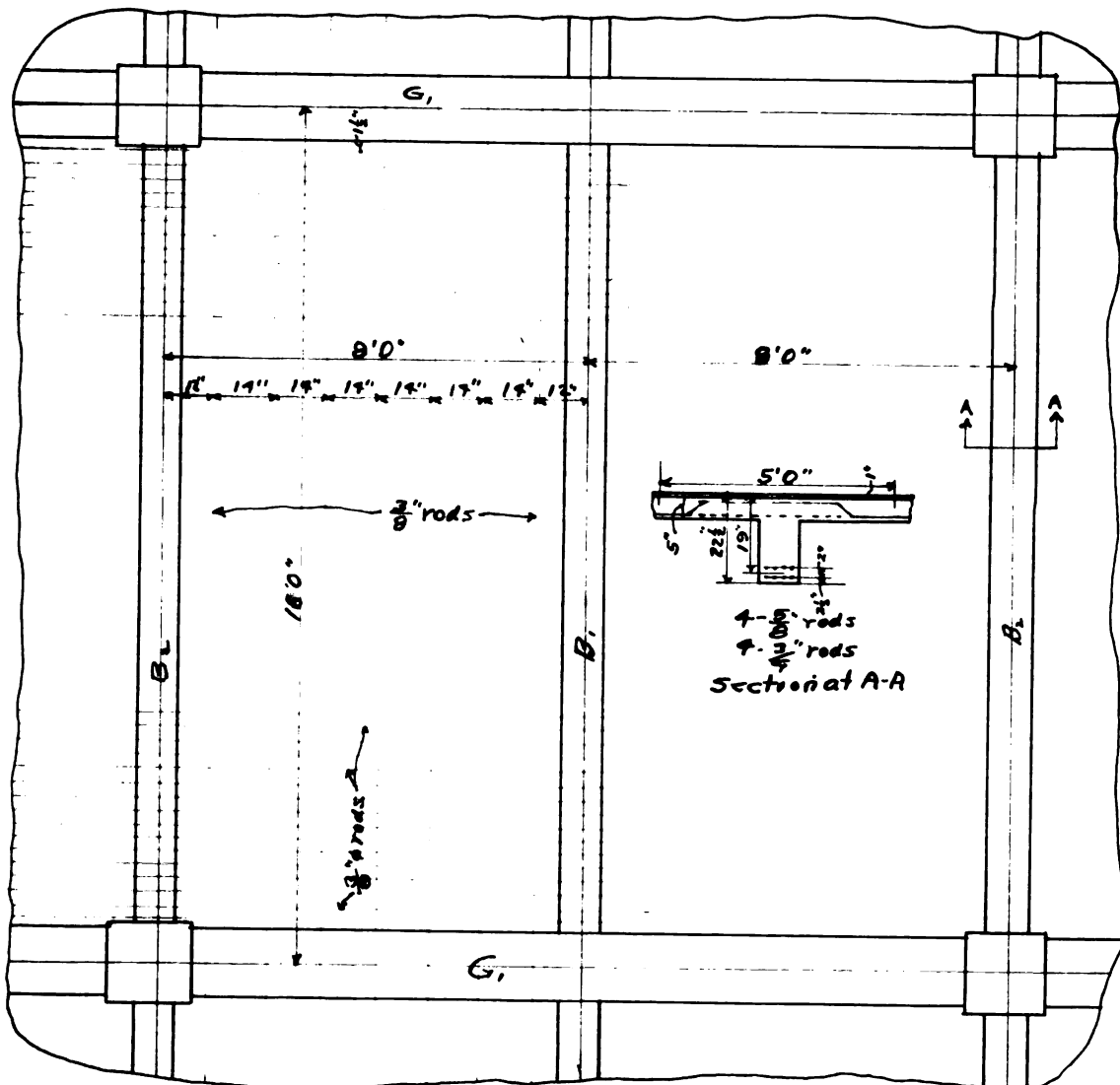
$$0.012 \times 30.5 = 0.366", \text{ max. dia., use } 3/8" \text{ round rods.}$$

$$s = \frac{3a_f s_j d}{2V} = \frac{3 \times 0.11 \times 2 \times 16000 \times 0.85 \times 30.5}{2 \times 43200} = 3.17 \text{ inches}$$

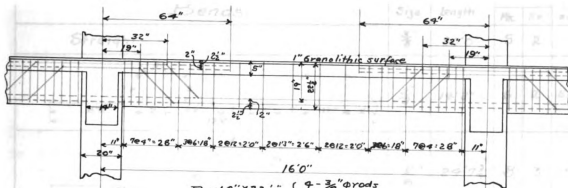
place first stirrup 2" from edge of column.

$$\text{at } \frac{1}{8}, s = \frac{4 \times 3.17}{3} = 4.23 \text{ inches}$$

$$\text{at } \frac{1}{4}, s = 2 \times 3.17 = 6.34 \text{ inches}$$

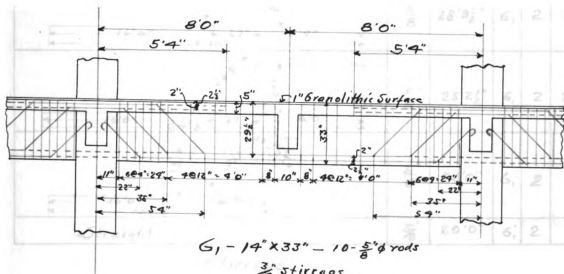


Scale $\frac{1}{4}" = 1'$



B, -10" x 22 1/2" { 7-#4 rods
3/8" stirrups.

Scale 1/4" = 1'0"

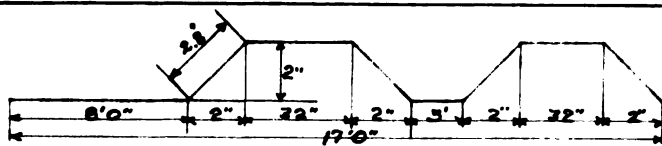
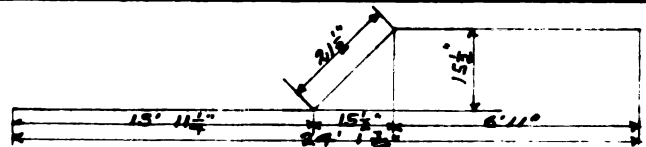
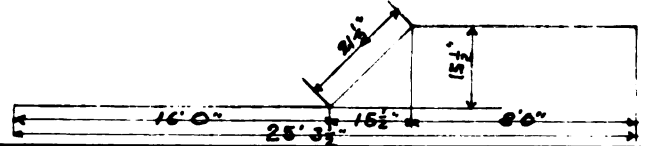
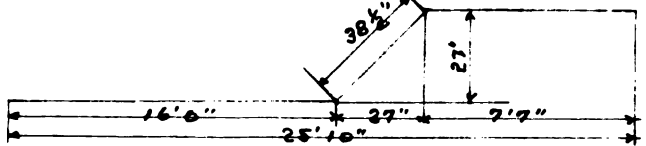
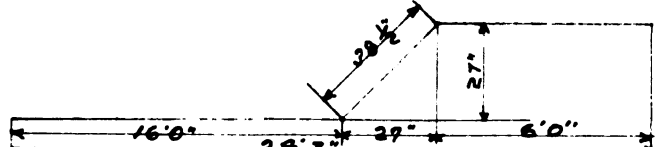
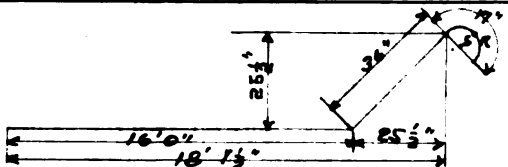


G, -14" x 33" - 10-#5 rods

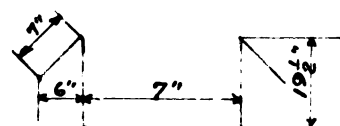
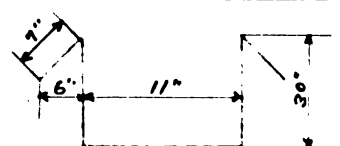
3/8" stirrups

Scale 1/4" = 1'0"

Steel Schedule.

Bends	Size	Length	B. or G.		No. each	Total
			Mk.	No.		
Straight	$\frac{3}{8}$ "	20'0"	5	2	4	8
	$\frac{3}{8}$ "	14'8"	6	2	52	52
	$\frac{5}{8}$ "	24'7 $\frac{3}{4}$ "	B,	3	4	12
	$\frac{5}{8}$ "	26'9 $\frac{1}{2}$ "	B,	3	4	12
	$\frac{5}{8}$ "	28'9 $\frac{1}{2}$ "	G,	2	2	4
	$\frac{5}{8}$ "	25'2 $\frac{1}{2}$ "	G,	2	2	4
	$\frac{5}{8}$ "	20'0"	G,	2	2	4
Straight	$\frac{5}{8}$ "	20'0"	6,	2	4	8

Stirrups.

Bends.						
	$\frac{3}{8}$ "	5'0"	B, B ₂	2	18	36
	$\frac{3}{4}$ "	7'1"	G,	2	35	70

In Chapter 6, there appears a typographical error and a mathematical error at the bottom of Page 190 in the sample design, under cross beams.

The equation is given

$$v = \frac{1535 \times 21}{2} = 16,000 \text{ lbs.}$$

but should solve out

$$v = \frac{1535 \times 21}{2} = 16,100 \text{ lbs.}$$

At the bottom of the page is stated $\frac{16,100}{105} = 153 \text{ sq. in.}$

There should be inserted before this statement the letter A =, to show what the 153 sq. in. refersto.

DESIGN NO. 9.

Interior roof bay, 5 ply felt and gravel roof. Live load 30 lbs per sq. ft. $1\frac{1}{2}$ ' cinder fill, 2" concrete surface. Columns 18' x 20' c-c. Girder span 18', cross beams 6' c-c.

Working stresses; Bearing, 32.5 percent compression.

Shear, 6 percent compression.

Bond, 4 percent compression.

Steel, 16000 lbs. sq. in.

Concrete 650 Lbs compression

n = 15 r = 60

Slab. Loads.

5 ply felt and gravel	0 lbs.
$1\frac{1}{2}$ ' cinders	67.5 lbs.
2" concrete surfacing	25.0 lbs.
Live load,	30.0 lbs.
Total	122.5 lbs, say 130 lbs. ft.

A 2 $\frac{1}{2}$ " slab (d=2) will carry $1.2 \times 151 = 181 \text{ lbs.}$

Load on slab $130 + 38 = 168 \text{ lbs.}$

$a_s = 0.208$ Use 5/16" round rods at 4 $\frac{1}{2}$ " c-c

Place 3 - 5/16" round rods transversely in each panel.

Assume load of stem at 150 lbs per linear foot

Total load $168 \times 6 \times 150 = 1158 \text{ lbs.}$

$$v = \frac{1158 \times 20}{2} = 11580 \text{ lbs.}$$

$$M = \frac{1158 \times 20^2 \times 12}{12} = 463,200 \text{ in. lbs.}$$

$$A = \frac{11580}{105} = 110.2 \text{ sq. in.}$$

b'	d	b'd	Depth for shear
5"	20.1"	100.5	22.0"
6"	17.0"	102.0	18.4"
7"	16.5"	108.5	15.7"
8"	14.7"	117.6	12.8"

Use $b = 8"$, $d = 20"$, total depth $22"$

weight of stem, $\frac{8 \times 19 \times 150}{144} = 163.3$ lbs.

$$b' = 8 + 8 \times 3 = 32"$$

$$M = \frac{463200}{bd^2} = 34.1 \quad \frac{t}{d} = \frac{3}{20} = 0.15 \quad f_c = 316 \quad j = 0.937$$

$$a_s = \frac{463200}{16000 \times 0.937 \times 20} = 1.54 \text{ sq. in.}$$

Use 4 - $3/4"$ round, total area 1.77 sq. in.

$$u = \frac{16000}{4 \times 2.356 \times 0.937 \times 20.5} = 88.4 \text{ lbs.}$$

This bond stress is allowable because of thorough provision for diagonal tension by vertical stirrups.

$$\frac{d'}{d} = \frac{2}{20.5} = 0.097 \quad p = \frac{1.77}{8 \times 20.5} = 0.011 = p'$$

$$L = 0.250 \quad K = 0.0089$$

$$f_c = \frac{463200}{8 \times 20.5^2 \times 0.25} = 551 \text{ lbs. sq. in.}$$

$$f_s = \frac{463200}{8 \times 20.5^2 \times 0.0089} = 15481 \text{ lbs. sq. in.}$$

$$x_2 = \frac{20}{2} \left(1 - \sqrt{\frac{2}{4}} \right) 112 = 36", \text{ bend up 2.}$$

$$x_1 = \frac{20}{2} - \frac{40 \times 10 \times 0.937 \times 18}{1158} = 4.2', 50.5" \text{ to point where web reinforcing is not needed.}$$

$$0.011 \times 18 = 0.198$$

Use $3/8"$ round stirrups bent at upper ends.

$$s = \frac{0.11 \times 2 \times 16000 \times 0.52 \times 20.5 \times 3}{2 \times 11580} = 7.9"$$

$$\text{at } \frac{1}{3}, s = \frac{4 \times 7.9}{3} = 10.5" \quad \text{at } \frac{1}{4}, s = 2 \times 7.9 = 15.8"$$

Length for bond of compression rods.

$$\frac{551 \times 18 \times 0.875}{4 \times 80} = 22.6"$$

GIRDER.

Reaction of concentrated loads $3 \times 11530 = 23160$ lbs.

Assumed dead load of stem 300 lbs

Equivalent loading per foot of length $\frac{23160 \times 3}{18} + 300 = 4160$ lbs

$$V = 23160 + 3 \times 300 = 25860 \text{ lbs.}$$

$$M = \frac{4160 \times 18 \times 12 \times 0.20}{12} = 1,213,056 \text{ in lbs.}$$

$$A = \frac{25860}{138} = 215 \text{ sq. inches.}$$

b'	d	$b'd$	Depth for shear
10"	22.8"	273.6	20.6"
8"	25.4"	203.2	31.0"

Use $b' = 10$, $d = 22"$, total depth $26"$.

$$\text{Weight of stem } \frac{10 \times 22 \times 150}{144} = 229 \text{ lbs.}$$

$$b = 10 + 8 \times 3 = 34"$$

$$\frac{M}{bd^2} = \frac{1213056}{34 \times 22^2} = 73.7$$

$$\frac{t}{d} = \frac{3}{25} = 0.12 \quad f_c = 634 \quad j = 0.948$$

$$a_s = \frac{1213056}{16000 \times 0.948 \times 25} = 3.2 \text{ sq. in.}$$

use 8 - 3/4" round rods, total area 3.53 sq. in.

$$u = \frac{25860}{8 \times 2.356 \times 0.948 \times 25} = 57.8$$

$$\frac{a'}{d} = \frac{2.5}{25} = 0.1 \quad p = \frac{3.53}{10 \times 25} = 0.014 \quad p' = 0.5p = 0.007$$

$$L = 0.261 \quad K = 0.0131 \quad k = 0.428$$

$$f_c = \frac{1213056}{12 \times 25 \times 0.261} = 623 \text{ lbs.}$$

$$f_s = \frac{1213056}{12 \times 25 \times 0.0131} = 12430 \text{ lbs.}$$

$$\frac{V}{jd} = \frac{25860}{0.85 \times 25} = 1217 \text{ lbs linear inch at support.}$$

$$\frac{V}{jd} = \frac{25860 - 360 \times 6}{0.948 \times 25} = 1015 \text{ lbs linear inch at one-third point}$$

Diagonal tension to be taken by 6 rods

$$\frac{1217 + 1015}{2} \times 6 \times 12 \times 0.66 = 83868 \text{ lbs.}$$

$$\frac{6 \times 0.442 \times 16000}{0.7} = 60617 \text{ lbs. tensile value of 6 rods.}$$

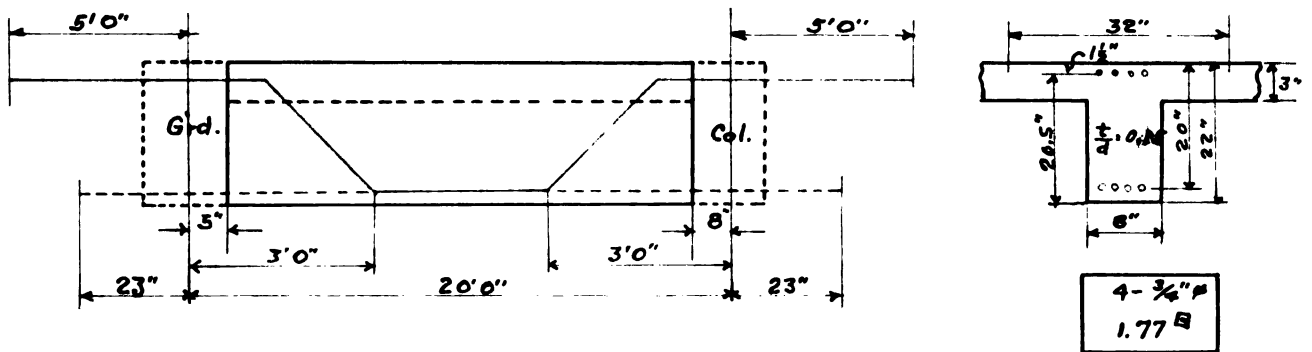
0.0131 \times 25 = 0.328", use 1/2" round stirrups bent at top

$$s = \frac{2 \times 0.196 \times 16000}{1015} = 6.1" \text{ at one-third point}$$

Length for bond,

$$\frac{623 \times 15 \times 0.375}{4 \times 60} = 25.3 \text{ inches}$$

Roof Beams



Load per lin. foot.

$$\text{Slab} = (168)(6) = 1008$$

Wall =

Stem =

Total = 1158

A + left support.

$$\frac{d'}{d} = \frac{2}{20.5} = 0.097 \quad p = 0.011 \quad p' = p$$

$L = 0.25$ $K = 0.0089$ $M = 463200$

$$f_0 = \frac{463200}{(8)(20.5)^2(0.25)} = 551^{\circ} \quad f_5 = \frac{463200}{(8)(20.5)^2(0.0089)} = 15981^{\circ}$$

$$u = \frac{16000}{(4)(2.356)(0.937)(20.5)} = 88.4^\circ$$

At right support

Same as left.

Cross Section.

$$V = \frac{(1150)(120)}{2} = 11580$$

$$M = \frac{(1158)(20)^2(12)}{12} = 463200$$

Area for shear = $\frac{11580}{105} = 110.2$ ft²

$$\text{Wgt. of stem} = \frac{(19)(150)}{144} = 158^{\text{m}} \text{ ok}$$

$$\frac{M}{bd^2} = \frac{463200}{(32)(20)^2} = 37.1$$

$$f_c = 316 \quad j' = 0.937$$

$$a_5 = \frac{463200}{(16000)(0.937)(20)} = 1.59, 4 - \frac{1}{2}'' \phi$$

Web reinforcement.

$$x_2 = \frac{2.0}{2} [1 - \sqrt{\frac{2}{3}}] = 36", \text{ bend up. } \underline{2}.$$

$$\lambda_1 = \frac{20}{2} - \frac{(40)(10)(0.937)(18)}{1158} = 4.2; 50\frac{1}{2}^\circ$$

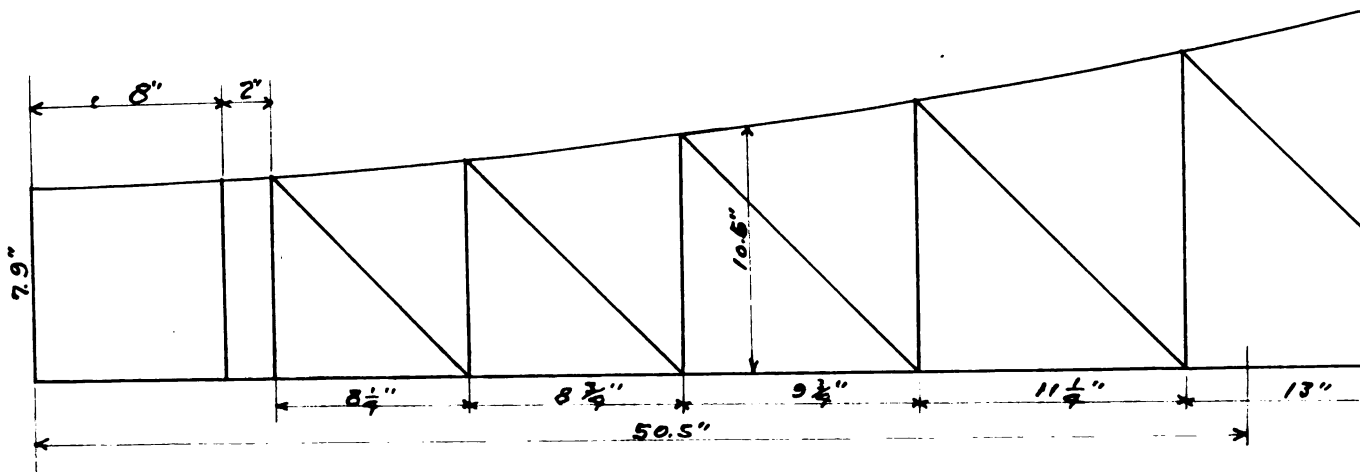
$$(0.011)(18) = 0.198, \text{ } \frac{1}{8}'' \text{ bent } \underline{OK}$$

$$S = \frac{(0.11)(2)(16000)(0.88)(30.5)(3)}{(2)(11580)} = 7.9''$$

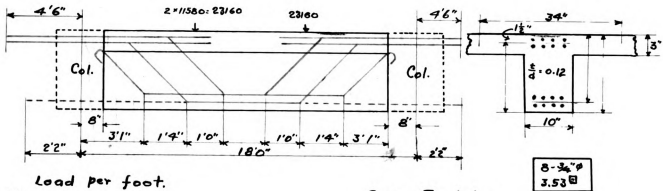
$$\text{at } \frac{1}{8}, v = \left(\frac{9}{5}\right)(7.5) = 10.5$$

$$\text{at } \frac{1}{4}, e_1(2) (7.5) = 15.0$$

$$\ell' = \frac{(551)(15)(0.875)}{(7)(80)} = 22.6'$$



Roof Girder.



Load per foot.

Slab
Wall
Stem
Total

$$\frac{(23160)(3)}{18} + 300 = 4160$$

Left Support.

$$\frac{d'}{d} = \frac{8.5}{25} = 0.1 \quad p = \frac{3.53}{(12)(25)} = 0.0119, \quad p' = 0.5$$

$$L = 0.261, \quad K = 0.0131, \quad M = 1,213,056$$

$$f_c = \frac{1,213,056}{(12)(25)^2(0.261)} = 623$$

$$f_s = \frac{1,213,056}{(12)(25)^2(0.0131)} = 12,430$$

$$u_s = \frac{25,760}{(8)(2.356)(0.948)(25)} = 57.8$$

Right same as left.

$$\frac{18}{2} \left[1 - \sqrt{\frac{(8)(0.948)(2)}{(12)(3.53)}} \right] 12 = 69.8'' \text{ bend 2}$$

$$\frac{18}{2} \left[1 - \sqrt{\frac{(8)(7)}{(12)(8)}} \right] 12 = 57.2'' \text{ bend 2}$$

$$\frac{1}{2} \times \frac{1}{3} = 3', 36'' \text{ bend down}$$

bending up will control.

$$X_u = \frac{18}{2} - \frac{(9)(10)(0.948)(25)}{4160} = 6.72' \text{ or } 80.6''$$

Cross Section

$$V = 23160 + (3)(300) = 25860$$

$$M = \frac{(4160)(18)^2(12)}{2} (0.9) = 1,213,056 \text{ inps}$$

$$\text{Area for shear} = \frac{25860}{10.5} = 248$$

$$\text{wt of stem} = \frac{(10)(22)(150)}{144} = 229 \text{ lb}$$

$$\frac{M}{bd} = \frac{1,213,056}{(39)(22)} = 73.7$$

$$f_c = 634 \quad j' = 0.948$$

$$a_s = \frac{1,213,056}{(16000)(0.948)(25)} = 3.2$$

Web Reinforcement

$$\frac{V}{jd} = \frac{25860}{(0.88)(25)} = 1217'' \text{ at support}$$

$$\frac{V}{jd} = \frac{25860 - (300)(6)}{(0.948)(25)} = 1015'' \text{ @ } \frac{1}{3} \text{ point}$$

total diag. tension taken by 6 rods.

$$\frac{1217 + 1015 - (6)(12)(\frac{7}{8})}{2} = 53568''$$

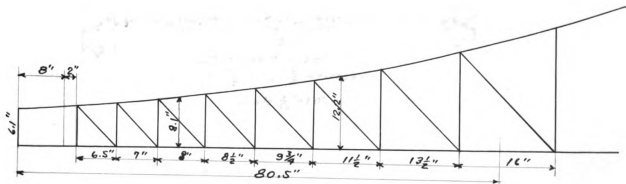
$$\frac{(6)(0.948)(16000)}{0.7} = 60.617''$$

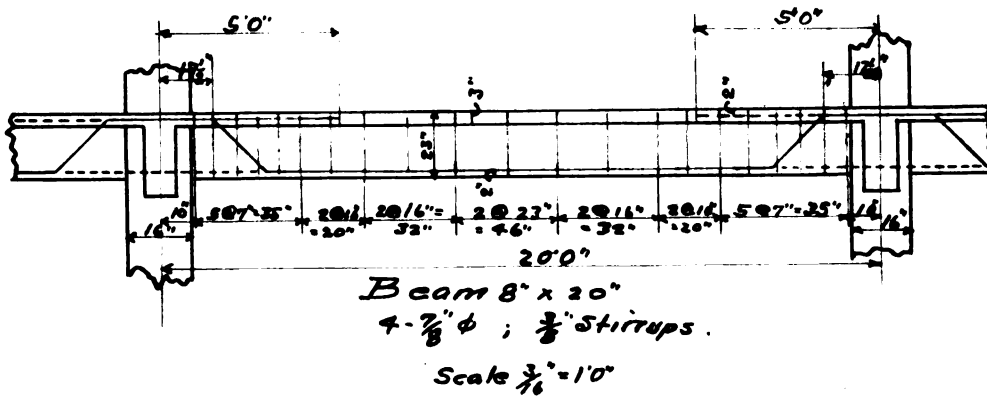
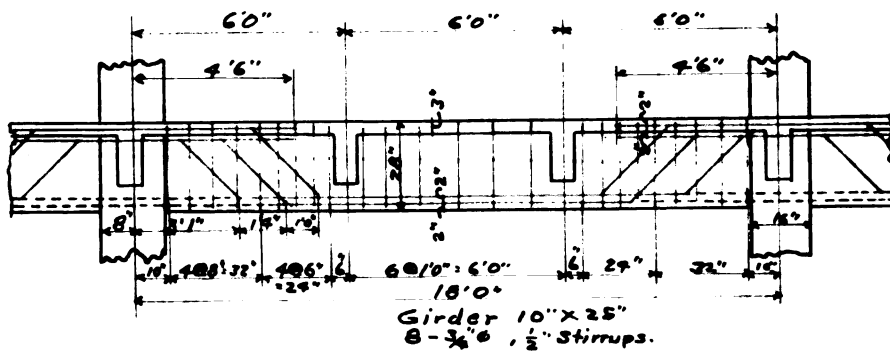
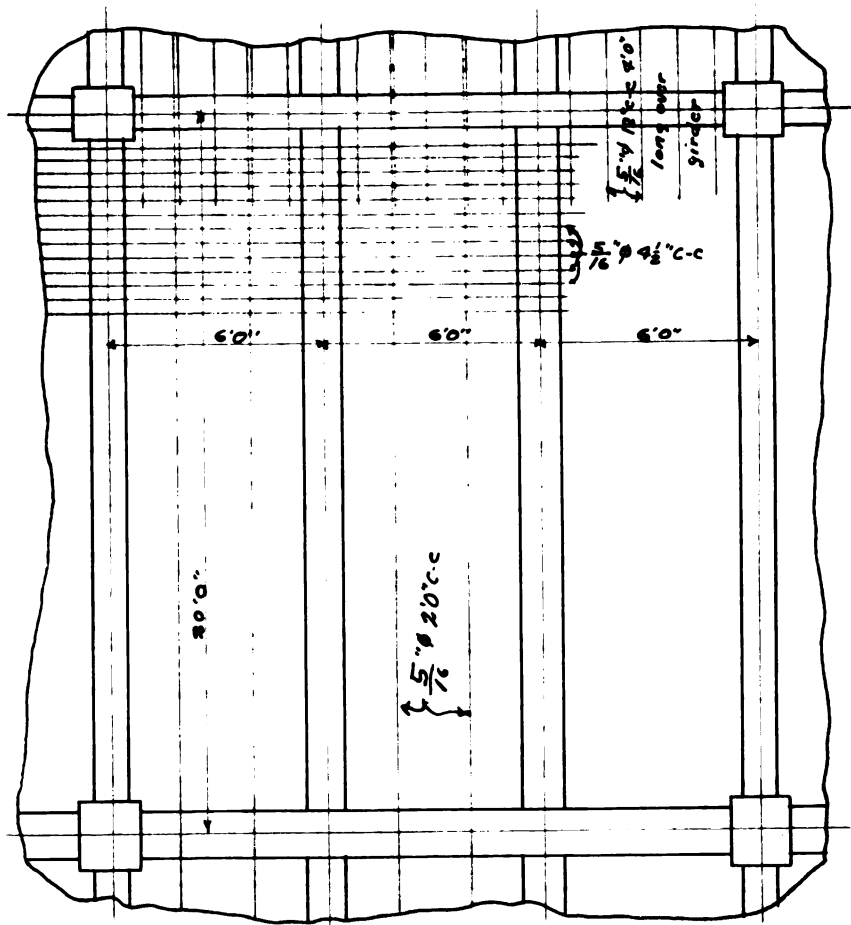
$$(0.0131)(25) = 0.328'', \text{ use } \frac{1}{2}'' \text{ stirrups.}$$

$$s = \frac{(2)(0.196)(16000)}{1015} = 6.1''$$

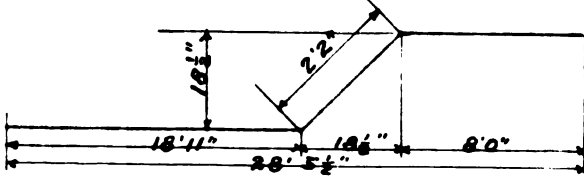
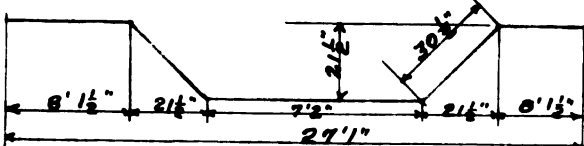
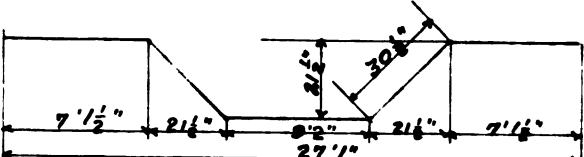
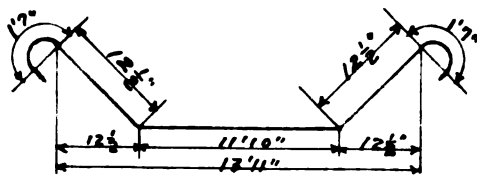
$$l = \frac{(623)(15)(0.875)}{(4)(80)} = 25.5''$$

$$\text{at } \frac{1}{8} = \frac{6}{3}(6.1) = 8.1'', \text{ at } \frac{1}{4} = (2)(6.1) = 12.2''$$

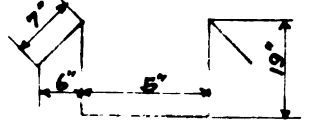
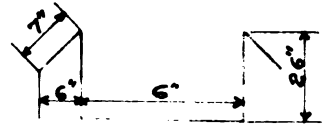




Beams & Girders

Bends	Size	Length	B or G		No. ca.	Total
			Mk	No.		
	$\frac{7}{8}$ "	29' 1"	B ₁	2	4	12
			B ₂	1	4	
	$\frac{3}{4}$ "	28' 6"	G ₁	1	2	2
	$\frac{3}{4}$ "	28' 6"	G ₁	1	2	2
	$\frac{3}{4}$ "	17' 1"	G ₁	1	2	2
	$\frac{5}{16}$ "	15' 2"	Slab	1	75	75
Straight	$\frac{3}{4}$ "	22' 4"	G ₁	1	2	2
"	$\frac{5}{16}$ "	24' 0"	Slab	1	9	9
"	$\frac{5}{16}$ "	4' 0"	Slab	1	17	17

Stirrups.

	$\frac{3}{8}$ "	4' 9"	B ₁	1	24	72
			B ₂	2	24	
	$\frac{1}{2}$ "	6' 0"	G ₁	1	23	23

Chapter 7.

This chapter could be made stronger by the introduction of some explanatory equations having to do with the finding of the effective areas of various types of columns; equations showing use of reduction percentages; equations showing computations for weights; and some steps in arriving at proportions of steel to be used. At present the reader finds very little of explanation, he then refers to Volume 1 and finds still less, of the basic work on column design. Unless the reader has access to other works, or can reason to conclusions logically; he has nothing on which to develop his design.

Design No. 10. Page 216.

Make a complete design of a typical interior column for a three story building using the floor and roof load determined for Problem 5, Page 45, and Problem 9, Page 72. Joint Committee recommendations used throughout. Height floor to floor, 13'; basement 11 feet. Concrete 1:1½:3. 810 lbs compressive strength when effectively hooped.

$$\text{Roof load } 2 \times 11580 + 2 \times 25860 = 74880 \text{ lbs.}$$

$$\text{Floor load } 2 \times 17300 + 2 \times 33650 = 111900 \text{ lbs.}$$

3rd Floor Column.

$$\text{Effective area column } \frac{74880}{860} = 133.7 \text{ sq. inches.}$$

$$nr^2 = 133.7$$

$$\text{Effective radius, } r = \sqrt{\frac{133.7 \times 7}{22}} = 6.5 "$$

Short diameter of octagonal column 13"

Use 1½" for fire protection, 13 + 3 = 16" outside-outside

$$\text{Wgt. Col. } \frac{1}{2} \times (16)^2 \times 16 \times \tan 22.5^\circ \times 15 \times 150 = 2872 \text{ lbs.}$$

Area col. 133.7 sq. in. 1 per cent steel, 1.337 sq. in.

8 - 1" round bars give 1.57 sq. in.

Place hoops of 1" round every 12"

2nd Floor column.

$$\text{Effective area } \frac{77752 + 111900}{810} = 246.8 \text{ sq. in.}$$

$$\text{Effective r. } \sqrt{\frac{246.8 \times 7}{22}} = 8.8 "$$

Short diameter 17.6" + 3" = 21" out to out

$$\text{Wgt. } 9.5 \times (21)^2 \times 16 \times 0.4142 \times 15 \times 150 = 4947 \text{ lbs.}$$

Area col. 246.8 sq. in. 1 per cent steel 2.468 sq. in.

10 - 9/16" rods give 2.48 sq. in.

Spiral hoops 1" x 17.6 = 2.6", use 2.5" spacing

$$\text{1 per cent vol. } \frac{4947}{150} \times 0.01 = 0.329 \text{ sq. in., use 5/16" rods}$$

1st Floor Column.

Effective area $\frac{194599+111900}{810} = 378.5$ sq. in.

Effective r , $\sqrt{\frac{378.5 \times 7}{22}} = 10.9"$

Short diameter, $21.8" + 3" = 24.8"$, use 25" out to out

Wgt. $0.5 \times (12.5/12) \times 16 \times 0.4142 \times 13 \times 150 = 70.2$ lbs.

Area col. 378.5, 1 per cent steel, 3.785 sq. in.

16 - 9/16" rods give 3.93 sq. in.

Spiral hoops, $\frac{1}{4} \times 21.8 = 5.6"$, space 2.5" or less.

1 per cent of vol. $\frac{70.2}{150} \times 0.01 = 0.467$ sq. in.

Use 2 - 5/16 rods at 0.48 sq. in.

Basement.

Effective area col. $\frac{513510+111900}{810} = 525.2$ sq. in.

Effective r , $\sqrt{\frac{525.2 \times 7}{22}} = 12.9"$

Short diameter, $25.8" + 3" = 28.8"$, use 29" out to out

wgt. $0.5 \times (14.5/12) \times 16 \times 0.4142 \times 11 \times 150 = 7983$ lbs.

Area 525.2 sq. in. 1 per cent steel 5.252 sq. in.

Use 16 - 11/16" rods gives 5.94 sq. in.

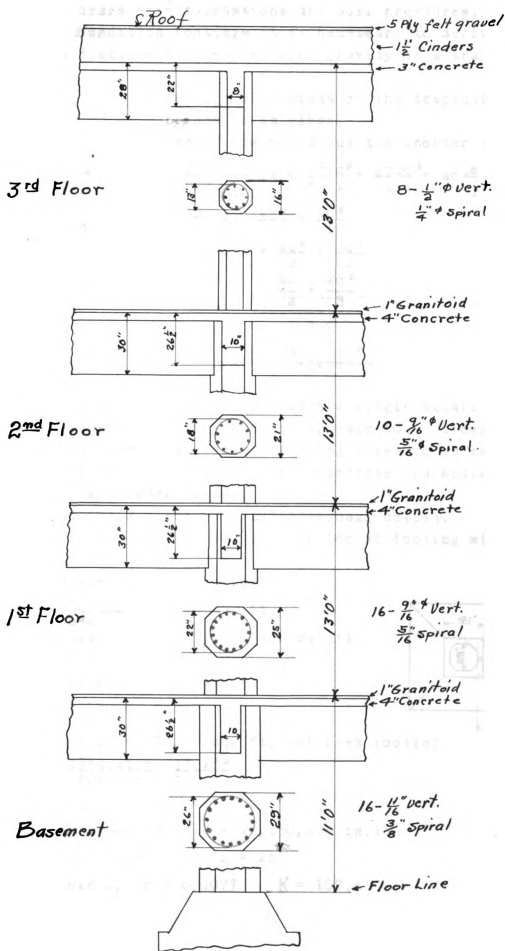
1 per cent vol. $\frac{7983}{150} \times 0.01 = 0.532$ sq. in.

Use 3/8" hoops spaced 2.25" c-c

Column Schedule

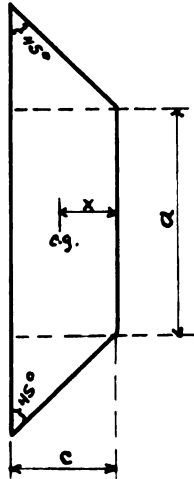
Story	Kind of Load	Amount of Load	Shape of Column	Vertical Steel (Rounds)	Spiral (Core 3" less than dia. column,
3rd	Roof	74980	16" Octagonal	8-1/2" at	1/4" at 12"
	Column	<u>2872</u>		1.57 sq. in.	
	Total	77752			
2nd	Floor	111900	21" octagonal	10-9/16" at	5/16" rods 2.5" spacing
	Column	<u>4247</u>		2.48 sq. in.	
	Total	194599		lap 27" +	
1st	Floor	111900	25" octagonal	16-9/16" at	5/16" at 2" pitch.
	Column	<u>7012</u>		3.93 sq. in.	
	Total	313510		lap 27" +	
Base	Floor	111900	29" octagonal	16-11/16" at	3/8" at 2.25" pitch
	Column	<u>7983</u>		5.94 sq. in.	
	Total	438393		lap 27" +	

Column



Chapter 8.

This chapter deals with foundations and soil pressures. For the design of foundation footings it is necessary to derive a formula for the distance to the center of gravity of a trapezoidal section.



Let the dimensions of the trapezoidal figure be as given.

Taking moments about the shorter side

$$\frac{a+c}{2} \times x = \frac{c}{3} \times \frac{c^2}{2} + \frac{c}{3} \times \frac{c^2}{2} + ac \times \frac{a}{2}$$

$$(a+c)x = \frac{2c^3}{3} + \frac{ac^2}{2}$$

$$(a+c)x = \frac{2c^3}{3} + \frac{2c^2}{2}$$

$$x = \frac{\frac{2c^3}{3} + \frac{2c^2}{2}}{a+c}$$

$$x = \frac{2ac + 4c^2}{6(a+c)}$$

Problem 11. Using 1." round bars, design a single square footing to carry a load as determined in Problem 10, page 80. Allowable soil pressure 2 tons per square foot. Working stresses as recommended by the Joint Committee for 2000 lb. concrete and medium steel. Design by cantilever method.

Total load 433,393 lbs. 29" octagonal column.

Let footing top have 6" ledge, thus top of footing will be 41" square.

Area necessary to carry column

$$\frac{433393}{2 \times 2000} = 108.35 \text{ sq. ft.}$$

Increased by 10 percent, 119.185 sq. ft.

Use a base 11' x 11' square.

Area of column 601 sq. inches

$$\frac{601}{144} = 4.17 \text{ sq. feet.}$$

(11x11) - 4.17 = 116.83 sq. ft. net area footing.

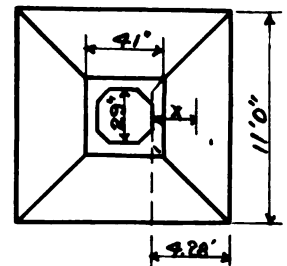
$$x = \frac{3 \times 2.43 \times 4.28 + 4 \times 4.28^2}{6(2.43 + 4.28)}$$

x = 2.6 ft.

$$M = 116.83 \times 1 \times 2000 \times 2.6 \times 12 = 14,580,384 \text{ in. lbs.}$$

$$f_s = 16000 \quad f_c = 650 \quad n = 15$$

$$\text{from table 2, } p = 0.0077 \quad K = 107.4$$



$$M = Kbd^2 \quad d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{1450000}{107.4 \times 29 \times 4}} = 34.2"$$

$$d \text{ for punching } \frac{116.83 \times 2 \times 2000}{4 \times 29 \times 105} = 38.3", \text{ use } 38.5"$$

$$a_s = pbd = 0.0077 \times 29 \times 4 \times 38.5 = 34.39 \text{ sq. in.}$$

use 5, 15/16" sq. bars per band.

The footing will be decreased by 12" at outer edge.

$$\frac{34.39}{3} = 4.29 \text{ sq. in. steel per band}$$

Shear on footing

$$d' = 38.5 - K = 40 - \frac{32.5^2}{45.5} = 16.79 \text{ inches at } 2.6' \text{ from column.}$$

$$\text{shear } \frac{110.75 \times 15 \times 2 \times 2000}{144} = 39993.0 \text{ lbs.}$$

$$\text{unit shear } \frac{39993.0}{113 \times 0.875 \times 16.8} = 23.1 \text{ lbs. sq. inch.}$$

footing needs no stirrups.

$$\text{bond } u = \frac{V}{\sum ojd} = \frac{116.83 \times 2 \times 2000}{40 \times 3.75 \times 0.875 \times 38.5} = 22.4 \text{ lb. sq. in.}$$

punching shear;

$$\frac{116.83 \times 2 \times 2000}{4 \times 29 \times 0.875 \times 40.5} = 113.6 \text{ lbs. sq. in.}$$

weight of concrete footing,

$$11 \times 11 \times 150 = 19,800$$

$$(132 + 11.7 \times \sqrt{1544.4}) \frac{32.5}{36} 150 = 24,781.25$$

$$\text{weight of column and floors } \underline{433,323}$$

$$\text{total load on earth under footing } 477,974.25 \text{ lbs.}$$

$$\text{allowable load on 11x11 earth } 520,000 \text{ lbs.}$$

Bending moment by flat slab method.

$$r_1 = \frac{11 + 1.41 \times 11}{4} = 6.63$$

$$r_o = \frac{2.43 + 1.41 \times 2.42}{4} = 1.46$$

$$\frac{r_1}{r_o} = \frac{6.63}{1.46} = 4.5$$

From Table 1 on page 62, $\alpha_1 = 11.77$

$$M = 2 \times 2000 \times 1.46^2 \times 11.77 = 100,358.73 \text{ in.pds. per in of perimeter}$$

$$M = 100,358.73 \times 2 \times 3.142 \times 1.51 \times 12 = 11,427,105.6 \text{ in pds. total } M$$

$$\text{By cantilever method } M = 14,580,384 \text{ in lbs.}$$

the 1990s, the number of people in the world who are under 15 years of age is expected to increase from 1.1 billion to 1.5 billion. The number of people aged 65 and over is expected to increase from 200 million to 400 million. The number of people aged 15 and over is expected to increase from 3.5 billion to 4.5 billion. The number of people aged 15 and over is expected to increase from 3.5 billion to 4.5 billion. The number of people aged 15 and over is expected to increase from 3.5 billion to 4.5 billion.

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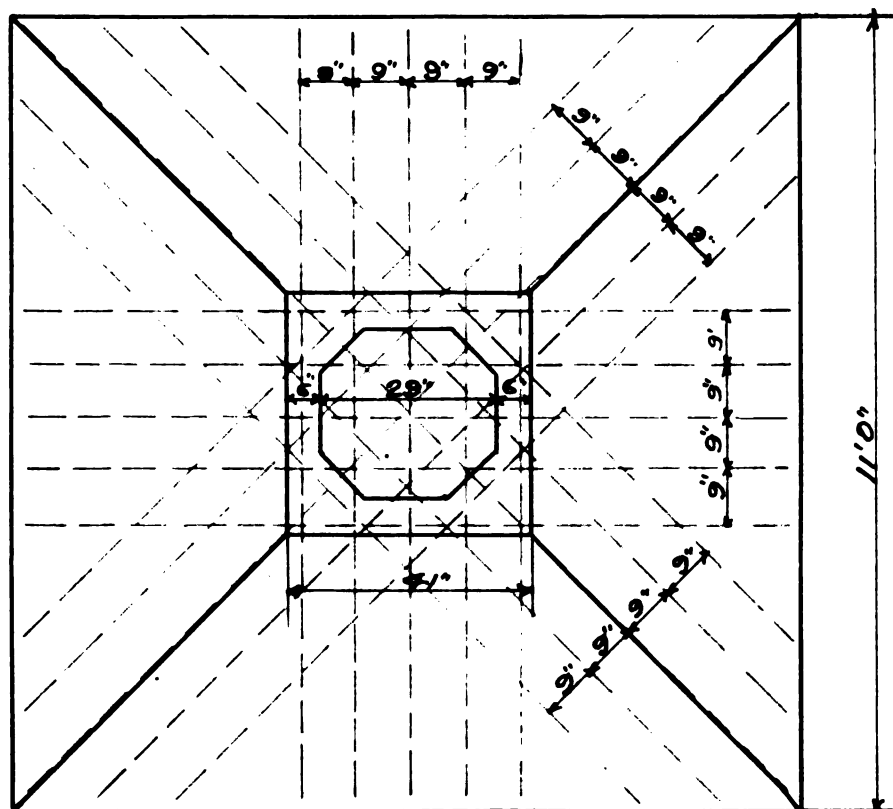
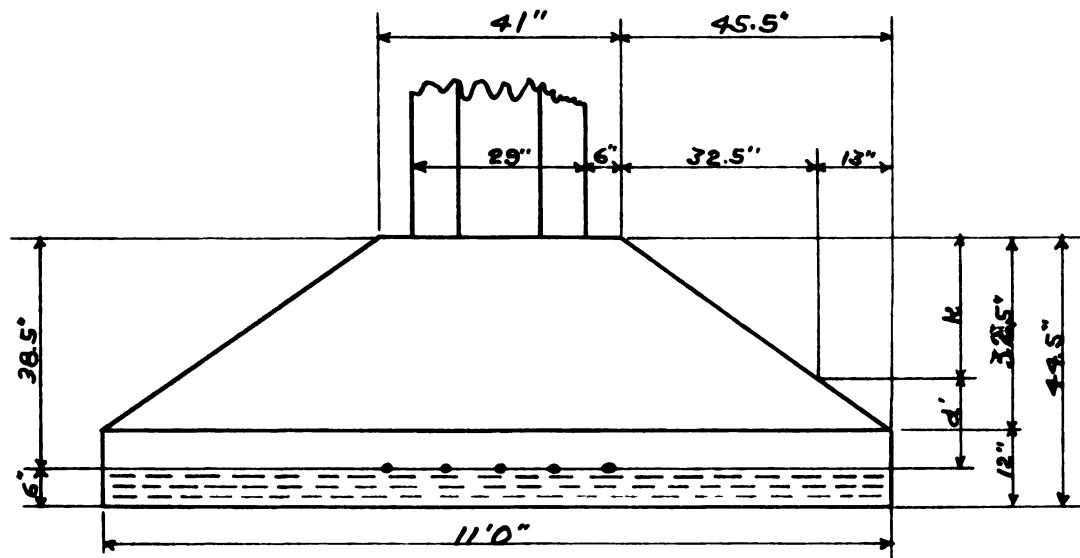
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the 1990s, the number of people in the world who are undernourished has declined from 1.1 billion to 800 million. The number of people who are malnourished has declined from 1.5 billion to 1 billion. The number of people who are obese has increased from 100 million to 300 million. The number of people who are overweight has increased from 100 million to 300 million. The number of people who are obese and overweight has increased from 100 million to 300 million. The number of people who are obese and overweight has increased from 100 million to 300 million.

Figure 1. The effect of the concentration of the *Agrobacterium* suspension on the transformation efficiency of *Agrobacterium* strains. The *Agrobacterium* strains were grown in the YEA medium for 24 h and then adjusted to the optical density of 0.5. The *Agrobacterium* strains were then mixed with the *Agrobacterium* suspension of different concentrations (10⁶, 10⁷, 10⁸, 10⁹, 10¹⁰, 10¹¹, 10¹², 10¹³, 10¹⁴, 10¹⁵, 10¹⁶, 10¹⁷, 10¹⁸, 10¹⁹, 10²⁰, 10²¹, 10²², 10²³, 10²⁴, 10²⁵, 10²⁶, 10²⁷, 10²⁸, 10²⁹, 10³⁰, 10³¹, 10³², 10³³, 10³⁴, 10³⁵, 10³⁶, 10³⁷, 10³⁸, 10³⁹, 10⁴⁰, 10⁴¹, 10⁴², 10⁴³, 10⁴⁴, 10⁴⁵, 10⁴⁶, 10⁴⁷, 10⁴⁸, 10⁴⁹, 10⁵⁰, 10⁵¹, 10⁵², 10⁵³, 10⁵⁴, 10⁵⁵, 10⁵⁶, 10⁵⁷, 10⁵⁸, 10⁵⁹, 10⁶⁰, 10⁶¹, 10⁶², 10⁶³, 10⁶⁴, 10⁶⁵, 10⁶⁶, 10⁶⁷, 10⁶⁸, 10⁶⁹, 10⁷⁰, 10⁷¹, 10⁷², 10⁷³, 10⁷⁴, 10⁷⁵, 10⁷⁶, 10⁷⁷, 10⁷⁸, 10⁷⁹, 10⁸⁰, 10⁸¹, 10⁸², 10⁸³, 10⁸⁴, 10⁸⁵, 10⁸⁶, 10⁸⁷, 10⁸⁸, 10⁸⁹, 10⁹⁰, 10⁹¹, 10⁹², 10⁹³, 10⁹⁴, 10⁹⁵, 10⁹⁶, 10⁹⁷, 10⁹⁸, 10⁹⁹, 10¹⁰⁰, 10¹⁰¹, 10¹⁰², 10¹⁰³, 10¹⁰⁴, 10¹⁰⁵, 10¹⁰⁶, 10¹⁰⁷, 10¹⁰⁸, 10¹⁰⁹, 10¹¹⁰, 10¹¹¹, 10¹¹², 10¹¹³, 10¹¹⁴, 10¹¹⁵, 10¹¹⁶, 10¹¹⁷, 10¹¹⁸, 10¹¹⁹, 10¹²⁰, 10¹²¹, 10¹²², 10¹²³, 10¹²⁴, 10¹²⁵, 10¹²⁶, 10¹²⁷, 10¹²⁸, 10¹²⁹, 10¹³⁰, 10¹³¹, 10¹³², 10¹³³, 10¹³⁴, 10¹³⁵, 10¹³⁶, 10¹³⁷, 10¹³⁸, 10¹³⁹, 10¹⁴⁰, 10¹⁴¹, 10¹⁴², 10¹⁴³, 10¹⁴⁴, 10¹⁴⁵, 10¹⁴⁶, 10¹⁴⁷, 10¹⁴⁸, 10¹⁴⁹, 10¹⁵⁰, 10¹⁵¹, 10¹⁵², 10¹⁵³, 10¹⁵⁴, 10¹⁵⁵, 10¹⁵⁶, 10¹⁵⁷, 10¹⁵⁸, 10¹⁵⁹, 10¹⁶⁰, 10¹⁶¹, 10¹⁶², 10¹⁶³, 10¹⁶⁴, 10¹⁶⁵, 10¹⁶⁶, 10¹⁶⁷, 10¹⁶⁸, 10¹⁶⁹, 10¹⁷⁰, 10¹⁷¹, 10¹⁷², 10¹⁷³, 10¹⁷⁴, 10¹⁷⁵, 10¹⁷⁶, 10¹⁷⁷, 10¹⁷⁸, 10¹⁷⁹, 10¹⁸⁰, 10¹⁸¹, 10¹⁸², 10¹⁸³, 10¹⁸⁴, 10¹⁸⁵, 10¹⁸⁶, 10¹⁸⁷, 10¹⁸⁸, 10¹⁸⁹, 10¹⁹⁰, 10¹⁹¹, 10¹⁹², 10¹⁹³, 10¹⁹⁴, 10¹⁹⁵, 10¹⁹⁶, 10¹⁹⁷, 10¹⁹⁸, 10¹⁹⁹, 10²⁰⁰, 10²⁰¹, 10²⁰², 10²⁰³, 10²⁰⁴, 10²⁰⁵, 10²⁰⁶, 10²⁰⁷, 10²⁰⁸, 10²⁰⁹, 10²¹⁰, 10²¹¹, 10²¹², 10²¹³, 10²¹⁴, 10²¹⁵, 10²¹⁶, 10²¹⁷, 10²¹⁸, 10²¹⁹, 10²²⁰, 10²²¹, 10²²², 10²²³, 10²²⁴, 10²²⁵, 10²²⁶, 10²²⁷, 10²²⁸, 10²²⁹, 10²³⁰, 10²³¹, 10²³², 10²³³, 10²³⁴, 10²³⁵, 10²³⁶, 10²³⁷, 10²³⁸, 10²³⁹, 10²⁴⁰, 10²⁴¹, 10²⁴², 10²⁴³, 10²⁴⁴, 10²⁴⁵, 10²⁴⁶, 10²⁴⁷, 10²⁴⁸, 10²⁴⁹, 10²⁵⁰, 10²⁵¹, 10²⁵², 10²⁵³, 10²⁵⁴, 10²⁵⁵, 10²⁵⁶, 10²⁵⁷, 10²⁵⁸, 10²⁵⁹, 10²⁶⁰, 10²⁶¹, 10²⁶², 10²⁶³, 10²⁶⁴, 10²⁶⁵, 10²⁶⁶, 10²⁶⁷, 10²⁶⁸, 10²⁶⁹, 10²⁷⁰, 10²⁷¹, 10²⁷², 10²⁷³, 10²⁷⁴, 10²⁷⁵, 10²⁷⁶, 10²⁷⁷, 10²⁷⁸, 10²⁷⁹, 10²⁸⁰, 10²⁸¹, 10²⁸², 10²⁸³, 10²⁸⁴, 10²⁸⁵, 10²⁸⁶, 10²⁸⁷, 10²⁸⁸, 10²⁸⁹, 10²⁹⁰, 10²⁹¹, 10²⁹², 10²⁹³, 10²⁹⁴, 10²⁹⁵, 10²⁹⁶, 10²⁹⁷, 10²⁹⁸, 10²⁹⁹, 10³⁰⁰, 10³⁰¹, 10³⁰², 10³⁰³, 10³⁰⁴, 10³⁰⁵, 10³⁰⁶, 10³⁰⁷, 10³⁰⁸, 10³⁰⁹, 10³¹⁰, 10³¹¹, 10³¹², 10³¹³, 10³¹⁴, 10³¹⁵, 10³¹⁶, 10³¹⁷, 10³¹⁸, 10³¹⁹, 10³²⁰, 10³²¹, 10³²², 10³²³, 10³²⁴, 10³²⁵, 10³²⁶, 10³²⁷, 10³²⁸, 10³²⁹, 10³³⁰, 10³³¹, 10³³², 10³³³, 10³³⁴, 10³³⁵, 10³³⁶, 10³³⁷, 10³³⁸, 10³³⁹, 10³⁴⁰, 10³⁴¹, 10³⁴², 10³⁴³, 10³⁴⁴, 10³⁴⁵, 10

Figure 1. The effect of the concentration of the *Agrobacterium* suspension on the transformation efficiency of *Agrobacterium* strains.

Figure 1. The effect of the number of trials on the number of correct responses. The number of correct responses was significantly higher than the number of incorrect responses in all cases. Error bars represent the standard error of the mean.



Rod List.

- 10- $\frac{15}{16}$ " 10'8" long
- 2- " 14'6" "
- 7- " 13'0" "
- 4- " 11'6" "

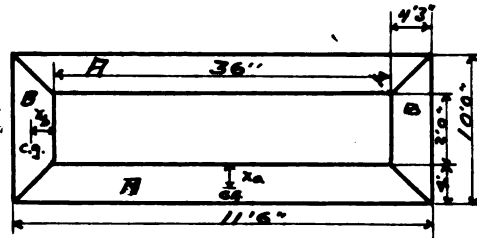
Scale $\frac{3}{8}$ " = 1'0"

Design No. 124

Design a wall column footing to sustain a rectangular column 24 inches by 36 inches, carrying 400,000 pounds. Allowable pressure on soil 2 tons per square foot. Reinforce with 1" round deformed rods. Joint Committee stresses for 2000 pound concrete, with mild steel. Volume 2, page 280.

footing for column $\frac{400000}{2 \times 2000} = 100$ sq. ft. area
 2×2000

add 10 per cent, area used 110 sq. ft.
 rectangular footing 10' x 11.5'



$$\text{Area } A = \left(\frac{11.5}{2} + 2 \right) 4 = 29.0 \text{ sq. ft.}$$

$$\text{Area } B = \left(\frac{10}{2} + 2 \right) 4.25 = 28.5 \text{ sq. ft.}$$

$$x_a = \frac{\frac{3}{2} \times \frac{4}{2} + 2/3(4)^2}{3 + 4} = 2.38 \text{ ft.}$$

$$x_b = \frac{\frac{2}{2} \times \frac{4.25}{2} + 2/3(4.25)^2}{2 + 4.25} = 2.61 \text{ ft.}$$

$$M_A = (29.0)(2)(2000)(2.38)(12) = 3,312,960 \text{ in. lbs.}$$

$$M_B = (28.5)(2)(2000)(2.61)(12) = 3,194,640 \text{ in. lbs.}$$

$$d_A = \sqrt{\frac{3312960}{107.4 \times 36}} = 29.27''$$

$$d_B = \sqrt{\frac{3194640}{107.4 \times 24}} = 35.2''$$

depth for punching shear.

$$d_A = \frac{(22)(2)(2000)}{(36)(105)} = 30.7''$$

$$d_B = \frac{(28.5)(2)(2000)}{(24)(105)} = 40.5''$$

punching shear on d_B will rule.

place rods lengthwise at 40.5", crosswise rods on top of longitudinals, 2.5" concrete below rods. total depth 43".

Longitudinal rods;

$$a_s = (0.0077)(24)(40.5) = 7.45 \text{ sq. in. steel}$$

use 11 - 10/16" round rods, 7.25 sq. in. area.

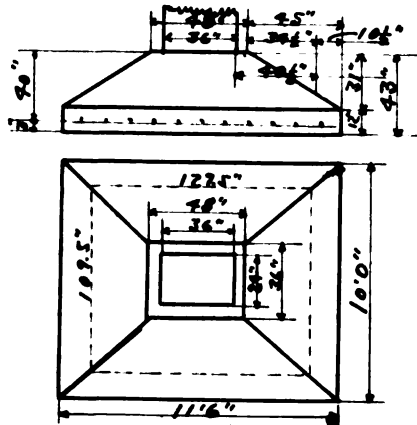
$$K = \frac{34.5 \times 51}{40} = 28.76''$$

$$a' = 40.5 - 28.76 = 11.74''$$

$$v = \frac{(120)(10.5)(2)(2000)}{144} = 33000 \text{ lbs.}$$

$$v = \frac{33000}{(107.5)(0.875)(16.74)} = 21.3 \text{ lbs. per sq. inch.}$$

$$u = \frac{(29)(2)(2000)}{(11)(2.945)(0.875)(40.5)} = 101.0 \text{ lbs. per sq. in.}$$



Crosswise reinforcement.

$$K = \frac{3124640}{(36)(39.5)^2} = 56.8 \quad p = 0.0032$$

$$a_s = (0.0032)(36)(39.5) = 4.546 \text{ sq. in.}$$

use 10 - 3/4 in round bars, area 4.62 sq. in.

$$v = \frac{(120)(10.5)(2)(2000)}{144} = 33,000 \text{ lbs.}$$

$$v = \frac{33,000}{(127.5)(0.875)(16.74)} = 22.3 \text{ lbs. sq. in.}$$

$$u = \frac{(25.2)(2)(2000)}{(10)(3)(0.875)(39.5)} = 38.3 \text{ lbs. sq. in.}$$

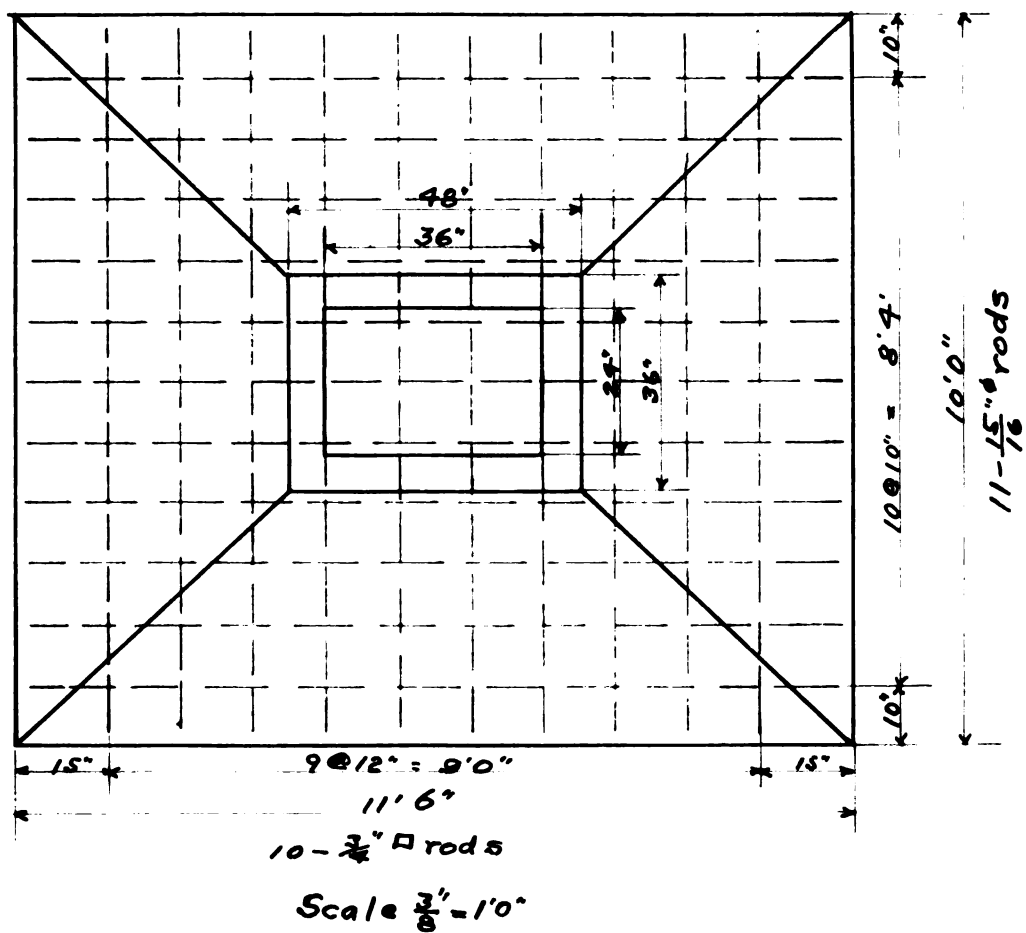
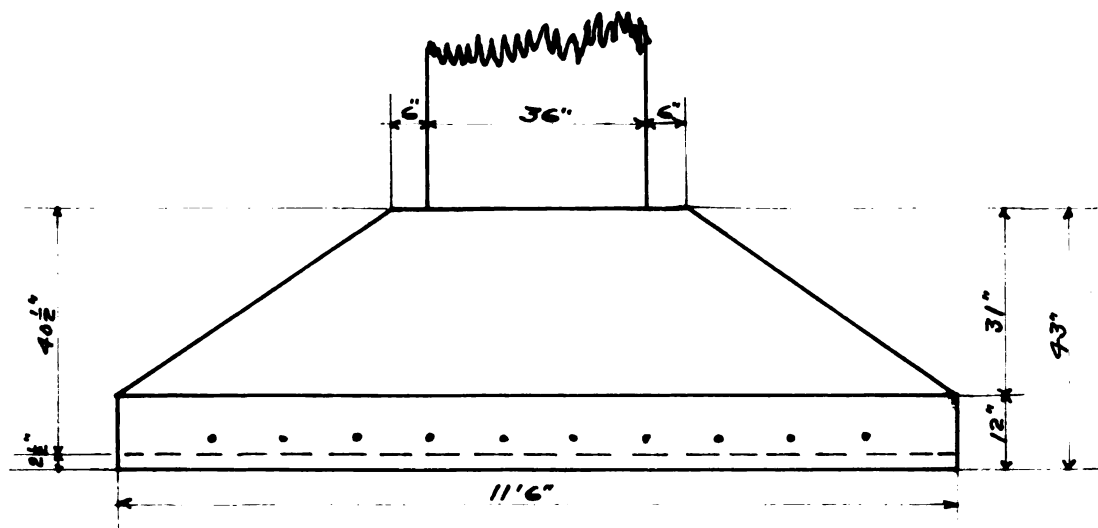
wgt. of column on base; $(11.5)(10)(150) = 17,250$

$$\frac{1}{33} [115 + 12 + \sqrt{(12)(15)}] 150 = 21,195$$

column load 400,000

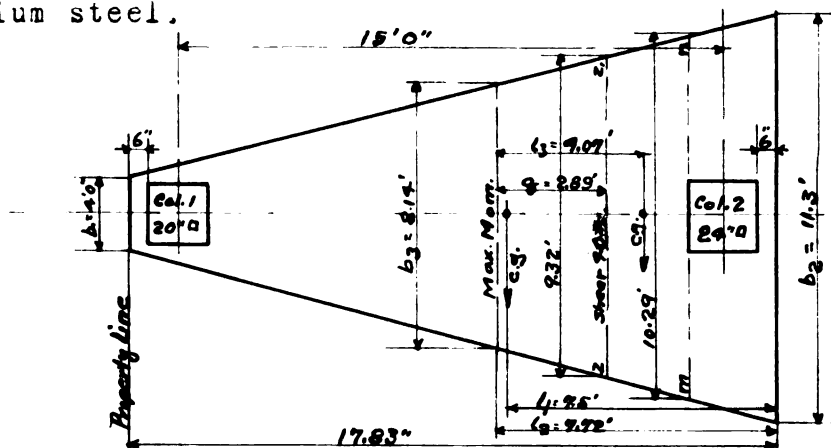
total load on earth 438,445

soil will carry $(11.5)(10)(4000) = 460,000$



Design number 18.

Consider load of 200,000 lbs on column No. 1, 20" x 20", and a load of 300,000 lbs on column No. 2, 24" x 24". Columns 15' c - c. Allowable soil pressure, 2 tons per sq. ft. Working stresses as recommended by the Joint Committee for 2000 lbs. concrete, medium steel.



Load on column 1,	200,000
Load on column 2,	300,000
Total,	500,000

$$\frac{500,000}{4000} = 125 \text{ sq. ft. area}$$

add 10 percent, area to use 138.8 sq. ft.

$$\text{area footing} = \frac{b_1 + b_2}{2} (17.83) = 138.8$$

$$b_1 + b_2 = 15.81 \text{ ft.} \quad (1)$$

$$l_1 - l_2 = \frac{200,000}{200,000} (12)$$

$$l_1 = 7.72 \text{ ft.}$$

c. g. of trapezoid

$$l_1 = 7.72 = \frac{17.83 \times b_2 + \frac{b_1 - b_2}{3} l_2}{b_2 + b_1} \quad (2)$$

solve (1) and (2) for b_1 and b_2

$$22.83b_2 + 22.83b_1 = 17.83b_2 + 20.00b_1$$

$$4.07b_2 - 18.17b_1 = 0$$

$$b_2 + b_1 = 15.81$$

$$4.07b_2 + 4.07b_1 = 71.4877$$

$$17.83b_1 = 71.4877$$

$$b_1 = 4.01 \text{ ft.}$$

$$b_1 + b_2 = 15.81$$

$$b_2 = 11.80 \text{ ft.}$$

$$[11.81b_2 - \frac{11.81 - 4.01}{2} (17.83)] 4000 = 200,000$$

$$(11.81b_2 - 0,204 \times 17.83) 4000 = 200,000$$

$$11.81b_2 - 0,204 \times 17.83 = 75$$

$$l_2 = 7.72$$

$$b_2 - b_1 = 11.8 - 4.01 = 7.78 \text{ ft.}$$

$$\frac{c_3 - c_1}{7.28} = \frac{17.58 - 7.72}{17.58}$$

$$c_3 - c_1 = 4.18$$

$$c_3 = 4.18 + 4.01 = 8.14 \text{ ft.}$$

$$l_3 = \frac{1}{2} \times \frac{c_3 + 2c_2}{c_3 + c_2} = \frac{7.72 \times 8.14 + 22.46}{8.14 + 11.8} = 4.07 \text{ ft.}$$

$$\text{Max. } M = 300000(7.72 - 1.8 - 4.07)12 = 7,740,000 \text{ in. lbs.}$$

Mon. for 1" along line of max. moment,

$$\frac{7,740,000}{(8.14)(12)} = 78,288 \text{ in. lbs.}$$

From Table 2, $K = 107.4$, $p = 0.0077$

$$a = \sqrt{\frac{78288}{107.4}} = 27.16", \text{ use } 27.5"$$

$$a_s = (0.0077)(12)(8.14)(27.16) = 3.01 \text{ sq. in. total}$$

use 17 - 1 1/4" round rods, 3.02 sq. in.

Max. shear 300000 lbs. from col. 2,

no. of rods required for bond if $u = 180$

$$\text{no. rods} = \frac{300000}{(8.98)(0.874)(27.5)(180)} = 18.7$$

bond controls, use 20 rods, 1 1/4" dia.

Transverse reinforcing, column 2.

Distributing beam, 8' 6".

$$M = \left(\frac{300000}{2} \right) \left(\frac{11.8 - 2}{11.8} \right) \left(\frac{4.69}{2} \right) (12) = 2,481,700 \text{ in. lbs.}$$

$$a = \sqrt{\frac{2481700}{(107.4)(8.8)(12)}} = 27.5"$$

$$K = \frac{M}{bd^2} = \frac{2481700}{(42)(27.5)(27.5)} = 106.0$$

from Table 1, $p = 0.0077$

$$a_s = (0.0077)(42)(27.5) = 9.01 \text{ sq. in.}$$

use 18 - 7/8" round rods, 9.02 sq. in.

Transverse reinforcing, column 1,

$$M = \left(\frac{200000}{2} \right) \left(\frac{2.01 - 1.67}{2.01} \right) \left(\frac{1.11}{2} \right) (12) = 408,200 \text{ in. lbs.}$$

$$a = \sqrt{\frac{408200}{(107.4)(8.8)}} = 10.2 \text{ in.}$$

$$p = \frac{408200}{(8.8)(27.5)} = 0.581$$

use 2, 1/2" round rods.

Bond stress on transverse reinforcing.

Column 2.

$$\text{Max. } S. = \frac{(200000)}{2} \left(\frac{11.8}{11.8} - 2 \right) = 120000 \text{ lbs.}$$

if u is taken at 150

$$\text{number rods required} = \frac{120000}{(1.874)(0.881)(42)(150)} = 16.1$$

use 17 - 5/8" rods instead of 16 (See page 28)

Column 1.

$$\text{Max. } S. = \frac{(200000)}{2} \left(\frac{4.0}{4.0} - 1.87 \right) = 50000 \text{ lbs.}$$

if u = 150

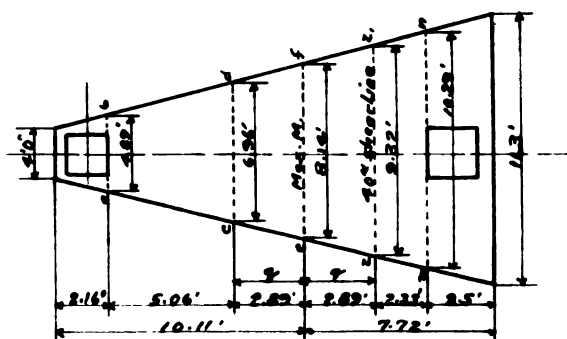
$$\text{number rods needed} = \frac{50000}{(1.871)(0.88)(36)(150)} = 7.2$$

use 8 - 1/2" rods instead of 7, (See page 28)

Place 7/8" rods at 24" centers between end beams.

Stirrups longitudinal in beam, Column 2.

At section where shear is 40 lbs.



$$(8.14)(9)(4000) = (27.8)(8.14)(12)(40)(0.875)$$

$$c = 2.88 \text{ ft. approx.}$$

$$m = 10.28 \text{ ft.}$$

$$m_1 = 8.82 \text{ ft.}$$

at m the unit shear is

$$\frac{200000}{(10.28)(27.8)(12)(0.875)} = 80.8 \text{ lb. sq. in.}$$

total shear to be taken by stirrups.

$$\frac{(80.8 - 24)}{66.8} \left(\frac{10.28}{2} \right) \left(\frac{10.28 + 8.82}{2} \right) (2.88)(144) = 48488 \text{ lbs.}$$

using 1/2" rods, number stirrups needed.

$$\frac{48488}{(18000)(0.118)} = 23.8, \text{ use 14}$$

stirrups for column 1, slab.

$$a = 4.88 \text{ ft.}$$

$$a_1 = 6.88 \text{ ft.}$$

at a the shear is

$$\frac{200000 - (4.88)(2.18)(4000)}{(4.88)(27.8)(12)(0.878)} = 111.8 \text{ lbs. sq. in.}$$

total shear to be taken by stirrups,

$$\left(\frac{111.8 - 40}{111.8}\right) \left(\frac{111.8}{2}\right) \left(\frac{4.88 + 6.86}{2}\right) (2.06)(144) = 18452 \text{ lbs.}$$

using 1/2" rods, number needed.

$$\frac{18452}{(16000)(0.196)} = 49.4, \text{ use } 50$$

negative moment at supports,

Column 2.

$$f = MV = \frac{(11.8)(1.8)(4000)(12)(14.75)(9)}{(10.68)(12)(27.8)^3} = 40.2 \text{ lbs. sq. in.}$$

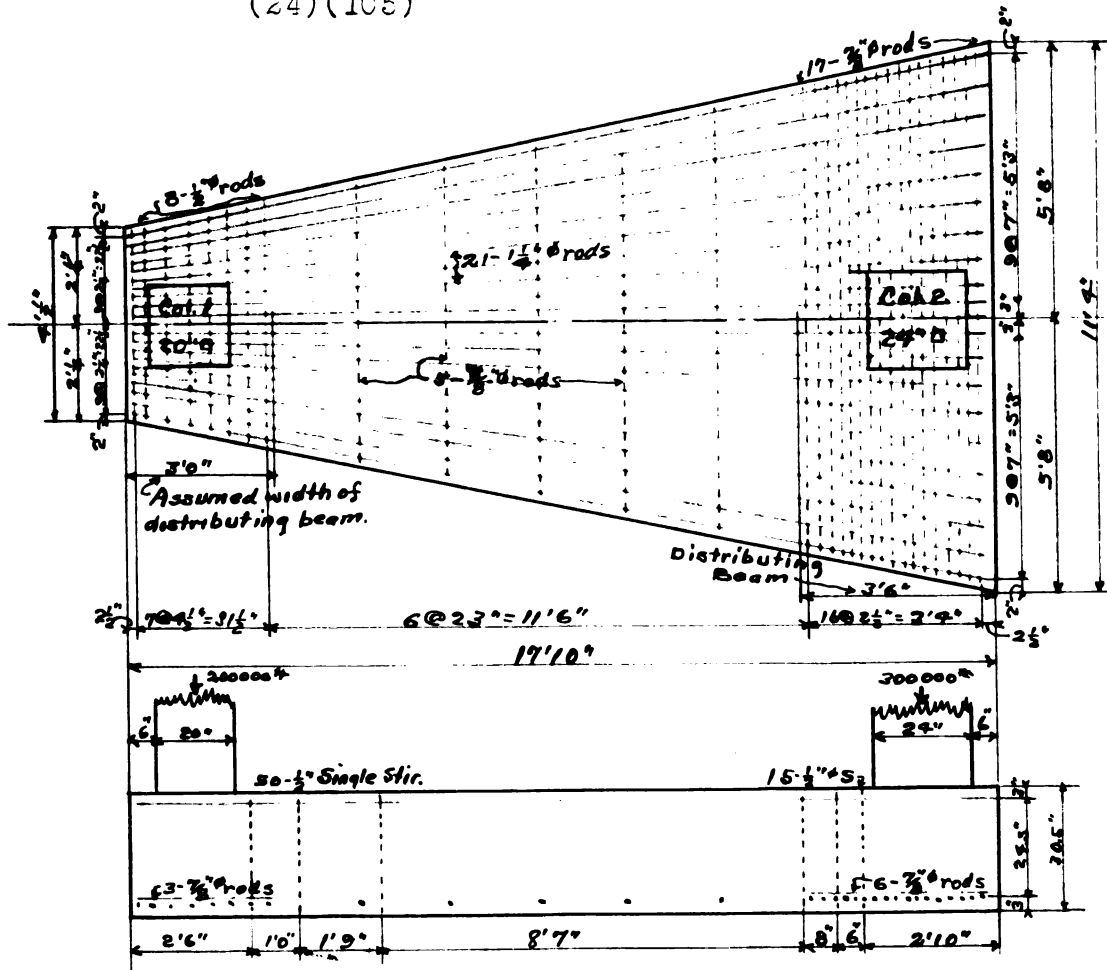
this is less than tensile strength of concrete.

place 6, 7/8" rods, 4' long in bottom at column 2, round rods.

place 3, 7/8" rods, 4' long under column 1. round rods.

depth for punching shear at column 1.

$$a = \frac{(3.825)(2)(2000)}{(24)(105)} = 3.8", \text{ safe.}$$



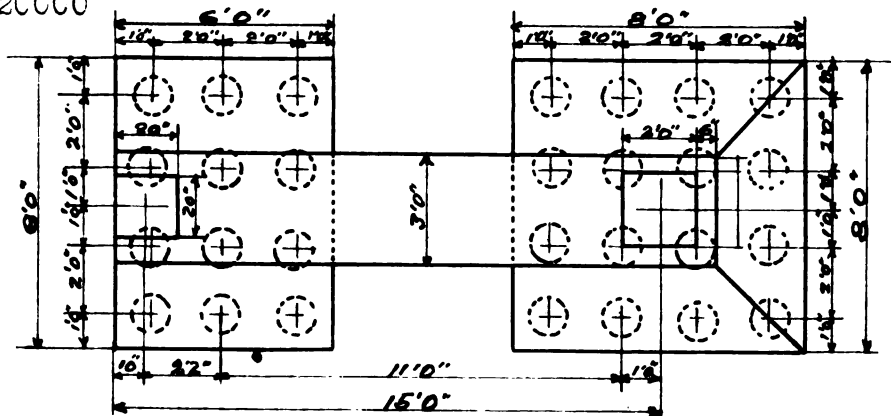
Scale 1/4" = 1'0"

Design No. 14.

Cantilever foundation. 200,000 lbs. on column No. 1, 20" x 20"; 300,000 lbs. on column No. 2, 24" x 24". 15 ft. c-c columns. Piles under foundation. Allowable load 20000 lbs. per pile. Working stresses as recommended by Joint Committee, for 2000 lb. concrete, medium steel.

$$\frac{200000}{20000} = 10, \text{ approximate number piles under wall footing.}$$

$$\frac{300000}{20000} = 15, \text{ approximate number piles under column footing.}$$



$$\frac{(200000)(2.83)}{11.83} = 49,073 \text{ lbs, uplift due dead load at outside col.}$$

Total load on wall footing,

$$\frac{(200000)(14.17)}{11.83} = 280,182 \text{ lbs.}$$

moment about outer edge of wall column

$$\frac{(280182)(3.67) - (200000)(0.61)}{12} = 6268.3 \text{ lbs. uplift on interior column}$$

Maximum moment on strap comes at

$$\frac{(200000)(7.8)}{280182} = 5.997, \text{ 6' from property line.}$$

$$M = [(200000)(5.17) - \frac{(280182)}{7.8}(6)(3)]12 = 5,204,184 \text{ in. pds.}$$

assume t at 18

$$d = \sqrt{\frac{5204184}{(107.4)(36)}} = 38.68", \text{ use 37"} "$$

Shear at inside of wall column.

$$v = \frac{(200000) - \frac{(280182)}{7.8}(1.67)}{(37)(0.875)(36)} = 128.8 \text{ lbs. allowable 130 lbs. with stirrups.}$$

increase depth to 38", total depth 41".

approximate point where shear becomes zero,

$$(2)(9)\left(\frac{280182}{(7.8)(37)}\right) = (36)(36)(40)(0.875)$$

$$x = 1.47'$$

7.6 - 1.47 = 6.03' inside inner column to point zero shear
use 1/2" stirrups, $\frac{1}{2}$ looped.

$$s = \frac{(2)(8)(0.125)(13000)(0.875)(39)}{(2)(144804)} = 8.8", \text{ use } 9"$$

unit shear at inner edge of wall footing

$$\frac{-(62865)}{(36)(0.875)(39)} = 51 \text{ lbs. sq. ft.}$$

place 2 stirrups just outside edge of footing.

Horizontal rods.

$$k = \frac{8204154}{(36)(39)^2} = 95.0, \quad p = 0.0068$$

$$a_s = (0.0068)(39)(36) = 9.55 \text{ sq. in.}$$

use 14 - 15/16" round rods, deformed

$$u = \frac{144804}{(14)(2.945)(0.88)(39)} = 101.8 \text{ lbs. sq. in.}$$

allowance for bond 20 diameters, 26.5"

bend rods down at inner end.

Stop alternate rods at 12 feet from wall.

Interior footing.

$$M = (200000)(2)(12)(12) = 5,760,000 \text{ in lbs.}$$

$$d = \sqrt{\frac{5760000}{(107.4)(36)}} = 38.8", \text{ beam is } 39" \text{ so O.K.}$$

no web reinforcement needed

$$k = \frac{5760000}{(24)(4)(39)^2} = 38.4, \quad p = 0.0027$$

$$a_s = (0.0027)(24)(4)(39) = 10.11 \text{ sq. in.}$$

place steel in two directions.

$$\frac{10.11}{4} = 2.53 \text{ sq. in. in each band.}$$

use 7 - 11/16" round rods each band.

$$\text{no. rods for bond } \frac{(12)(2.53)}{(4)(16)(2.15)(0.915)(39)} = 5+, \text{ use as above.}$$

Wall column.

$$M = 80000 \times 15 = 1,200,000 \text{ in lbs. on edge of strap.}$$

$$k = \frac{1200000}{(72)(39)^2} = 8.5, \quad p = 0.0007$$

$$a_s = (0.0007)(72)(39) = 1.97 \text{ sq. in.}$$

use 6 - 2/16" round rods.

$$u = \frac{60000}{(12)(1.767)(0.931)(39)} = 77.8 \text{ lbs. sq. in.}$$

total load, columns, strap and footings, 348,888 lbs.

$$\frac{348888}{20000} = 29+, \text{ use } 25 \text{ piles.}$$

place 16 under inner column, 12 under wall column.

Design 1c.

Design an 8 inch casement wall, 10 ft. high, to extend between wall columns, 16" x 24", which are spaced 16 ft. on centers. Consider that the wall is to sustain earth level with the top of the wall. Reinforced vertically for earth pressure. The wall is to have no footing of its own but will rest on the wall column footings. Assume earth pressure as due to a fluid weighting 80 lbs. per cubic foot.

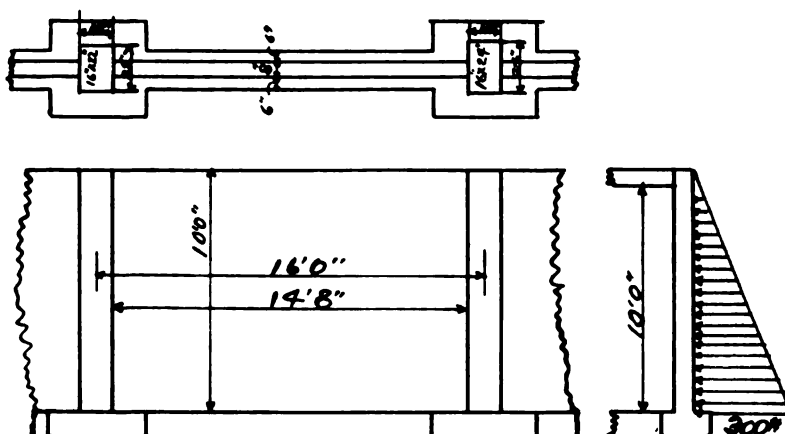
Working stresses as given by the joint committee.

Wall designed as a cantilever supported by basement floor.

Pressure at base of wall 800 lbs. per foot.

Total pressure 12800 lbs.

$$M = \frac{wl^3}{6} = \frac{(800)(16)^3(12)^3}{6} = 17,280,000 \text{ in. lbs.}$$

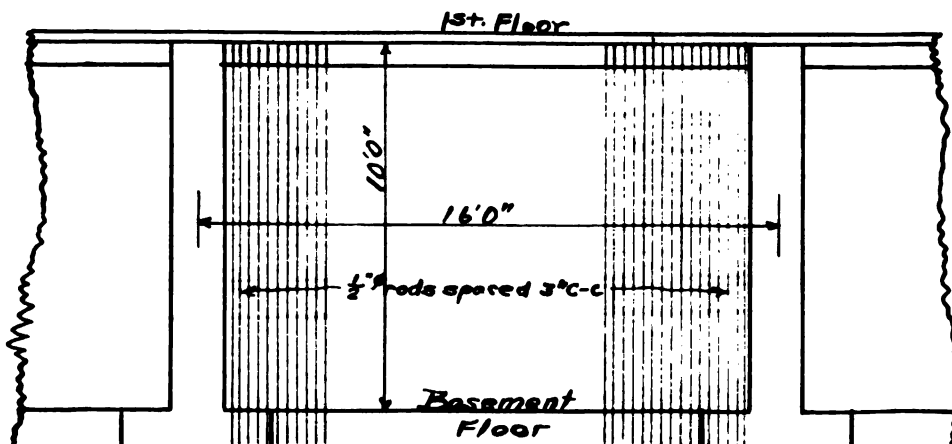
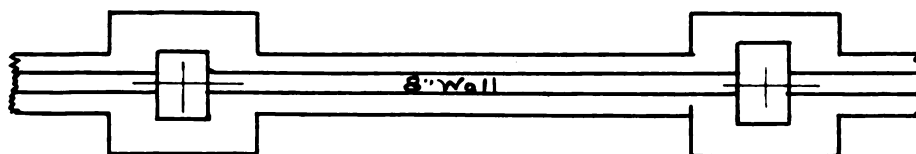


$$M = cd^2jfs_o$$

$$p = \frac{17,280,000}{(2)(8)^2(0.87)(16,000)} = 0.805$$

$$a_s = 0.805 \times 8 \times 12 = 0.7757 \text{ sq. in.}$$

use $r = 1/2$ " round rods spaced 6" c-c



Design 16.

Design an 8" basement wall 16 ft. high to extend between wall columns, 16" x 24", which are spaced 16' c-c. Consider that the wall is to sustain earth level with the top of the wall. Reinforce horizontally for earth pressure. The wall is to have no footing but is to rest on the wall column footings. Assumed earth pressure equal to a fluid weighing 80 lbs. per cubic foot.

Working stress as given by Joint Committee.

Wall designed as slab supported by two columns.

Pressure at base of wall 800 lbs.

Pressure tending to shear wall from column,

$$\frac{14.66 \times 800}{2} = 2200 \text{ lbs.}$$

Shear taken by one foot of wall above base,

$$(8)(12)(40) = 3840 \text{ lbs, wall safe for shear.}$$

Consider one foot at base,

$$M = \frac{Wl}{8} = \frac{(800)(14.66)(14.66)(12)}{8} = 22,500 \text{ in pas.}$$

$$p = \frac{22,500}{(8)^2(12)(16000)(0.85)} = 0.00817$$

$$a_s = 0.00817 \times 8 \times 12 = 0.87 \text{ sq. in. steel}$$

use 8 - 3/4" round at 8" c-c for 1st 8' from bottom.

at 7' from ceiling,

$$M = \frac{(7)(80)(14.66)(14.66)(12)}{8} = 27760 \text{ in pas.}$$

$$p = \frac{27760}{(8)^2(12)(16000)(0.85)} = 0.00645$$

$$a_s = (0.00645)(8)(12) = 0.622 \text{ sq. in. steel}$$

use 2 - 5/8" round rods at 8" c-c for next 3 feet.

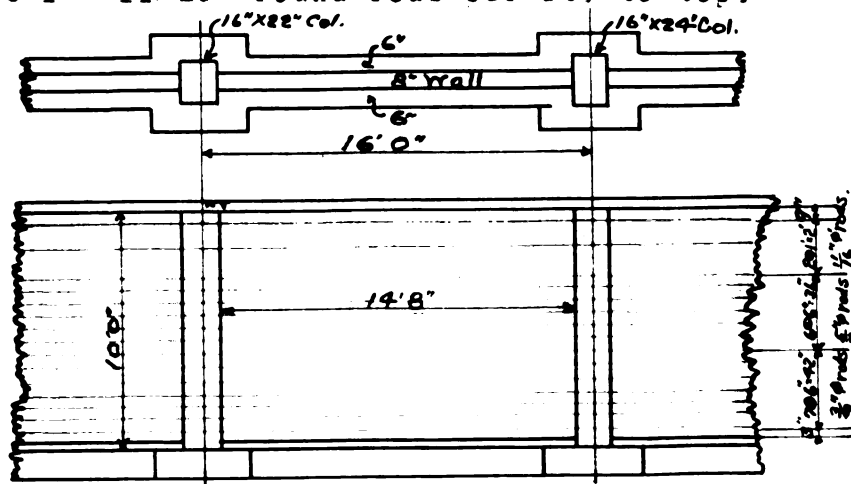
at 4' from ceiling,

$$M = \frac{(4)(80)(14.66)(14.66)(12)}{8} = 23720 \text{ in. pas.}$$

$$p = \frac{23720}{(8)^2(12)(16000)(0.85)} = 0.0037$$

$$a_s = (0.0037)(8)(12) = 0.368 \text{ sq. in. steel.}$$

use 1 - 1 1/16" round rods per ft. to top.



Design 17.

Interior floor bay with 1" granolithic finish supporting a live load of 200 l s. per sq. ft. Columns spaced 15' x 20' on centers. Girder span 15', beams 6' on centers. Use a part of bay for a stair shaft, provide enclosure partition. Height finished floor to finished floor 15 feet. Design stairway thru one story using 3 runs. Load on stair 100 lbs. per horizontal sq. foot. Working stresses as per Joint Committee.

Design of 3 lower steps and slab. $f_c = 650$ $p = 0.0077$
 width 6' 4" $f_s = 18000$ $k = 0.875$
 length 8' 4" $n = 12$ $j = 0.874$
 total live load $(6.33)(8.33)(100) = 5210$ lbs, 100 per ft.
 dead load assumed $\frac{5210}{2} = 2605$ lbs, 100 per ft.
 $\frac{5210}{2} = 2605$ lbs, 100 per ft.

assumed depth of steel 3.5", depth below steel 1", total 4.5"

$$M = \frac{wL^2}{12} = \frac{(126)(8.33)^2(12)}{12} = 13492 \text{ in. pas.}$$

$$M = K R x = (0.0077)(18000)(0.874)(12)d^2 = 13492$$

$$d = 3.2, \text{ use } 3.5" + 1" = 4.5" \text{ OK}$$

$$\text{area steel } (3.5)(12)(0.0077) = 0.328 \text{ sq. in.}$$

use 3/8" round rods spaced 4" c-c

Design of mid section.

width 6' 4"
 length 15' 4"
 $M = \frac{(126)(15.33)^2(12)}{12} = 52248 \text{ in pas.}$
 $M = (0.0077)(18000)(0.874)(12)d^2 = 52248$
 $d = 6.8", \text{ use } 6.5" + 1" = 7.5"$
 area steel $(6.5)(12)(0.0077) = 0.6 \text{ sq. in.}$
 use 4 - 7/16" round rods spaced 3" c-c

Top section.

width 6' 4"
 length 15' 4"
 $M = \frac{(126)(15.33)^2(12)}{12} = 52108 \text{ in pas.}$
 $M = (0.0077)(18000)(0.874)(12)(d)^2 = 52108$
 $d = 6.7", \text{ use } 6.75" + 1" = 7.75" \text{ total depth}$
 $a_s = (6.75)(12)(0.0077) = 0.6237 \text{ sq. in.}$
 use 6 - 3/8" round rods 2" c-c

Place 1/4" rods every 12" crosswise of steps for shrinkage reinforcement.

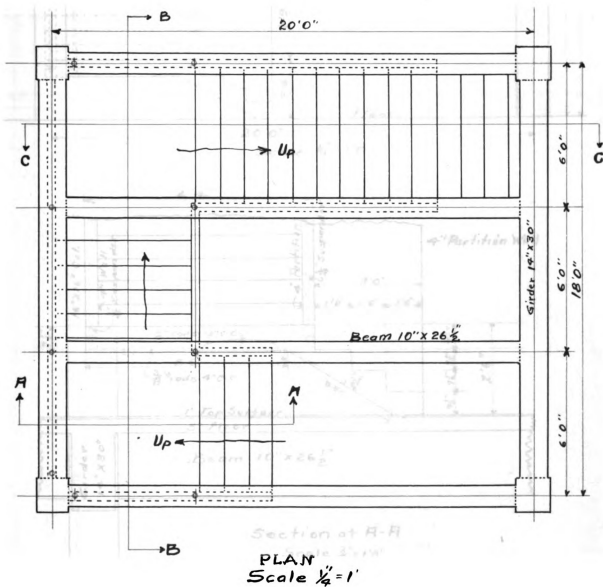
Suspension rods. 4 rods at 1st landing.

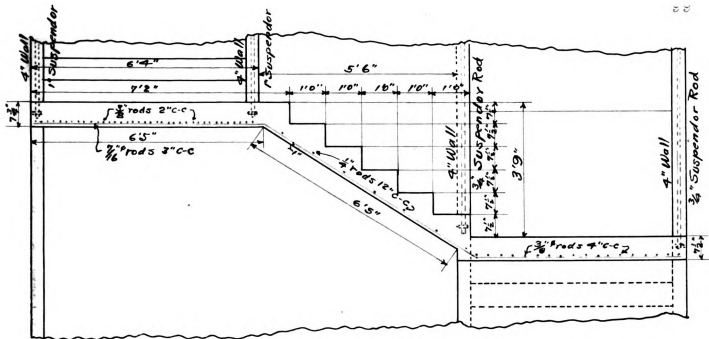
total load on one rod 622 1/2 lbs. lower suspenders

$\frac{6241}{16000} = 0.390$ sq. in. steel needed, use 3/4" round rods for four lower suspenders.

total load on one rod 11883 lbs, upper susceeners.

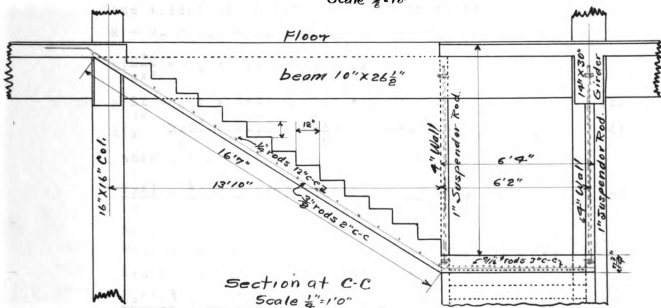
$\frac{11888}{16000} = 0.743$ sq. in. steel needed, use 1" round rods
for four upper suspenders.





Section at B-B

Scale $\frac{3}{8}$ "=1'0"



Section at C-C

Scale $\frac{1}{4}$ "=1'0"

Steel List.

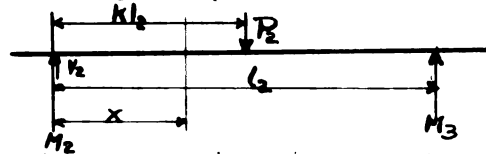
No.	Dimensions	Location
18	$\frac{3}{8}$ " - 9'0"	Lower rise and slab
4	$\frac{1}{4}$ " - 6'0"	" " shrinkage
24	$\frac{1}{2}$ " - 18'0"	Second rise and slab
6	$\frac{1}{2}$ " - 6'0"	" " shrinkage
36	$\frac{3}{8}$ " - 23'6"	Upper rise and slab
15	$\frac{1}{4}$ " - 6'0"	" " shrinkage
5	$\frac{3}{4}$ " - 12'6"	Lower Landing Suspender
4	1" - 8'7"	Upper Landing Suspender

Claperyon's theorem of three moments.

This theorem is stated in equation form on page 648 in Chapter 12 as follows;

$$M_2 l_2 + 2M_3(l_2 + l_3) + M_4 l_3 = -2E_2 I_2^2 (1 - \alpha^2) - 2E_3 I_3^2 (2(1 - \alpha)^2 + \alpha^2)$$

Consider a single span first



take moment at section for part to left.

$$M = M_2 + V_2 x$$

$$EI \frac{d^2 y}{dx^2} = M = M_2 + V_2 x$$

$$EI \frac{dy}{dx} = M_2 x + \frac{V_2 x^2}{2} + C_1 \quad (1)$$

$$EI y = \frac{M_2 x^2}{2} + \frac{V_2 x^3}{6} + C_1 x + C_2 \quad (2)$$

when $x = 0$ $y = 0$ therefore $C_2 = 0$

take moment at section for part to right.

$$M = M_3 + P_2 x - P_2 (x - \alpha l_2)$$

$$EI \frac{d^2 y}{dx^2} = M_3 + V_3 x - P_2 (x - \alpha l_2)$$

$$EI \frac{dy}{dx} = V_3 x + \frac{V_3 x^2}{2} - \frac{P_2 x^2}{2} + P_2 \alpha l_2 + C_3 \quad (3)$$

$$EI y = \frac{V_3 x^3}{6} + \frac{V_3 x^3}{6} - \frac{P_2 x^3}{6} + \frac{P_2 \alpha l_2 x^2}{2} + C_3 x + C_4 \quad (4)$$

when $y = 0$ $x = l_2$

$$\frac{M_2 l_2^2}{2} + \frac{V_2 l_2^3}{6} - \frac{P_2 l_2^3}{6} + \frac{P_2 \alpha l_2^3}{2} + C_1 l_2 + C_4 = 0 \quad (5)$$

it is same for each section

when y is same for each part of beam, $y = \alpha l_2$

place equation (4) = equation (2)

$$\frac{M_2 \alpha^2 l_2^2}{2} + \frac{V_2 \alpha^3 l_2^3}{6} - \frac{P_2 \alpha^3 l_2^3}{6} + \frac{P_2 \alpha^2 l_2^3}{2} + C_1 \alpha l_2 + C_4 =$$

$$\frac{M_2 \alpha^2 l_2^2}{2} + \frac{V_2 \alpha^3 l_2^3}{6} + C_1 \alpha l_2 \quad (6)$$

$$\frac{P_2 \alpha^3 l_2^3}{6} + C_4 \alpha l_2 + C_4 = C_1 l_2 \quad (7)$$

when $x = \alpha l_2$, the $\frac{dy}{dx}$ of each equation are equal

place equation (1) = equation (3)

$$M_2 \alpha l_2 + \frac{V_2 \alpha^2 l_2^2}{2} + C_1 = M_3 \alpha l_2 + \frac{V_3 \alpha^2 l_2^2}{2} - \frac{P_2 \alpha^2 l_2^2}{2} + P_2 \alpha^2 l_2 + C_3 \quad (8)$$

$$C_1 = \frac{P_2 \alpha^2 l_2^2}{2} + C_3 \quad (9)$$

multiply equation (9) by αl_2

$$C_1 \alpha l_2 = \frac{P_2 \alpha^3 l_2^3}{2} + C_3 \alpha l_2 \quad (10)$$

Place equation (7) = equation (10)

$$\frac{P_2 K^3 l_2^3}{2} + C_3 l_2 + C_4 = \frac{P_2 K^3 l_2^3}{2} + C_3 l_2 \quad (11)$$

$$C_4 = \frac{P_2 K^3 l_2^3}{6} \quad (12)$$

Substitute equation (12) in equation (8)

$$\frac{M_2 l_2^2}{2} + \frac{V_2 l_2^3}{6} - \frac{P_2 l_2^3}{6} + \frac{P_2 K l_2^3}{2} + C_3 l_2 + \frac{P_2 K^3 l_2^3}{6} = 0 \quad (13)$$

$$C_3 = -\frac{M_2 l_2}{2} - \frac{V_2 l_2^2}{6} + \frac{P_2 l_2^2}{6} - \frac{P_2 K l_2^2}{2} - \frac{P_2 K^3 l_2^2}{6} \quad (14)$$

substitute equation (14) in equation (8)

$$C_1 = \frac{P_2 K^3 l_2^3}{2} - \frac{M_2 l_2}{2} - \frac{V_2 l_2^2}{6} + \frac{P_2 l_2^2}{6} - \frac{P_2 K l_2^2}{2} - \frac{P_2 K^3 l_2^3}{6} \quad (15)$$

the tangent equations are equal when $x = l_1$,

left section, substitute equation (15) in equation (1)

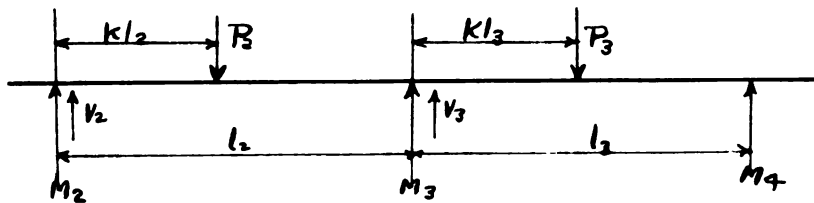
$$l_1 \frac{dY}{dx} = M_2 x + \frac{V_2 x^2}{2} - \frac{P_2 K^3 l_2^3}{2} + \frac{M_2 l_2}{2} - \frac{V_2 l_2^2}{6} + \frac{P_2 l_2^2}{6} - \frac{P_2 K l_2^2}{2} - \frac{P_2 K^3 l_2^3}{6} \quad (16)$$

right section, substitute equation (14) in equation (2)

$$l_1 \frac{dY}{dx} = M_2 x + \frac{V_2 x^2}{2} - \frac{P_2 x^2}{2} + P_2 l_2 x - \frac{V_2 l_2}{2} - \frac{V_2 l_2^2}{6} + \frac{P_2 l_2^2}{6} - \frac{P_2 K l_2^2}{2} - \frac{P_2 K^3 l_2^2}{6} \quad (17)$$

In a continuous beam the tangents are common immediately

over the support under consideration.



For a point over V_2 place equation (17) = equation (16), using the proper notation of the span in which the equation is placed. Equation (17) has notation of span V_2, V_3 and equation (16) has notation of span V_3, V_4 . $x = l_1$ in equation (17). $x = P$ in equation (16). 6 is the common denominator.

$$6M_2 l_1 + 6V_2 l_1^2 - 6P_2 l_1^2 + 6P_2 K l_1^2 - 6M_2 l_2 - V_2 l_1^2 + P_2 l_1^2 - 6P_2 K l_1^2 - P_2 K^3 l_1^2 = 6P_2 K^3 l_1^2 - 6M_2 l_2 - V_2 l_1^2 + P_2 l_1^2 - 6P_2 K l_1^2 - P_2 K^3 l_1^2 \quad (18)$$

Collecting terms;

$$6M_2 l_1 + 6V_2 l_1^2 + 2V_2 l_1^2 + V_2 l_1^2 = -6V_2 l_1^2 + 2P_2 l_1^2 + P_2 K^3 l_1^2 + 6P_2 K^3 l_1^2 - 6P_2 K l_1^2 + P_2 l_1^2 - P_2 K^3 l_1^2 \quad (19)$$

$$V_2 l_1 = V_2 - P_2 + P_2 l_1 - P_2 K l_1 \quad (20)$$

$$2V_2 l_1^2 = 2V_2 l_1^2 - 2V_2 l_1^2 + 2P_2 l_1^2 - 2P_2 K l_1^2 \quad (21)$$

$$V_2 l_1^2 = V_4 - V_2 + P_2 l_1 - P_2 K l_1 \quad (22)$$

$$V_2 l_1^2 = V_4 l_1 - V_2 l_1 + P_2 l_1^2 - P_2 K l_1^2 \quad (23)$$

Substitute equations (21) and (23) in equation (18)

$$3M_2l_2 + 3M_3l_3 + 2V_2l_2 - 2V_3l_2 + 2P_2l_2^2 - 2P_3l_1l_2 + M_4l_3 - V_3l_3 + P_3l_3^2 - P_3Kl_3^2 = 2P_2l_2^2 + P_3l_3^2 - 3P_2Kl_2^2 + P_2K^3l_2^2 - 3P_3Kl_1l_2^2 + 3P_3K^2l_1l_2^2 - P_3K^3l_1l_2^2 \quad (24)$$

Simplifying:

$$M_2l_2 + 2M_3l_2 + 2M_3l_3 + M_4l_3 = -P_2Kl_2^2 + P_2K^3l_2^2 - 2P_3Kl_1l_2^2 + 3P_3K^2l_1l_2^2 - P_3K^3l_1l_2^2 \quad (25)$$

Grouping terms=

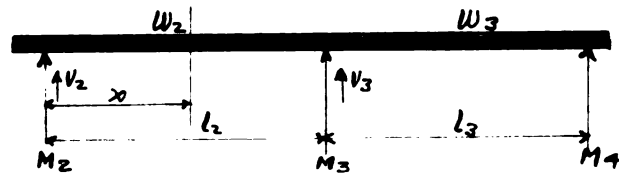
$$M_2l_2 + 2M_3(l_2 + l_3) + M_4l_3 = -P_2l_2^2(K - K^3) - P_3l_3^2(2K - 3K^2 + K^3)$$

This is in the form as stated. If more than one concentrated load occurs in a span, use the sign 2 before the two terms in the last part of the equality.

On page 348 is given the equation for three moments for uniform loads,

$$M_2l_2 + 2M_3(l_2 + l_3) + M_4l_3 = -\frac{1}{4}w_2l_2^3 - \frac{1}{4}w_3l_3^3$$

Derivation as follows:



In span V_2, V_3 moments for left section,

$$M = V_2 + V_2x - w_2x \frac{x}{2}$$

$$\frac{d}{dx} \frac{1}{2} w_2 x^2 = V_2 + V_2x - w_2 \frac{x^2}{2}$$

$$\frac{d}{dx} \frac{1}{2} w_2 x^2 = V_2x + \frac{V_2x^2}{2} - \frac{w_2x^3}{6} + C_1 \quad (1)$$

$$\frac{d}{dx} y = \frac{V_2x^2}{2} + \frac{V_2x^3}{6} - \frac{w_2x^4}{24} + C_1x + C_2 \quad (2)$$

when $x = 0$ the $y = 0$, therefore $C_2 = 0$

when $x = l_2$ then $y = 0$

$$C_1 = -\frac{V_2l_2^3}{2} - \frac{V_2l_2^3}{6} - \frac{w_2l_2^4}{24} \quad (3)$$

$$\frac{d}{dx} \frac{1}{2} w_2 x^2 = V_2x + \frac{V_2x^2}{2} - \frac{w_2x^3}{6} - \frac{V_2l_2^3}{2} - \frac{V_2l_2^3}{6} - \frac{w_2l_2^4}{24} \quad (4)$$

similarly in span V_3, V_4

$$\frac{d}{dx} \frac{1}{2} w_3 x^2 = V_3x + \frac{V_3x^2}{2} - \frac{w_3x^3}{6} - \frac{V_3l_3}{2} - \frac{V_3l_3^2}{6} - \frac{w_3l_3^3}{24} \quad (5)$$

These two tangents are common over support at V_3

here $x = l_2$ for equation (4), $x = 0$ for equation (5)

$$V_2l_2 + \frac{V_2l_2^2}{2} - \frac{w_2l_2^3}{6} - \frac{V_2l_2^3}{2} - \frac{V_2l_2^3}{6} - \frac{w_2l_2^4}{24} = -\frac{V_3l_3}{2} - \frac{V_3l_3^2}{6} - \frac{w_3l_3^3}{24} \quad (6)$$

$$\frac{V_2l_2}{2} + \frac{V_3l_3}{2} + \frac{V_2l_2^2}{6} + \frac{V_3l_3^2}{6} = \frac{3w_2l_2^3}{24} + \frac{w_3l_3^3}{24} \quad (7)$$

$$V_2l_2 = V_3 - V_2 + \frac{w_2l_2^2}{2} \quad (8)$$

$$V_3 l_3 = M_4 - M_3 + \frac{w_3 l_3^2}{2} \quad (8)$$

Substitute equations (6) and (8) in equation (7)

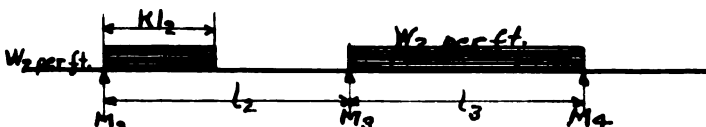
$$\frac{M_2 l_2}{2} + \frac{M_3 l_3}{2} + \frac{M_3 l_2}{6} - \frac{M_3 l_2}{6} + \frac{w_2 l_2^3}{24} + \frac{M_4 l_3}{6} - \frac{M_3 l_3}{6} + \frac{w_3 l_3^3}{12} = \frac{3w_2 l_2^3}{24} + \frac{w_3 l_2^3}{24} \quad (10)$$

$$\frac{M_2 l_2}{6} + \frac{2M_3 l_3}{6} + \frac{M_3 l_2}{6} + \frac{M_4 l_3}{6} = \frac{3w_2 l_2^3}{24} - \frac{w_2 l_2^3}{6} + \frac{w_3 l_3^3}{24} - \frac{w_3 l_3^3}{12} \quad (11)$$

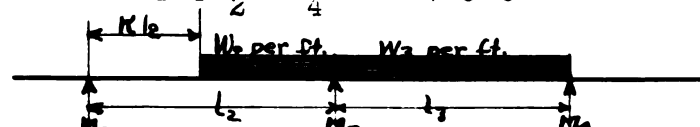
$$M_2 l_2 + 2M_3(l_2 + l_3) + M_4 l_3 = -\frac{w_2 l_2^3}{4} - \frac{w_3 l_3^3}{4}$$

this is in the form as stated.

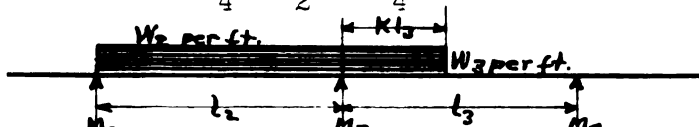
The ten forms for this three moment equation are given on page 351. The derivation of these ten are made in a similar manner to the two which are made a part of this thesis.

(a) 

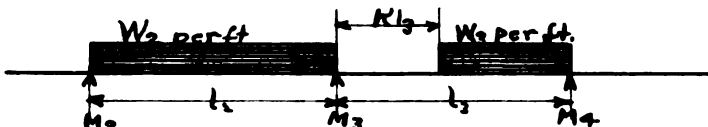
$$r = \pi w_2 l_2^3 \left(\frac{k^2}{2} - \frac{k^4}{4} \right) - \frac{1}{4} w_3 l_3^3$$

(b) 

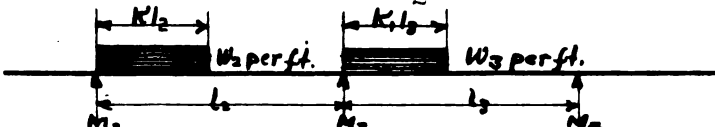
$$r = -w_2 l_2^3 \left(\frac{1}{4} - \frac{k^2}{2} + \frac{k^4}{4} \right) - \frac{1}{4} w_3 l_3^3$$

(c) 

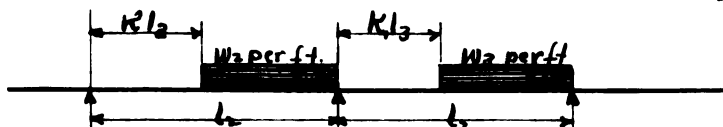
$$r = -\frac{1}{4} w_2 l_2^3 - w_3 l_3^3 \left(k^2 - k^3 + \frac{k^4}{4} \right)$$

(d) 

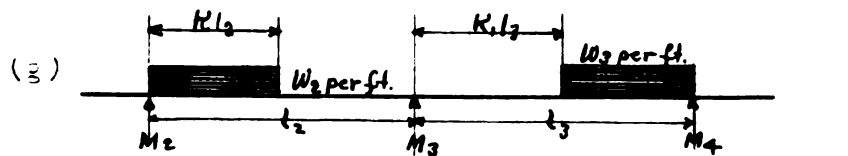
$$r = -\frac{1}{4} w_2 l_2^3 - w_3 l_3^3 \left(\frac{1}{2} - k^2 + k^3 - \frac{k^4}{8} \right)$$

(e) 

$$r = -w_2 l_2^3 \left(\frac{k^2}{2} - \frac{k^4}{4} \right) - w_3 l_3^3 \left(k_1^2 - k_1^3 + \frac{k_1^4}{4} \right)$$

(f) 

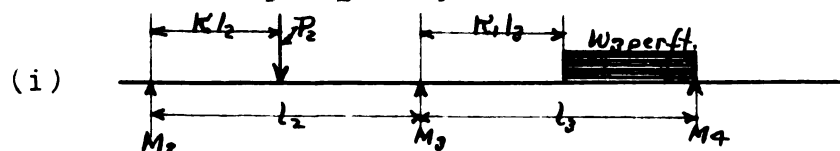
$$r = -w_2 l_2^3 \left(\frac{1}{2} - \frac{k}{2} + \frac{k^4}{4} \right) - w_3 l_3^3 \left(\frac{1}{4} - k_1^2 + k_1^3 - \frac{k_1^4}{4} \right)$$



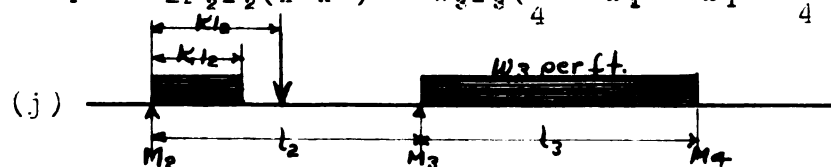
$$r = -w_2 l_2^3 \left(\frac{k^2}{2} - \frac{k^4}{4} \right) - w_3 l_3^3 \left(\frac{1}{4} - k_1^2 + k_1^3 - \frac{k_1^4}{4} \right)$$



$$r = -w_2 l_2^3 \left(\frac{1}{4} - \frac{k^2}{2} + \frac{k^4}{4} \right) - w_3 l_3^3 \left(k_1^2 - k_1^3 + \frac{k_1^4}{4} \right)$$



$$r = -\Sigma P_2 l_2^2 (k - k^3) - w_3 l_3^3 \left(\frac{1}{4} - k_1^2 + k_1^3 - \frac{k_1^4}{4} \right)$$

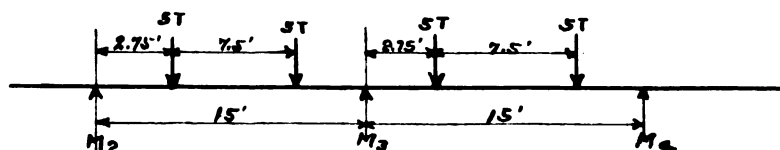


$$r = -\Sigma P_2 l_2^2 (k - k^3) - w_3 l_3^3 \left(\frac{k_1^2}{2} - \frac{k_1^4}{4} \right) - \frac{1}{4} w_3 l_3^3$$

In the above equation $r = \Sigma M_2 l_2 + 2M_3 (l_2 + l_3) + M_4 l_3$

Design 18.

A continuous beam has two 15 foot spans and fixed ends. Compute the moment over the center support for 10,000 lb. loads at the quarter points in each span.



$$M_2 l_2 + 2M_3 (l_2 + l_3) + M_4 l_3 = -P_1 l_2^2 (k - k^3) - P_2 l_2^2 (k_1 - k_1^3) - P_3 l_3^2 (2k_2 - 3k_2^2 + k_2^3) - P_4 l_3^2 (2k_3 - 3k_3^2 + k_3^3)$$

$$- P_1 = P_2 = P_3 = P_4 = 10,000 \text{ lbs.}$$

$$k = 0.25 \quad k_1 = 0.75 \quad k_2 = 0.25 \quad k_3 = 0.75$$

$$M_2 = M_4 \quad M_3 = 2M_2 = 2M_4$$

$$15M_2 + 60M_3 + 15M_4 = -10000(15)^2 [0.25 - (0.25)^3] - 10000(15)^2 [0.75 - (0.75)^3] - 10000(15)^2 [2(0.25) - 3(0.25)^2 + (0.25)^3] - 10000(15)^2 [2(0.75) - 3(0.75)^2 + (0.75)^3]$$

$$15M_2 + 15M_4 = 30M_2$$

$$\frac{30M_2}{2M_2} = 15M_3$$

$$75M_2 = -827,400 - 75M_3, 225 - 827,400$$

$$M_3 = 88750 \text{ ft. lbs.}$$

Design 19.

A continuous beam of three 18 foot spans with supported ends is subjected to a moving load of 2000 pounds anywhere along its length. Find maximum moment and maximum shear which occur in this beam.

Maximum moment load on first span.

Load at 6.75' from end support,

$$M = 2.048 \times 1.8 \times 2000 = 76812.8 \text{ ft. lbs.}$$

Maximum moment load on middle span.

Load at middle of second span.

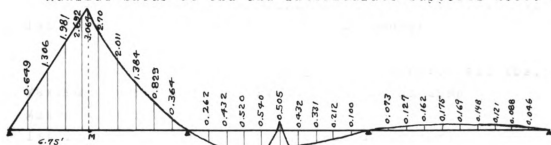
$$M = 1.72 \times 1.8 \times 2000 = 62025 \text{ ft. lbs.}$$

Maximum moment over intermediate support.

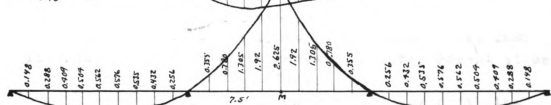
Load 6' from end support.

$$M = 1.024 \times 1.8 \times 2000 = 36860 \text{ ft. lbs.}$$

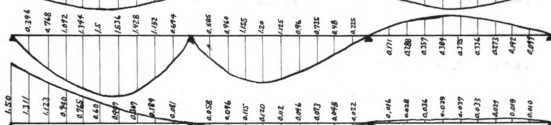
Maximum shear at end and intermediate supports 2000 lbs.



Mon. at
M.

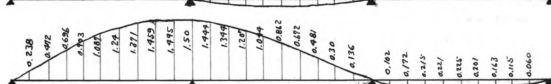


Mon. at
M.

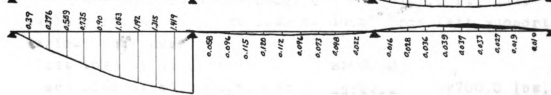


Mon. over
Intermediate
Support.

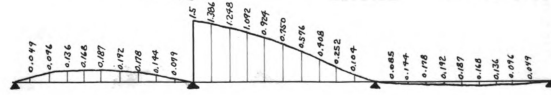
End Shears



Intermediate
Reactions.



Shear, end
span at
Intermediate
Support.



Shear in
center span
at Inter-
mediate
Support.

Design 20.

A continuous beam of three 10' spans with supported ends is subject to two moving loads of 20000 pounds each, spaced 8' apart. Find maximum moments and maximum shear which may occur. See diagrams on page 103.

Loads moving from left to right, maximum moment end span.

1st load 8.75' from left support, moment at 8.75' from end.

$$20000 \times 2.048 \times 1.5 = 76812.8$$

$$20000 \times 0.482 \times 1.5 = 16267.2 \quad 82580.0 \text{ ft. lbs.}$$

1st load at middle of span, moment at center.

$$20000 \times 2.0 \times 1.5 = 75000$$

$$20000 \times 0.62 \times 1.5 = 22220 \quad 97220 \text{ ft. lbs.}$$

1st load 5' from support, moment at center.

$$20000 \times 1.48 \times 1.5 = 56575.0$$

$$20000 \times 1.012 \times 1.5 = 37260.0 \quad 83835.0 \text{ ft. lbs.}$$

1st load 5' from support, moment at center.

$$20000 \times 1.850 \times 1.5 = 66250.0$$

$$20000 \times 0.745 \times 1.5 = 27225.0 \quad 93475.0 \text{ ft. lbs.}$$

Maximum moment with 1st load at middle of span

Moments in middle span.

1st load at middle of span, moment at center.

$$20000 \times 1.78 \times 1.5 = 63025.0$$

$$20000 \times 0.428 \times 1.5 = 11124.0 \quad 74149.0 \text{ ft. lbs.}$$

1st load at 7.5' from intermediate support, moment at center.

$$20000 \times 1.88 \times 1.5 = 66750.0$$

$$20000 \times 0.82 \times 1.5 = 24200.0 \quad 90950.0 \text{ ft. lbs.}$$

Moment over intermediate support.

1st load 5' from left support.

$$20000 \times 1.024 \times 1.5 = 30660.0$$

$$20000 \times 0.888 \times 1.5 = 26220.0 \quad 56880.0 \text{ ft. lbs.}$$

1st load at 10.5' from left support.

$$20000 \times 0.882 \times 1.5 = 33060.0$$

$$20000 \times 0.714 \times 1.5 = 21100.0 \quad 54160.0 \text{ ft. lbs.}$$

1st load at 12' from left support.

$$20000 \times 0.718 \times 1.5 = 21000.0$$

$$20000 \times 0.888 \times 1.5 = 26220.0 \quad 47220.0 \text{ ft. lbs.}$$

Maximum moment with 1st load at 10.5' from left support.

Maximum end shear.

$$1st \text{ load at support.} \quad 20000.0$$

$$2nd \text{ load at 5', } 20000 \times 0.888 = 17760.0 \quad 37760.0 \text{ lbs.}$$

Intermediate reaction.

1st load at 10.5' from end support.

$$0.914 \times 25000 = 22850$$

$$0.858 \times 25000 = \underline{21450} \quad 47800 \text{ lbs.}$$

1st load at 1.5' past support.

$$0.963 \times 25000 = 24075$$

$$0.953 \times 25000 = \underline{23825} \quad 47900 \text{ lbs.}$$

1st load at 12' from end support

$$0.975 \times 25000 = 24375$$

$$0.941 \times 25000 = \underline{23525} \quad 47800 \text{ lbs.}$$

1st load at 13.5' from end support

$$0.987 \times 25000 = 24675$$

$$0.868 \times 25000 = \underline{21700} \quad 46575 \text{ lbs.}$$

Shear in end span at intermediate support.

$$1st \text{ load at support} \quad 25000$$

$$2nd \text{ load } 25000 \times 0.764 = \underline{19100} \quad 44100 \text{ lbs.}$$

Shear in center span at intermediate support.

1st load 8' from support

$$0.891 \times 25000 \quad 17275$$

$$2nd \text{ load at support} \quad 25000 \quad 42275 \text{ lbs.}$$

$$\text{Maximum moment in end span} \quad 98240 \text{ ft. lbs.}$$

$$\text{Maximum moment in middle span} \quad 51455 \text{ ft. lbs.}$$

$$\text{Maximum moment over intermediate support} \quad 67200 \text{ ft. lbs.}$$

$$\text{Maximum end shear} \quad 32700 \text{ lbs.}$$

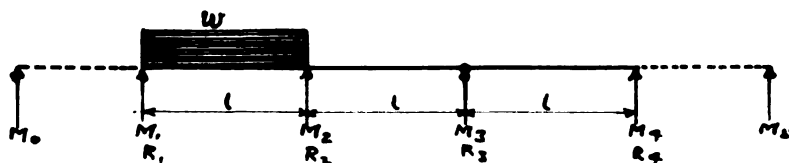
$$\text{Maximum intermediate reaction} \quad 47900 \text{ lbs.}$$

$$\text{Shear in end span at intermediate support} \quad 44100 \text{ lbs.}$$

$$\text{Shear in center span at intermediate support} \quad 42275 \text{ lbs.}$$

Design 21.

Construct moment and shear diagram for a beam of three spans loaded on only one of the outer spans. Consider both fixed and supported ends. $w = 200 \text{ lbs.}$ $l = 15'$



ends fixed.

$$M_0 l + 2M_1(l+1) + M_2 l = -\frac{1}{4}w_0 l^3 - \frac{1}{4}w_1 l^3$$

$$M_1 l + 2M_2(l+1) + M_3 l = -\frac{1}{4}w_1 l^3 - \frac{1}{4}w_2 l^3$$

$$M_2 l + 2M_3(l+1) + M_4 l = -\frac{1}{4}w_2 l^3 - \frac{1}{4}w_3 l^3$$

$$M_3 l + 2M_4(l+1) + M_5 l = -\frac{1}{4}w_3 l^3 - \frac{1}{4}w_4 l^3$$

$$M_0 = 0, w_0 = 0, w_2 = 0, w_3 = 0, w_4 = 0, l_1 = 0, l_5 = 0$$

$$2M_1 + M_2 = -\frac{1}{4}w_1 l^2$$

$$M_1 + 4M_2 + M_3 = -\frac{1}{4}w_1 l^2$$

$$M_2 + 4M_3 + M_4 = 0$$

$$M_3 + 2M_4 = 0$$

$$M_3 = -2M_4$$

$$M_2 - 8M_4 + M_4 = 0$$

$$M_2 = 7M_4$$

$$M_1 + 28M_4 - 2M_4 = -\frac{1}{4}w_1 l^2$$

$$2M_1 + 7M_4 = -\frac{1}{4}w_1 l^2$$

$$2M_1 + 82M_4 = -\frac{1}{4}w_1 l^2$$

$$48M_4 = -\frac{1}{4}w_1 l^2$$

$$48M_4 = -\frac{1}{4}(200)(15)^2$$

$$M_4 = -250$$

$$M_3 = -1750$$

$$M_3 = 500$$

$$2M_1 = -11250 + 1750 = -9500$$

$$M_1 = -4750$$

$$V = \frac{M_2 - M_1}{l} + \frac{1}{2}w_1 l$$

$$V_1 = \frac{-1750 - (-4750)}{15} + \frac{1}{2}(200)(15) = 1700$$

$$V_2 = V_1 - wx$$

$$V_2 = 1700 - 2000 = -1500$$

$$V_3 = \frac{-250 - 500}{15} = -50$$

$$V_2 = 150$$

1st span.

$$M_4 = M_1 + V_1 4 - \frac{1}{2}w(4)^2$$

$$M_4 = -4750 + (1700)(4) - \frac{1}{2}(200)(4)^2 = 450$$

$$M_3 = -4750 + (1700)(5) - \frac{1}{2}(200)(5)^2 = 2450$$

$$M_{11} = -4750 + (1700)(11) - \frac{1}{2}(200)(11)^2 = 1550$$

2nd span.

$$M_4 = M_3 + V_2 4 - 0$$

$$M_4 = -1750 + (150)(4) = -1150$$

$$M_3 = -1750 + (150)(5) = -550$$

$$M_{12} = -1750 + (150)(12) = 50$$

3rd span.

$$M_4 = M_2 + V_3 4$$

$$M_4 = 500 + (-50)(4) = 300$$

$$M_3 = 500 + (-50)(5) = 100$$

$$M_{12} = 500 + (-50)(12) = -150$$

ends freely supported.

$$M_1 + 4M_2 + M_3 = -\frac{1}{2}wl^2$$

$$M_2 + 4M_3 + M_4 = 0$$

being freely supported $M_1 = M_4 = 0$

$$4M_2 + M_3 = -\frac{1}{2}wl^2$$

$$M_2 + 4M_3 = 0$$

$$M_2 = -4M_3$$

$$-16M_3 + M_3 = -\frac{1}{2}(200)(15)^2$$

$$M_3 = 750$$

$$M_2 = -3000$$

taking moments at R_2

$$15R_1 - (200)(15)(7.5) = -3000$$

$$15R_1 = 22500 - 3000$$

$$R_1 = 1300$$

taking moments at R_3

$$15R_2 + (1300)(30) = 750$$

$$15R_2 = 750 - 39000$$

$$R_2 = -2550$$

Taking moments at R_4

$$15R_3 + (-2550)(30) + (1300)(45) = 0$$

$$15R_3 = 15000$$

$$R_3 = 1000$$

taking moments at R_5 for R_4

$$15R_4 = 750$$

$$R_4 = 50$$

1st span

$$M_4 = M_1 + V_1x - \frac{1}{2}w(x)^2$$

$$M_4 = 0 + (1300)(4) - \frac{1}{2}(200)(16) = 2600$$

$$M_8 = 0 + (1300)(8) - \frac{1}{2}(200)(64) = 4000$$

$$M_{12} = 0 + (1300)(12) - \frac{1}{2}(200)(144) = 1200$$

$$M_{14} = 0 + (1300)(14) - \frac{1}{2}(200)(196) = -1400$$

2nd span.

$$V_2 = \frac{M_3 - M_2}{15} = \frac{750 - (-3000)}{15} = 250$$

$$M_4 = M_2 + V_2x$$

$$M_4 = -3000 + (250)(4) = -2000$$

$$M_8 = -3000 + (250)(8) = -1000$$

$$M_{12} = -3000 + (250)(12) = 0$$

3rd Span.

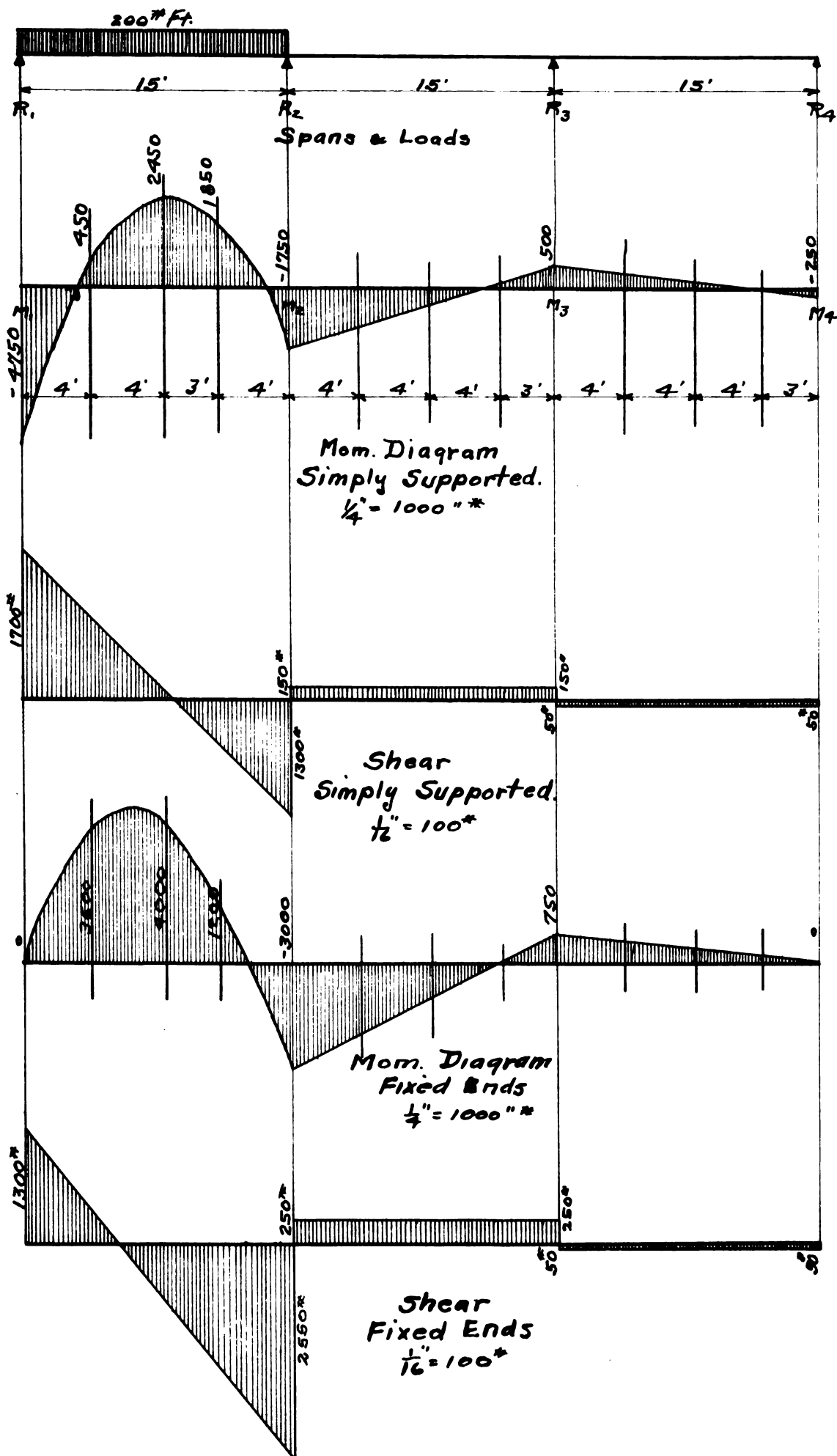
$$V_3 = 450$$

$$M_4 = 750 - (50)(4) = 550$$

$$M_8 = 750 - (50)(8) = 350$$

$$M_{12} = 750 - (50)(12) = 150$$

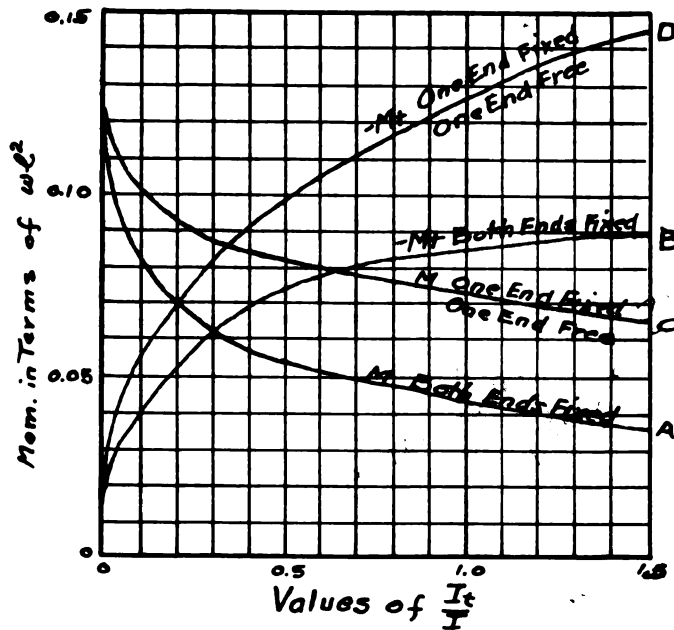
$$M_{12}$$



Design 22.

If the center of a span of a continuous beam with practically fixed ends is reinforced for a moment of $\frac{wl^2}{10}$, what is the least amount of steel that can be safely placed over the top of the beam at the supports.

$$\text{Moment at center for } \frac{wl^2}{10} = 0.068 \, wl^2$$



$-M_t$ denotes Max. Mom. at Support.
 M " " " between Supports.
 I_t = Mom. of inertia at Support.
 I = " " " " center Span.

from the curves we find the intersection of 0.068 with $-M_t$ for both ends fixed. This intersects at 0.2 for the value of

$$\frac{I_t}{I}$$

$$\text{Moment at center} = 0.068 \, wl^2$$

$$\text{Moment at support} = 0.068 \, wl^2$$

for a 0.2 steel, or the steel is stressed $\frac{0.136}{0.068} = 1.00$ times the assured working stress.

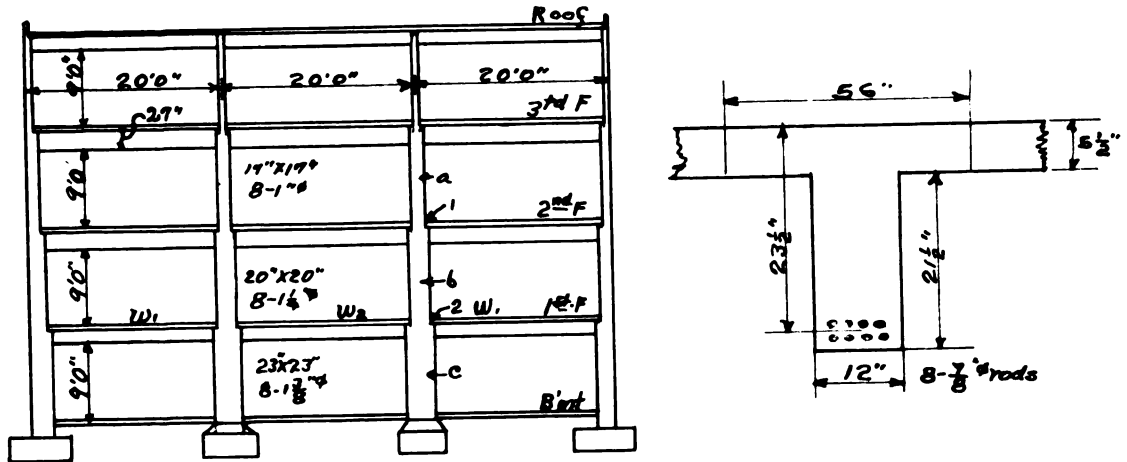
Design 28.

Two parts, (a) and (c).

(a) Find stresses in interior columns b and c, considering the ends of beam as freely supported and that $B_1 = B_2$, $K = 4$

Live load 200 lbs. Dead load 60 lbs.

20' spans, beams 10' c-c.



$$w_d = (60)(10) + \frac{(12)(21.5)(150)}{144} = 880 \text{ lbs.}$$

$$w_t = (200)(10) + 880 = 2880 \text{ lbs.}$$

$$C = \frac{1}{10} = \frac{(0.055)(20)(21.5)^2}{(20)(12)} = 109$$

The worst condition of loading is when the two end spans are fully loaded and the middle span unloaded except for the dead load.

Live load on one span, $L = 0.551(w_t + w_d) = 48120 \text{ lbs.}$

Live load on two spans, $L = (1.25)(2880)(20) = 74000 \text{ lbs.}$

Direct load on column c,

$$(2)(74000) + 48120 = 196120 \text{ lbs.}$$

Joint 1. live load in one span.

$$C = 109$$

$$C_a = \frac{(0.055)(14)(14)^2 + (12)(21.5)(7)^2}{(8)(12)} = 64$$

$$C_c = \frac{(0.055)(17)(17)^2 + (12)(21.5)(9.5)^2}{(8)(12)} = 148$$

$$K_a C_a + K_c C_c = 4(64 + 148) = 820$$

$$a_2 = \frac{1^2}{121} \times \frac{1.25 w_t + w_d}{C_c + 60} = \frac{(20)^2 (12)^2 (3280)}{121 \times (12)(1878)} = \frac{1018.5}{11}$$

stress due to bending,

$$\frac{M x_1}{I} = \frac{410.5 x_1}{1} = \frac{(4)(1018.5)(9.5)}{(8)(12)} = 318.1 \text{ lbs. sq. in.}$$

Equivalent concrete area of column,

$$(17)(17) + (8.82)(11) = 387.0 \text{ sq. in.}$$

Stress due to direct compression,

$$\frac{191120}{897} = 481.4 \text{ lbs. sq. in.}$$

Maximum stress $481.4 + 318.1 = 800.5 \text{ lbs. sq. in.}$

Joint 1, live load on both spans.

$$R = (3)(74000) = 222000 \text{ lbs.}$$

$$\text{stress } \frac{222000}{897} = 240 \text{ lbs.}$$

design for $1:1\frac{1}{2} : 3$ concrete, $f_c = 240 \text{ lbs.}$

Joint 2, live load on one span.

$$C_c = \frac{(0.053)(20)(20)^2 + (12)(11.55)(10)^2}{(2)(12)} = 285$$

$$K_c C_c + K_c C_c = 4(145 + 285) = 1592$$

$$a_2 = \frac{1^2}{12} \times \frac{(1.2W_1 - W_2)}{K_c + 25} = \frac{(20)^2(12)^2(5450)}{(12)^2(2137)} = 351.3$$

Stress due to bending

$$\frac{M x_1}{I} = \frac{410 a_2 x_1}{1} = \frac{(4)(351.3)(10)}{(2)(12)} = 241.2 \text{ lbs. sq. in.}$$

Equivalent concrete area of column,

$$(20)(20) + (11.55)(11) = 530.7 \text{ sq. in.}$$

total load on C column, 265120 lbs.

stress due to direct compression,

$$\frac{265120}{530.7} = 499.8 \text{ lbs. per sq. in.}$$

Maximum stress $499.8 + 241.2 = 740.7 \text{ lbs. sq. in.}$

Joint 2, live load on both spans.

$$R = (74000)(4) = 296000$$

$$\text{stress } \frac{296000}{530.7} = 557 \text{ lbs.}$$

design for $1 : 1\frac{1}{2} : 3$ concrete, $f_c = 550 \text{ lbs.}$

(c) Assume exterior columns of concrete with $K_2 C_2 = K_1 C_1$. Find stress in outside lower column by exact and approximate methods. Worst condition of loading for three spans; two outside spans fully loaded, middle span unloaded.

$$W_1 = 880, \quad W_2 = 2960, \quad S = 108$$

Load on C Column

$$\text{from top 3 floors, } \left(\frac{27}{12}\right)(2960)(20)(3) = 66840 \text{ lbs.}$$

$$\text{from 1st floor, } S_1, \left(\frac{150}{20}\right)(2960)(20) = 26840 \text{ lbs.}$$

$$\text{direct load on column} = 112480 \text{ lbs.}$$

Exact method:

$$K_2 C_2 = K_1 C_1 = (4)(285) = 1020$$

$$a_1 = - \frac{1^2}{12} \left[\frac{W_1(K_2 C_2 - 25)}{(K_c)^2 + 10K_2 C_2 + 20C^2} - \frac{2W_2 S}{12} \right]$$

$$= - \frac{(20)^2(12)^2}{12} \left[\frac{(2960)(1020 - 25)}{(1020)^2 + (10)(1020)(108) + (20)(108)^2} - \frac{(2)(2960)(108)}{(20)(108)^2} \right] = - 312$$

Stress due to bending.

$$\frac{Mx_1}{I} = \frac{48000 \times 1}{1} = \frac{(4)(2800)(10)}{(8)(12)} = 302.2 \text{ lbs. sq. in.}$$

Equivalent concrete area 339.7 sq. in, if $k_2 C_2 = k_1 C_1$

Stress due to compression, $\frac{112400}{339.7} = 211.8 \text{ lbs. sq. in.}$

Maximum stress $302.2 + 211.8 = 514.1 \text{ lbs. sq. in.}$

Approximate method.

$$R = \frac{W_1}{2} = \frac{(2800)(20)}{2} = 28000 \text{ lbs.}$$

Total load on C column $(4)(28000) = 112000 \text{ lbs.}$

$$k = 6, kC = 1800$$

$$\alpha_1 = - \frac{(20)^2(12)^2}{12 \times 1} \left[\frac{(2800)(1800+572)}{(12)[(1800)^2 + (10)(1800)(102) + (20)(102)^2]} - \frac{(2800)(102)}{(12)(1800)(102) + (20)(102)^2} \right]$$

$$= - \frac{614}{1}$$

Stress due to bending,

$$\frac{Mx_1}{I} = \frac{(112000)(10)}{(8)(12)} = 341.1 \text{ lbs. sq. in.}$$

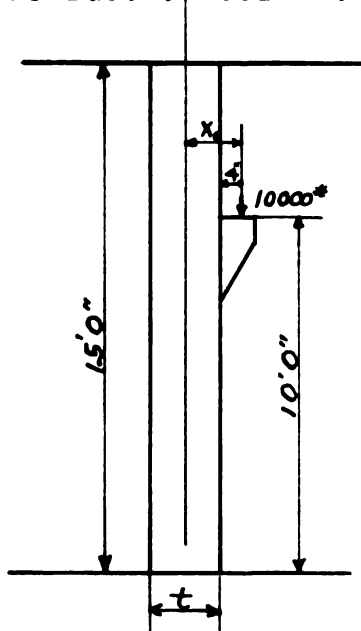
Stress due to compression 211.8 lbs. sq. in.

Maximum stress 552.9 lbs. sq. in.

Approximate method gives higher values for stresses and is therefore on the side of safety.

Design 24.

Design a rectangular column 12 feet long to have fixed ends and support a 10000 lb. load on a bracket placed at two thirds the height of the column from the base. The load must be placed 4" from the face of column.



Allowable axial compression 22.8 per cent compression strength, or 400 lbs. sq. in.

Use 1 per cent reinforcement. $1 = 12 \text{ dia.}$

$$\frac{(12)(12)}{12} = 12 \text{ " side of column, } c = 16 \text{ "}$$

$$12 + 2 + 2 = 16 \text{ " overall, use } 20 \text{ " for } t$$

$$A = 12 \times 16 = 192 \text{ sq. in.}$$

$$a_s \neq 16 \text{ } 192 = 1.92 \text{ sq. in. steel}$$

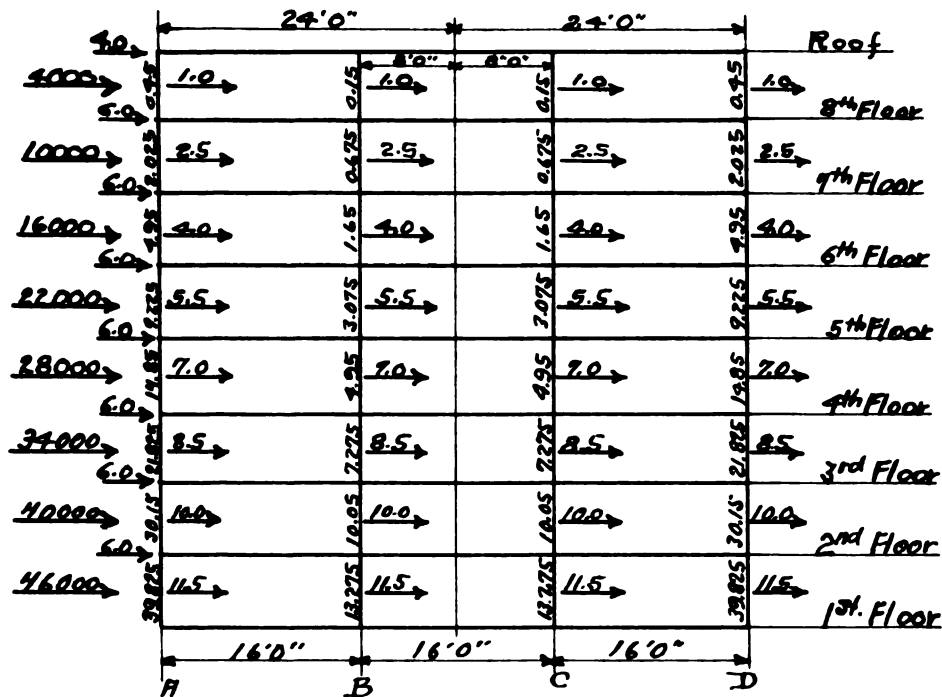
$$p_o = \frac{a_s}{A} = \frac{1.92}{192} = 0.01$$

$$\frac{x_o}{t} = \frac{14}{20} = 0.7, k = 0.87, \quad \lambda = 0.108$$

$$f_c = \frac{Px_o}{At^2} = \frac{(10000)(14)}{(0.108)(12)(20)^2} = 408.2 \text{ lbs. sq. in.}$$

Design 20.

Compute moments and all direct stresses in all members in frame given below. 12' ceilings.



Stresses in
hundreds of pounds.

Let ax be stress in B and C columns,
 $24x$ be stress in A and D columns.

Direct stress in $8A$, $8C$, $8D$, $8B$.

$$(4000)(8) = (24x)(24+24) + (ax)(8+8)$$

$$28000 = ax(16+16)$$

$$ax = 100 \text{ lbs. stress in } 8B, 8D$$

$$24x = 400 \text{ lbs. stress in } 8A, 8C$$

Direct stress in $7A$, $7B$, $7C$, $7D$,

$$(10000)(16) + (4000)(8) = (24x)(24+24) + (ax)(8+8)$$

$$160000 = (ax)(16+16)$$

$$ax = 5000 \text{ lbs. stress in } 7B, 7D$$

$$24x = 20000 \text{ lbs. stress in } 7A, 7C$$

Direct stress in $6A$, $6B$, $6C$, $6D$

$$(4000)(32) + (10000)(16) + (4000)(8) = (24x)(24+24) + (ax)(8+8)$$

$$280000 = (ax)(16+16)$$

$$ax = 10000 \text{ lbs. stress in } 6B, 6D$$

$$24x = 40000 \text{ lbs. stress in } 6A, 6C$$

Direct stress in 3A, 3B, 3C, 3D,

$$(4000)(42) + (6000)(20+15+5) = (24x)(24+24) + (5x)(5+5)$$

$$482000 = (5x)(100)$$

$$5x = 4820 \text{ lbs. stress in 3B, 3C}$$

$$24x = 8228 \text{ lbs. stress in 3A, 3D.}$$

Direct stress in 4A, 4B, 4C, 4D

$$(4000)(24) + (6000)(42+20+15+5) = (24x)(24+24) + (5x)(5+5)$$

$$782000 = (5x)(100)$$

$$5x = 7820 \text{ lbs. stress in 4B, 4C,}$$

$$24x = 14520 \text{ lbs. stress in 4A, 4D}$$

Direct stress in 5A, 5B, 5C, 5D

$$(4000)(26) + (6000)(24+42+20+15+5) = (24x)(24+24) + (5x)(5+5)$$

$$1184000 = (5x)(100)$$

$$5x = 11840 \text{ lbs. stress in 5B, 5C}$$

$$24x = 21528 \text{ lbs. stress in 5A, 5D}$$

Direct stress in 2A, 2B, 2C, 2D

$$(4000)(75) + (6000)(26+24+42+20+15+5) = (24x)(24+24) + 5x(5+5)$$

$$1808000 = (5x)(100)$$

$$5x = 18080 \text{ lbs stress in 2B, 2C}$$

$$24x = 36160 \text{ lbs. stress in 2A, 2D}$$

Direct stress in 1A, 1B, 1C, 1D

$$(4000)(80) + (6000)(75+26+24+42+20+15+5) = (24x)(24+24) + (5x)(5+5)$$

$$2124000 = (5x)(100)$$

$$5x = 18278 \text{ lbs. stress in 1B, 1C}$$

$$24x = 36228 \text{ lbs stress in 1A, 1D}$$

The horizontal shear on any column is the sum of all the wind loads above the floor line of that column, equally distributed among the columns on that floor.

Moments.

Roof girders.

$$M_1 = M_6 = (1000)5 = 5000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 5000 + (-450)15 = -1200 \text{ ft. lbs.}$$

$$M_3 = M_4 = -1200 + 5000 = 4800 \text{ ft. lbs.}$$

5th floor girders.

$$M_1 = M_6 = (1000)5 + (2000)5 = 21000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 21000 - (2025-450)15 = -4200 \text{ ft. lbs.}$$

$$M_3 = M_4 = -4200 + 21000 = 16800 \text{ ft. lbs.}$$

7th floor girders.

$$M_1 = M_6 = (2000)5 + (4000)5 = 30000 \text{ ft. lbs.}$$

$$M_2 + M_5 = 30000 - (4800-2025)15 = -7500 \text{ ft. lbs.}$$

$$M_3 = M_4 = -7500 + 30000 = 22500 \text{ ft. lbs.}$$

8th floor girders.

$$M_1 = M_6 = (4000)6 + (8500)6 = 87000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 87000 - (8225-4850)16 = -11400 \text{ ft. lbs.}$$

$$M_3 = M_8 = -11400 + 87000 = 45600 \text{ ft. lbs.}$$

5th floor girders.

$$M_1 = M_6 = (8500)6 + (7000)6 = 78000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 78000 - (14550-8225)16 = -18000 \text{ ft. lbs.}$$

$$M_3 = M_4 = -18000 + 78000 = 60000 \text{ ft. lbs.}$$

4th floor girders.

$$M_1 = M_6 = (7000)6 + (8500)6 = 88000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 88000 - (21525-14550)16 = -18600 \text{ ft. lbs.}$$

$$M_3 = M_4 = -18600 + 88000 = 74400 \text{ ft. lbs.}$$

3rd floor girders.

$$M_1 = M_6 = (8500)6 + (10000)6 = 111000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 111000 - (20150-21525)16 = -20600 \text{ ft. lbs.}$$

$$M_3 = M_4 = -20600 + 111000 = 90400 \text{ ft. lbs.}$$

2nd floor girders.

$$M_1 = M_6 = (10000)6 + (11200)6 = 128000 \text{ ft. lbs.}$$

$$M_2 = M_5 = 128000 - (32825-20150)16 = -25600 \text{ ft. lbs.}$$

$$M_3 = M_4 = -25600 + 128000 = 102400 \text{ ft. lbs.}$$

Compression in floor girders.

$$6000 - 1500 = 4500 \text{ lbs. A to B}$$

$$4500 - 1500 = 3000 \text{ lbs. B to C}$$

$$3000 - 1500 = 1500 \text{ lbs. C to D}$$

Roof girders.

$$4000 - 1000 = 3000 \text{ lbs. A to B}$$

$$3000 - 1000 = 2000 \text{ lbs. B to C}$$

$$2000 - 1000 = 1000 \text{ lbs. C to D.}$$

the first of these is the fact that the system is not a simple one, but a complex one, in which the various parts are interrelated and interdependent. The second is that the system is not a static one, but a dynamic one, in which the parts are constantly changing and evolving. The third is that the system is not a closed one, but an open one, in which the parts are constantly interacting with the environment. The fourth is that the system is not a linear one, but a non-linear one, in which the parts are constantly interacting with each other in a non-linear fashion. The fifth is that the system is not a deterministic one, but a probabilistic one, in which the parts are constantly interacting with each other in a probabilistic fashion. The sixth is that the system is not a simple one, but a complex one, in which the parts are interrelated and interdependent. The seventh is that the system is not a static one, but a dynamic one, in which the parts are constantly changing and evolving. The eighth is that the system is not a closed one, but an open one, in which the parts are constantly interacting with the environment. The ninth is that the system is not a linear one, but a non-linear one, in which the parts are constantly interacting with each other in a non-linear fashion. The tenth is that the system is not a deterministic one, but a probabilistic one, in which the parts are constantly interacting with each other in a probabilistic fashion.

DESIGN 26.

Develope all data on beams, girders, columns and footings exclusive of those at stair and elevator shafts.

Working stresses;

Axial compression on columns reinforced with longitudinal steel and bands, 500 lbs. per sq. in.

Concrete in compression in slabs, beams, and girders, 650 lbs. per sq. in.

Concrete in compression over supports of continuous slabs, beams and girders, 750 lbs. per sq. in.

Shearing of concrete as measured by diagonal shear, 40 lbs. per sq. in.

Shear of concrete with proper reinforcement, 120 lbs. sq. in.

Bond stress between concrete and plain rods, 80 lbs. sq. in.

Tensile stress in reinforcing steel, 16000 lbs. sq. in.

BUILDING;

3 story and basement concrete building.

Width c-c wall columns,	147' 0"
Length c-c end columns,	147' 0"
Span of slabs,	7' 0"
Span of beams,	21' 0"
Span of girders,	21' 0"
Floor to floor,	13' 0"
Basement,	11' 0"
Live load on roof,	30 lbs. sq. ft.
Live load on floor,	250 lbs. sq. ft.

Ratio of unit cost of steel in place to unit cost of concrete in place,	r = 60
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n = 45

ROOF SLAB:

Loads,

Five ply felt and gravel,	9 lbs.
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2 ft. cinder fill	90 lbs.
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2" concrete surfacing	25 lbs.
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Snow load	30 lbs.
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Total	151 lbs. sq. ft.
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Assume a 3 $\frac{1}{2}$ " slab with d = 2 $\frac{1}{4}$ "

This will carry $(165)(1.2) = 198$ lbs. per sq. ft.

Slab load, $151 + 44 = 195$ lbs. per sq. ft.

$$M = \frac{wl^2}{12} = \frac{(195)(7)^2(12)}{12} = 9555 \text{ in. pds. bending.}$$

$$p = 0.0077, \quad k = 0.378, \quad j = 0.874, \quad K = 107.4$$

$$d = \sqrt{\frac{9555}{(107.4)(12)}} = 2.69", \text{ use } 2\frac{3}{4}"$$

$$\text{add } \frac{1}{4}" \text{ for insulation, } 2\frac{3}{4}" + \frac{1}{4}" = 3\frac{1}{2}"$$

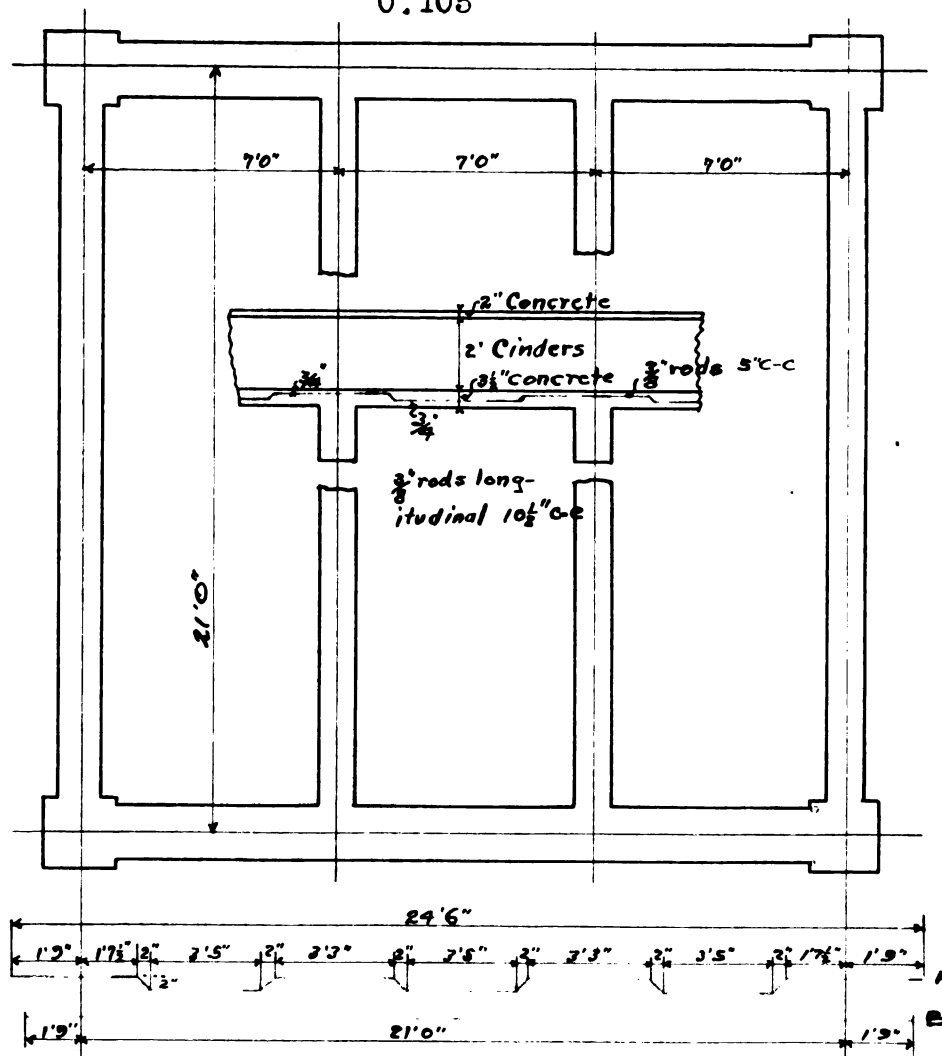
$$a_s = \frac{9555}{(16000)(0.874)(2.75)} = 0.248 \text{ sq. in., use } 3/8" \text{ round rods}$$

$$\frac{0.1104}{0.248} \times 12 = 5.36", \text{ use } 5" \text{ c-c spacing.}$$

bend each alternate rod up at the quarter point over support to provide for negative bending; extend thru to quarter point of next span.

$$(0.0024)(3.5)(12) = 0.105 \text{ sq. in. traverse reinforcing.}$$

$$\text{use } 3/8" \text{ rods spaced } \frac{0.1104}{0.105} \times 12 = 10.5" \text{ c-c}$$



Alternate Rods A & B, 5" c-c

ROOF BEAMS.

B₁ Assumed load per ft. of length, 175 lbs. for stem
 -- Total load (195)(7) + 175 = 1540 lbs. per ft. length.

$$V = \frac{(1540)(21)}{2} = 16170 \text{ lbs.}$$

$$M = \frac{(1540)(21) \cdot (12)}{12} = 679,140 \text{ in. lbs.}$$

$$A = \frac{16170}{105} = 153 \text{ sq. in. for shear.}$$

$$\text{Economical depth, } d = \sqrt{\frac{rM}{f_s b'}} + \frac{t}{2}$$

b'	d	b'xd	Depth for shear
8	19.6	156.8	19.1
9	18.6	167.4	17.0
10	17.7	177.0	15.3
11	16.9	185.9	13.9

Use b' = 10", d = 18", total depth 20"

$$\text{Weight of stem } \frac{(10)(16.5)(150)}{144} = 171.8 \text{ lbs. per ft.}$$

$$B = 10 + (8)(3.5) = 38 \text{ inches.}$$

$$\frac{M}{bd^2} = \frac{679140}{(38)(18)^2} = 55.16$$

$$\frac{t}{d} = \frac{3.5}{18} = 0.194$$

$$f_c = 650 \text{ lbs.}$$

$$p = 0.002 + \frac{25.46}{29.10}(0.002) = 0.0037 \quad j = 0.9186$$

$$a_s = \frac{679140}{(16000)(0.9186)(18)} = 2.56 \text{ sq. in.}$$

use 5, 13/16" round rods.

$$u = \frac{16170}{(5)(2.553)(0.92)(18)} = 76.4 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{2}{18} = 0.1111 \quad p' = \frac{2.56}{(10)(18)} = 0.0142$$

$$L = 0.318$$

$$K = 0.0133$$

$$f_c = \frac{679140}{(10)(18)^2(0.318)} = 659 \text{ lbs.}$$

$$f_s = \frac{679140}{(19)(18)^2(0.0133)} = 15756 \text{ lbs.}$$

$$x^2 = \frac{21}{2} \left[1 - \sqrt{\frac{(8)(2)}{(12)(5)}} \right] 12 = 60.4", \text{ bend up two.}$$

$$(0.0142)(18) = 0.2556, \text{ use } 3/8" \text{ stirrups}$$

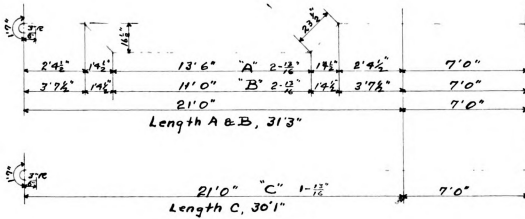
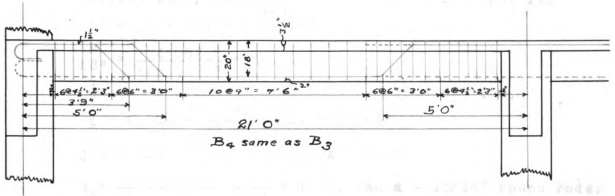
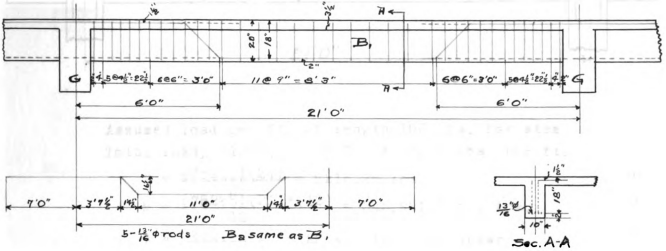
$$s = \frac{(3)(0.11)(1)(16000)(0.85)(18)}{(2)(16170)} = 4.9", \text{ use } 4\frac{1}{2}" \text{ spacing for stirrups.}$$

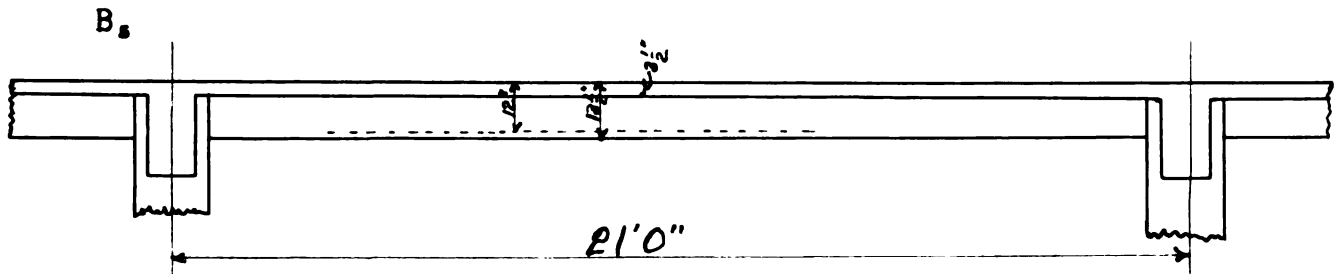
$$\text{at } \frac{1}{8}, s = (4/3)(4.5) = 6"$$

$$\text{at } \frac{1}{4}, s = (2)(4.5) = 9"$$

Bonding length

$$\frac{(659)(10)(0.875)}{(4)(80)} = 18" \text{ embedment.}$$





Assumed load per ft. of length 100 lbs. for stem
 Total load, $(195)(3.5) + 100 = 782.5$ lbs. per ft.

$$V = \frac{(782.5)(21)}{2} = 8216.25 \text{ lbs.}$$

$$M = \frac{(782.5)(21)^2(12)}{12} = 345,982.5 \text{ in. lbs.}$$

$$A = \frac{8216.25}{105} = 78.25 \text{ sq. in. for shear}$$

$$b = 10", \quad d = 12", \quad \text{total } d = 13.5"$$

$$\text{weight stem, } \frac{(10)(10.6)(150)}{144} = 103.0 \text{ lbs. per ft.}$$

$$b' = 10 + (8)(3.5) = 38"$$

$$\frac{M}{bd^2} = \frac{345082.5}{(38)(12)^2} = 64.52$$

$$\frac{t}{d} = \frac{3.5}{12} = 0.29$$

$$f_c = 650 \text{ lbs.}$$

$$p = 0.004 + \frac{6.7}{27.5}(0.002) = 0.0045$$

$$j = 0.900$$

$$k = 0.35$$

$$a_s = \frac{345082.5}{(16000)(0.9)(12)} = 1.98, \text{ use } 4 - 13/16" \text{ round rods.}$$

$$u = \frac{8216.25}{(4)(2.553)(0.9)(12)} = 74.5 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{1.5}{12} = 0.125$$

$$p = \frac{2.074}{(19)(12)} = 0.0173$$

$$L = 0.332$$

$$K = 0.0151$$

$$f_s = \frac{345082.5}{(10)(12)^2(0.0151)} = 15870 \text{ lbs.}$$

$$f_c = \frac{345082.5}{(10)(12)^2(0.322)} = 722 \text{ lbs.}$$

$$x_2 = \frac{21}{2} \left(1 - \sqrt{\frac{(8)(2)}{(12)(4)}} \right) = 53", \quad 4' 5"$$

use 3/8" stirrups spaced 4 1/2" to 1/8, 6" to 1/4, 9" at center

$$\frac{(722)(10)(0.9)}{(4)(80)} = 20.3" \text{ embedment for bond.}$$

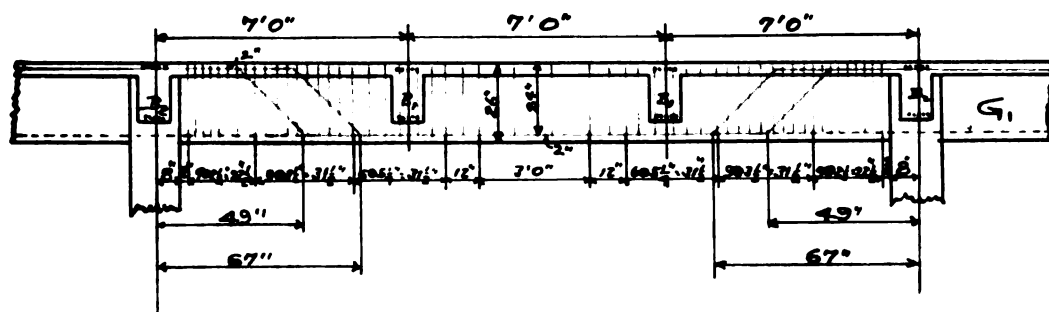
B₆, same dimensions as B₅.



B₇, B₈, B₉, B₁₀ are at stairway and elevator shaft.

GIRDERS

G₁



Assumed load of girder, 350 lbs. per linear foot.

Reaction of beam loads (2)(16170) = 32340 lbs.

Equivalent uniform loading on girder

$$\frac{(32340)(3)}{21} + 350 = 4970 \text{ lbs. per ft.}$$

$$M = \frac{wl^2}{12} = \frac{(4970)(21)^2(12)}{12} = 2,191,770 \text{ in lbs.}$$

$$\begin{array}{r} \text{less 10 per cent} \\ \text{acting moment} \end{array} \quad \begin{array}{r} 219,177 \\ 1,972,593 \end{array} \text{ in lbs.}$$

$$V = (32340) + (10.5)(350) = 36015 \text{ lbs.}$$

$$A = \frac{36015}{105} = 343 \text{ sq. in. for shear}$$

$$bd = 343 \text{ sq. in., let } b = 5/8d \text{ then } \frac{5d^2}{8} = 343 \text{ and } d = 23.4''$$

$$\text{use } d = 24'', \text{ total } d = 26'', \text{ then } b = 15'', b' = 43.0$$

$$\frac{M}{bd^2} = \frac{1972593}{(43)(24)^2} = 79.6$$

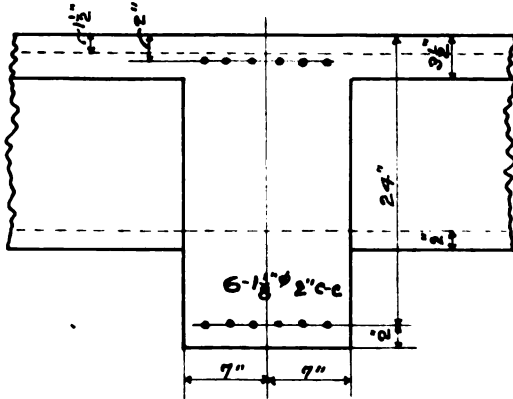
$$\frac{t}{d} = \frac{3.5}{24} = 0.146 \quad f_c = 725 \quad j = 0.933$$

$$a_s = \frac{1972593}{(16000)(0.983)(24)} = 5.6 \text{ sq. in.}$$

use 6 - 1 $\frac{1}{8}$ " rods, area 5.96 sq. in.

$$u = \frac{36015}{(6)(3.534)(0.933)(24)} = 74.9 \text{ lbs.}$$

$$\text{wgt. girder, } \frac{(14)(22.5)(150)}{144} = 329 \text{ lbs.}$$



$$\frac{d'}{d} = \frac{2}{24} = 0.0833$$

$$p = \frac{5.96}{(14)(24)} = 0.0177$$

$$p' = \frac{p}{2} = \frac{0.0177}{2} = 0.0088$$

$$L = 0.291 \quad K = 0.0155$$

$$f_c = \frac{1972593}{(14)(24)^2(0.291)} = 747.8 \text{ lbs./sq.in.}$$

$$f_s = \frac{1972593}{(14)(24)^2(0.0155)} = 14040 \text{ lbs./sq.in.}$$

$$\frac{V}{jd} = \frac{36015}{(0.85)(24)} = 1765.4 \text{ lbs. per linear inch.}$$

at the one third point

$$\frac{V}{jd} = \frac{36015 - (350)(7)}{(0.95)(24)} = 1472.1 \text{ lbs. linear inch.}$$

diagonal tension taken by four bent rods

$$\frac{(1765.4 + 1472.1)}{2} (7)(14)(0.666) = 105755 \text{ lbs.}$$

tensile value of four rods

$$\frac{(4)(0.994)(16000)}{0.7} = 90880 \text{ lbs.}$$

$$105755 - 90880 = 14875 \text{ lbs taken by stirrups}$$

$$\frac{21}{2} - \frac{(40)(14)(0.95)(24)}{4970} = 7.0', \text{ use 96" stirrups needed}$$

$$\frac{21}{2} (1 - \sqrt{\frac{(8)(2)}{(12)(6)}}) 12 = 66.8 \text{ inches, bend two rods}$$

$$\frac{21}{2} (1 - \sqrt{\frac{(8)(4)}{(12)(6)}}) 12 = 41.6 \text{ inches, bend two more rods.}$$

$$66.8 - 41.6 = 25.2", \text{ to large, must not be over 18", bend two at 49".}$$

a_s of stirrups, $(0.012)(24) = 0.288$, use 7/16 inch rods.

$$s = \frac{(3)(0.15)(2)(16000)(0.85)(24.5)}{(2)(36015)} = 2.7"$$

$$\text{at } 1/8, s = (4/3)(2.7) = 3.6"$$

$$\text{at } 1/4, s = (2)(2.7) = 5.4"$$

$$\text{length for bond } \frac{(748)(15)(0.875)}{(4)(80)} = 31"$$

At support, $V = \frac{19320}{jd (0.915)(18)} = 1173$ lbs. linear inch
at the one third point

$$\frac{V}{j'd} = \frac{19320 - 2100}{(0.915)(18)} = 1045.4 \text{ lbs. linear inch}$$

diagonal tension to be taken by rods.

$$\frac{1173 + 1045.4}{2} (7)(12)(0.666) = 62115.2 \text{ lbs.}$$

bend up four rods

$$\frac{(4)(0.36)(16000)}{0.7} = 33828 \text{ lbs taken by rods}$$

$$62115.2 - 33828 = 28287 \text{ lbs taken by stirrups}$$

$$(0.012)(18) = 0.216, \text{ use } 3/8" \text{ stirrups with } 0.2208 \text{ sq.in.area}$$

$$\text{spacing, } \frac{(3)(0.2208)(16000)(0.915)(18)}{(2)(19320)} = 4.5"$$

$$\text{at } 1/8, s = (2.333)(4.5) = 6", \text{ at } 1/4, s = 9"$$

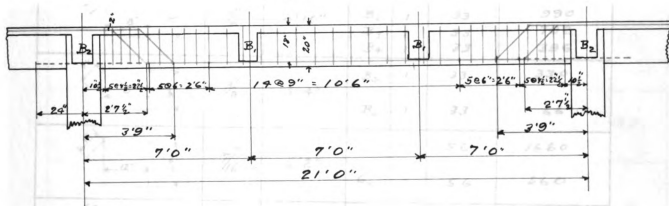
$$\text{bend up two rods at } \frac{21}{2} (1 - \sqrt{\frac{2}{8}}) = 3.75' \text{ from support}$$

$$\text{bend two more at } \frac{21}{2} (1 - \sqrt{\frac{4}{8}}) = 2.25' \text{ from support}$$

$$s = 3.75 - 2.25 = 1.5'$$

$$\text{allowable } s, 0.75d = (0.75)(18) = 13.5"$$

$$\text{length for bond, } \frac{(769)(11)(0.875)}{(4)(80)} = 23.2"$$



Girder G⁴ same as girder G³ but rods have hooks at one end.

Girder G⁵ same as girder G³

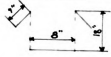
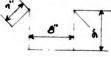
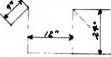
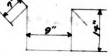
Girders G⁶, G⁷, G⁸, G⁹, G¹⁰ at stairway and elevator.

ROOF. BEAM & GIRDER DATA.

REINFORCEMENT

MK	SIZE		DEPTH--STEEL		CENTER	OVER SUPPORTS				Notes.	
	W	D	CENTER	SUPPORTS		BEND UP	$\frac{a}{b}$	$\frac{a}{b}$	Straight of Rods		Straight of Bottom
B ₁	10"	20"	18"	18½"	5 - 1⅜" ϕ rods	2	1½			3	
B ₂	10"	20"	18"	18½"	5 - "	2	1½			3	
B ₃	10"	20"	18"	18½"	5 - "	4	1½			3 inner 1 outer	hooks
B ₄	10"	20"	18"	18½"	5 - "	4	1½			3 inner 1 outer	hooks
B ₅	10"	13½"	12"	12"	4 - "	2	1½			2	
B ₆	10"	13½"	12"	12"	4 - "	2	1½			2	hooks
G ₁	14"	26"	24"	24"	6 - 1⅝" ϕ rods	4	2			2	
G ₂	14"	26"	24"	24"	6 - "	4	2			2	hooks
G ₃	11"	20"	18"	18"	8 - 1⅞" ϕ rods	4	2			4	
G ₄	11"	20"	18"	18"	8 - "	4	2			4	hooks
G ₅	11"	20"	18"	18"	8 - "	4	2			4	

STIRRUP SCHEDULE

BENDS	SIZE	LENGTH	B or G		NO. EACH B or G	TOTAL NUMBER
			MK	No.		
	3/8"	4'10"	B ₁	1	33	2310
			B ₂	1	33	990
			B ₃	1	33	792
			B ₄	1	33	396
	3/8"	3'4"	B ₅	1	33	330
			B ₆	1	33	66
	7/16"	6'2"	G ₁	1	56	1680
			G ₂	1	56	560
	3/8"	4'3"	G ₃	1	35	315
	3/8"	4'3"	G ₄	1	35	70

ROOF BEAM & GIRDER

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BENDS	SIZE	LENGTH	For Beam or Girder		Number per Bdg	Total Wanted
			Mk.	No.		
	$\frac{13}{16}$	34'1 1/2"	B ₁ B ₂	1 1	2 2	140 60
	$\frac{13}{16}$	35'0"	B ₁ B ₂	1 1	3 3	210 90
	$\frac{13}{16}$	31'2 1/2"	B ₃ B ₄	1 1	2 2	48 24
	$\frac{13}{16}$	31'2 1/2"	B ₃ B ₄	1 1	2 2	48 24
	$\frac{13}{16}$	30'1"	B ₃ B ₄	1 1	1 1	24 12
	$\frac{13}{16}$	35'8 1/2"	B ₅	1	2	20
	$\frac{13}{16}$	35'0"	B ₅	1	2	20
	$\frac{13}{16}$	30'9 1/2"	B ₆	1	2	4
	$\frac{13}{16}$	30'1"	B ₆	1	2	4
	$\frac{1 1}{8}$	36'6"	G ₁	1	2	60

Bends	Size	Length	For beam or girder		Number for A or G	Total No. Wanted
			Mk.	No.		
	1 1/8"	34'6"	G ₁	2	2	60
	1 1/8"	35'0"	G ₁	1	2	60
	1 1/8"	31'7"	G ₂	1	2	20
	1 1/8"	31'7"	G ₂	2	2	20
	1 1/8"	30'1"	G ₂	1	2	20
	1 1/16"	35'6"	G ₃	1	2	18
	1 1/16"	35'6"	G ₃	2	2	18
	1 1/16"	25'0"	G ₃	3	4	36
	1 1/16"	31'3"	G ₄	1	2	4
	1 1/16"	31'3"	G ₄	2	2	4
	1 1/16"	30'1"	G ₄	3	4	8

1st, 2nd and 3rd floor members.

Live load 250 lbs. sq. ft.

Bending moment $\frac{wl^2}{12}$

$r = 60$

1" granolithic top finish, included in 250 lb. load

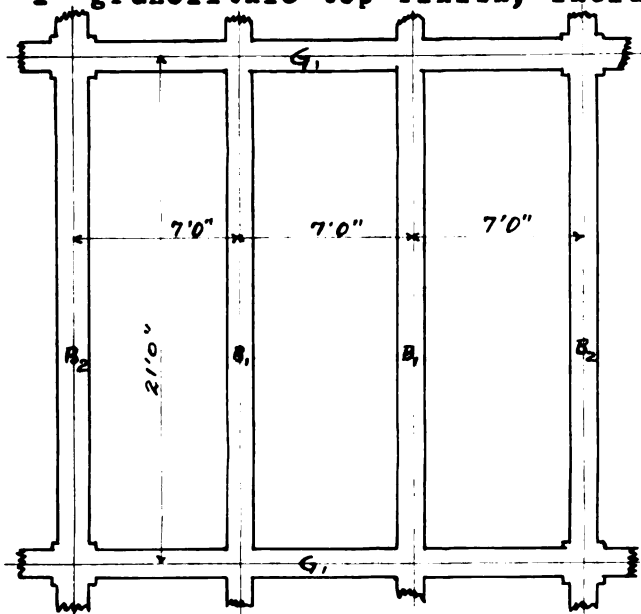
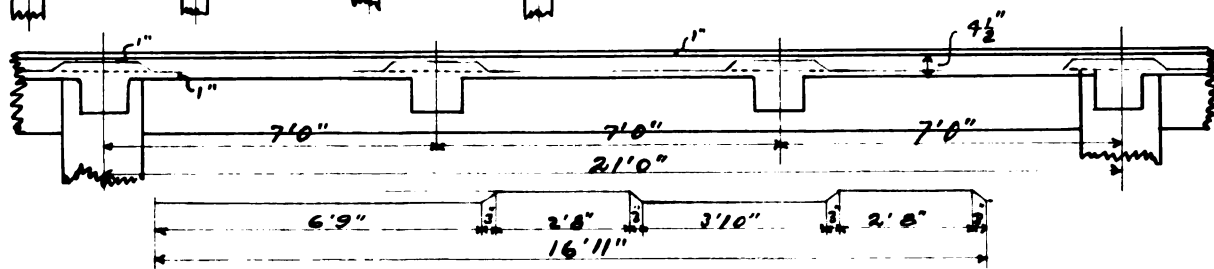


Table 6 shows 4½" slab, $d = 3½"$ span 7' will carry a live plus dead load of $(269)(1.2) = 323$ lb.sq. slab weight 56lb.

total $250 + 56 = 306$ lbs.

$a_s = 0.323$ sq. in. Table 6.

3/8" rods, 4" c-c, crosswise, Tb.4. use 4 - 3/8" rods transversely for shrinkage.

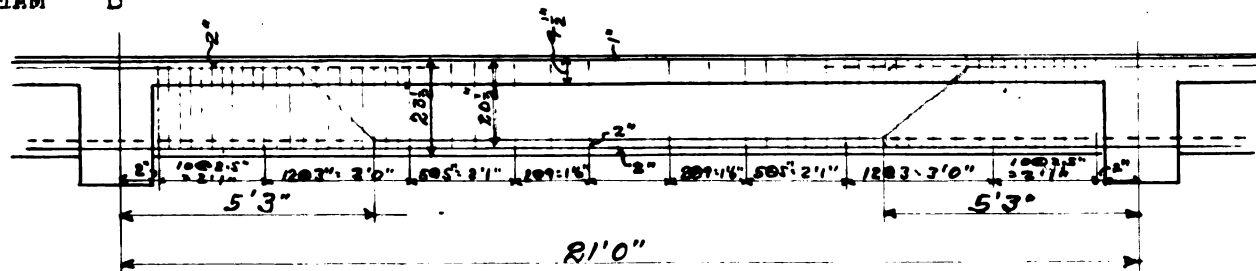


6 rods per panel, 17' 0" long

30 rods per panel, 24' 0" long

46 panels each floor, 3 floors.

BEAM B1



Load per ft of beam, $7 \times 306 = 2142$ lbs.

assumed load stem 240 lbs. per ft.

total load per ft. 2382 lbs.

Maximum shear $\frac{(2382)(21)}{2} = 25011$ lbs.

Maximum moment $\frac{(2382)(21)^2(12)}{12} = 1,050,462$ in. lbs.

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Area web for shear $\frac{25011}{106} = 238.2$ sq. in.

$b = 12"$, $d = 20.5"$, total $d = 22.5"$

wgt. stem $\frac{(12)(19)(150)}{144} = 237.5$ lbs. per ft.

$\frac{M}{bd^2} = \frac{1050462}{(48)(20.5)^2} = 52.06$ $\frac{t}{d} = \frac{4.5}{20.5} = 0.219$

from Table 9, $p = 0.002 + \frac{22.37}{28.70}(0.002) = 0.0035$

$j = 0.9159$

$a_s = (48)(20.5)(0.0035) = 3.444$ sq. in.

use 8 - $3/4"$ rods in two rows, area steel 3.53 sq. in.

bond for one rod $\frac{25011}{(2.356)(0.9159)(20.5)} = 565.4$ lbs.

number to carry straight, $\frac{565.4}{(1.5)(80)} = 4+$, run 6 thru

web reinforcement needed out to

$\frac{21}{2} - \frac{(40)(12)(0.916)(20.5)}{2380} = 6.72'$, 6' 9"

bend two rods at $\frac{21}{2} (1 - \sqrt{\frac{(2)(0.4418)}{3.53}}) = 5.25'$, 5' 3" from support.

$\frac{d'}{d} = \frac{2}{21.5} = 0.093$ $p' = \frac{3.53}{(12)(20.5)} = 0.014$, $p' = \frac{p}{2}$

from table 11, $L = 0.254$, $K = 0.012$

$f_c = \frac{1050462}{(12)(21.5)^2(0.254)} = 746$ lbs.

$f_c = \frac{1050462}{(12)(21.5)^2(0.0125)} = 15372$ lbs.

$(0.012)(20) = 0.24$, use $7/16"$ stirrups

space $\frac{(3)(0.15)(2)(16000)(0.875)(20.5)}{(2)(25011)} = 2.5"$

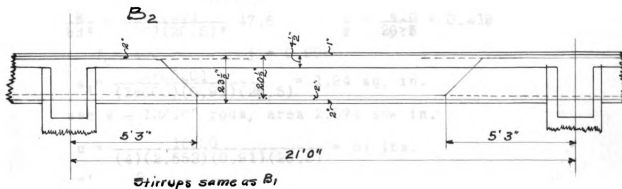
at $1/8$, $s = (0.333)(2.5) = 3.33"$

at $1/4$, $s = (2)(2.5) = 5"$

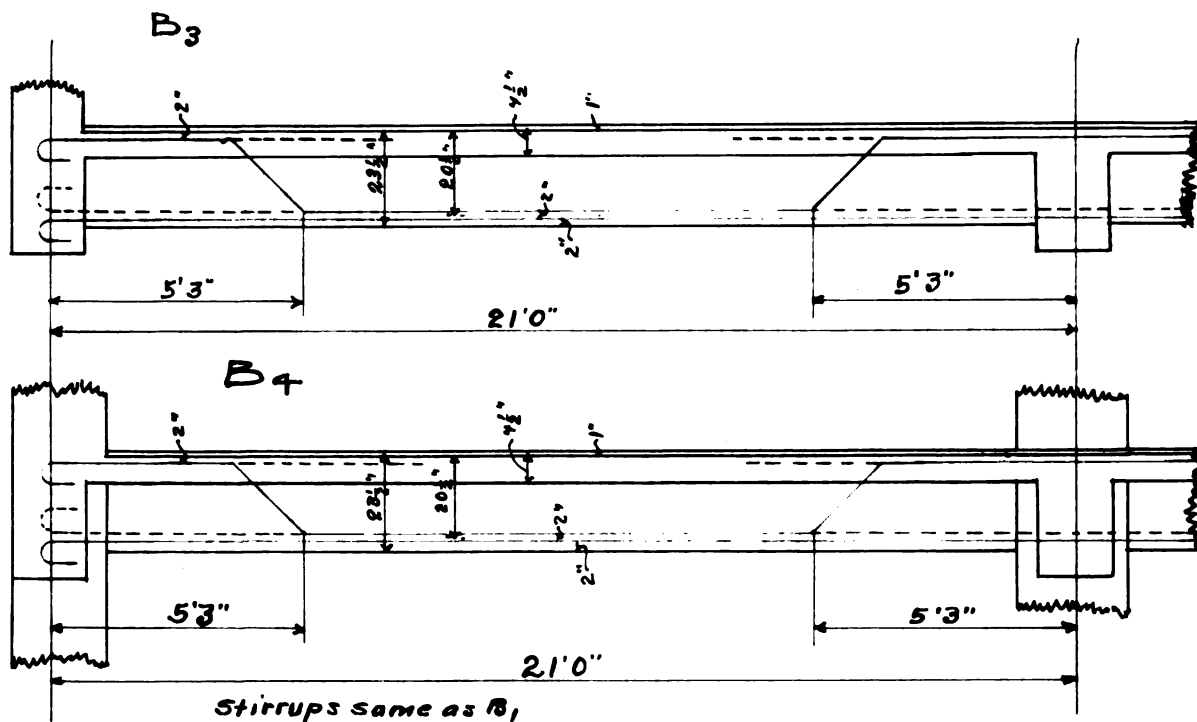
12 at $2 \times 5"$, 7 at $3"$, 6 at $5"$, rest at $9"$

length of bond for compression rods,

$\frac{(746)(15)(0.875)}{(4)(80)} = 30.6"$



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Beam B⁵

Live load from floor $\frac{7 \times 250}{2} = 875$ lbs.

wall load $(0.666)(2.83)(150) = 283$ lbs.

+ 10 per cent, window 28 lbs.

total load on stem 1186 lbs. linear foot

assumed load for beam 175 lbs.

total load acting 1361 lbs. linear foot.

$$V = \frac{(1361)(21)}{2} = 14290.5 \text{ lbs.}$$

$$M = \frac{(1361)(21)^2(12)}{12} = 600201 \text{ in lbs.}$$

$$\text{area for shear, } \frac{14291}{105} = 136.1 \text{ sq. in.}$$

use $b = 8"$, $d = 20.5"$, total $d = 22.5"$

$$\text{wgt. beam} = \frac{(8)(18)(150)}{144} = 136.6 \text{ lbs.}$$

$$b' = 12 \times 4 \times 4.5 = 30"$$

$$\frac{M}{bd^2} = \frac{600201}{(30)(20.5)^2} = 47.6 \quad \frac{t}{d} = \frac{4.5}{20.5} = 0.219$$

$$f_c = 400 \quad j = 0.905$$

$$a_s = \frac{600201}{(16000)(0.91)(20.5)} = 1.94 \text{ sq. in.}$$

use 4 = 13/16" rods, area 2.074 sq. in.

$$u = \frac{16000}{(4)(2.553)(0.91)(20.5)} = 84 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{2}{20.5} = 0.097$$

$$p = \frac{2.074}{(7)(20.5)} = 0.0126 = p'$$

$$L = 8.309 \quad K = 0.0115$$

$$f_c = \frac{600291}{(8)(20.5)(0.309)} = 578 \text{ lbs.}$$

$$f_s = \frac{600291}{(8)(20.5)^2(0.0115)} = 12480 \text{ lbs.}$$

$$\text{bend 2 rods at } \frac{21}{2} \left(1 - \sqrt{\frac{(2)(0.5185)}{2.074}} \right) = 3.15 \text{ ft.}, 37.8"$$

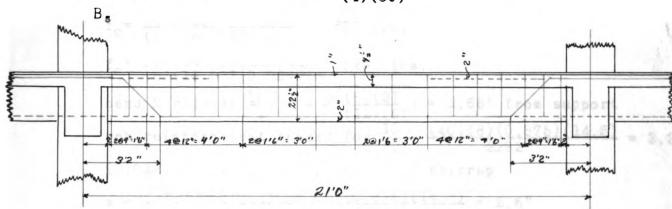
$$\text{web reinforcement needed for } \frac{21}{2} - \frac{(40)(8)(0.91)(20.5)}{1361} = 6.1'$$

$$(0.012)(20.5) = 0.246, \text{ use } 7/16" \text{ stirrups}$$

$$\text{stirrup spacing, } \frac{(3)(0.15)(2)(16000)(0.7)(20.5)}{(2)(8)(14291)} = 9" \text{ c-c}$$

$$\text{at } \frac{1}{8}, s = 4/3 \times 9 = 12", \quad \frac{1}{4}, s = (2)(9) = 1'6"$$

$$\text{length of rods for bond } \frac{(642)(15)(0.875)}{(4)(80)} = 26.3"$$



B_6 same dimensions as B_5 but with hooks at one end of steel stirrups, 2 at 9", 3 at 12", 2 at 1'6"

B_7 and B_{10} at stair and elevator shaft.

FLOOR SLAB FOR LOADING PLATFORM.

Panel 10' x 7', same as floor in building.

B_{11}

Load per ft. of beam $7 \times 306 = 2142 \text{ lbs.}$

assumed load of stem 150 lbs.

Total 2292 lbs linear foot.

$$M_{\max} \text{ shear } \frac{(2292)(10)}{2} = 11460 \text{ lbs.}$$

$$\text{Mom. } \frac{(2292)(10)^2(12)}{8} = 343600 \text{ in. lbs.}$$

$$\text{area for shear } \frac{11460}{105} = 109.1 \text{ in.}$$

$$\phi \Rightarrow 8", \quad d = 14.5", \quad d' = 17"$$

use 4 rods

$$\text{weight of stem} = \frac{(8)(17)(150)}{144} = 134.8 \text{ lbs}$$

$$b' = 8 + 8 \times 4.5 = 44"$$

$$v = \frac{11460}{(8)(14.5)} = 98.8 \text{ lbs, use web reinforcement.}$$

$$\frac{343800}{(16000)(0.875)(14.5)} = 1.69 \text{ sq. in. needed}$$

$$\text{use } 4 \text{ } 3/4" \text{ rods, } 1.77 \text{ sq. in.}$$

$$u = \frac{11460}{(4)(2.356)(0.875)(14.5)} = 107 \text{ lbs.}$$

must use more rods, say 6

$$u = \frac{11460}{(6)(2.356)(0.875)(14.5)} = 63.9 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{2.5}{14.5} = 0.172, \quad p' = \frac{1.77}{(8)(14.5)} = 0.015 = \frac{p}{2}$$

4 at top, 8 at bottom.

$$L = 0.339 \quad K = 0.0248$$

$$f_c = \frac{343800}{(8)(14.5)(0.339)} = 603 \text{ lbs.}$$

$$f_c = \frac{343800}{(8)(14.5)(0.0248)} = 8243 \text{ lbs.}$$

$$\text{bend 2 rods at } \frac{10}{2} \left(1 - \sqrt{\frac{(8)(2)}{(12)(6)}} \right) = 2.65' \text{ from support}$$

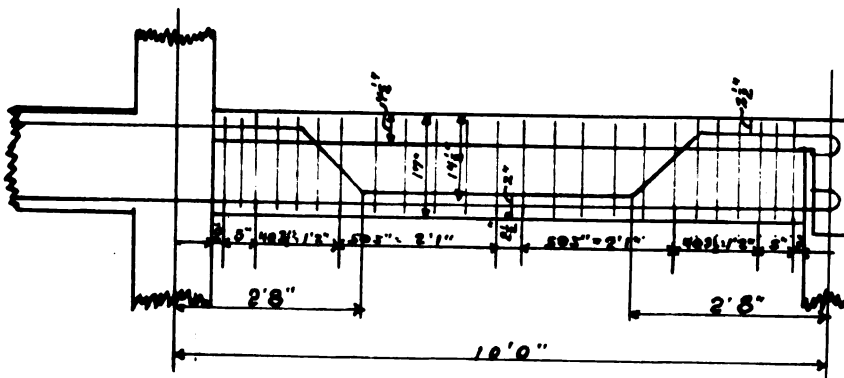
$$\text{web reinforcement needed for } \frac{10}{2} - \frac{(40)(8)(0.875)(14.5)}{2292} = 3.3'$$

$$(0.012)(14.5) = 0.174", \text{ use } 1/4" \text{ stirrup}$$

$$s = \frac{(8)(0.0491)(2)(16000)(0.875)(14.5)}{(2)(11460)} = 2.6"$$

$$\text{at } \frac{1}{8}, (4/8)(2.6) = 1.3", \quad \frac{1}{4}, (2)(2.6) = 5.2"$$

$$\text{length for bond, } \frac{(603)(14)(0.75)}{(4)(80)} = 21.2"$$



E₁₂ same as B₁₁ put hooks on steel at both ends

E₁₃ same as B₁₁

G₁, floors

Reaction of concentrated loads, $2 \times 25011 = 50022$ lbs.

Assumed dead load of stem 600 lbs.

Equivalent loading per ft. length $\frac{(50022)(3)}{21} + 600 = 7746$ lbs

$V = 50022 + (10.5)(600) = 56322$ lbs.

$M = \frac{(7746)(21)^2(12)}{12} = 3,415,988$

less 10 per cent 341,599

active moment 3,074,387 in lbs.

$\frac{56322}{105} = 536.4$ sq. in. area needed

$t = 4.5"$ $r = 60$

$b = 18"$, $d = 30"$ $bd = 540$ sq. in. $d' = 32$

wgt. of stem $\frac{(18)(32)(150)}{144} = 600$ lbs.

$b' = 18 + (8)(4.5) = 54"$

$\frac{M}{bd^2} = \frac{3074387}{(54)(30)^2} = 63.2$, $\frac{t}{d} = \frac{4.5}{30} = 0.15$

$f_c = 625$ $j = 0.938$

$a_s = \frac{3074387}{(16000)(0.938)(30)} = 6828$ sq. in.

use 10 - 15/16" round rods, 6.9 sq. in.

$u = \frac{56322}{(10)(2.945)(0.938)(30)} = 67.0$ lbs.

$\frac{a'}{d} = \frac{1}{30} = 0.051$ $p = \frac{6.9}{(18)(30)} = 0.0127 = p'$

$L = 0.205$ $K = 0.0116$

$f_c = \frac{3074387}{(18)(20)^2(0.308)} = 16.1$ lbs.

$f_c = \frac{3074387}{(18)(30)^2(0.0116)} = 16360$ lbs.

at support $\frac{V}{jd} = \frac{56322}{(0.938)(30)} = 2001.4$ lbs linear inch

at 3rd point. $\frac{V}{jd} = \frac{56322}{(0.938)(30)} = 1852.2$ lbs.

diagonal tensiokn to be taken by rods,

$\frac{(2001.4 + 1852.2)(17)(12)(0.666)}{2} = 107900.8$ lns.

value of 4 rods $\frac{(4)(0.69)(16000)}{0.7} = 63085.7$ lbs.

2. The second section focuses on the role of technology in modern business operations. It highlights how digital tools and software can streamline processes, reduce errors, and improve overall efficiency. The author argues that embracing technology is not just a competitive advantage but a necessity for staying relevant in today's fast-paced market. Examples of various software solutions and their benefits are provided to illustrate this point.

3. The third part of the document addresses the challenges of managing a diverse workforce. It discusses the importance of effective communication and collaboration across different departments and cultures. The text offers practical advice on how to foster a positive work environment, resolve conflicts, and ensure that all team members are aligned with the organization's goals. It also touches upon the need for continuous training and development to keep the workforce up-to-date with the latest industry trends.

4. The fourth section explores the impact of external factors on business performance. It examines how economic conditions, market fluctuations, and regulatory changes can affect an organization's bottom line. The author provides strategies for risk management and contingency planning to help businesses navigate these uncertainties. It stresses the importance of staying informed about the external environment and being prepared to adapt quickly to any changes.

107900.8 - 63085.7 = 44815.1 lbs to be taken by stirrups.
 $(0.012)(30) = 0.36$, use 3/8" rods bent.

$$\text{Spacing } \frac{(3)(4)(0.11)(0.938)(30)(16000)}{(2)(56322)} = 5.2"$$

use double loops

$$\text{at } \frac{1}{8}, \frac{4}{3} \times 5.2 = 6.0" \quad \text{and at } \frac{1}{4}, (2)(5.2) = 10.4"$$

$$\text{bend up two rods at } \frac{21}{2} \left(1 - \sqrt{\frac{(8)(2)}{(12)(10)}}\right) = 6.72', 6'8\frac{1}{2}"$$

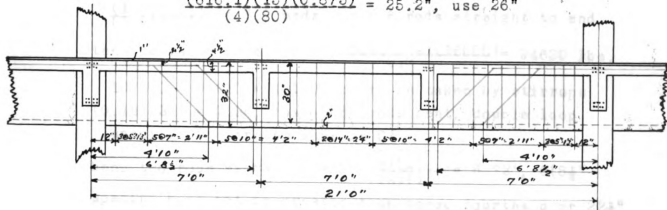
$$\text{bend two more at } \frac{21}{2} \left(1 - \sqrt{\frac{(8)(4)}{(12)(16)}}\right) = 3.15', 3'2"$$

rods must not be bent at a greater distance than $\frac{3d}{4}$ or $22\frac{1}{2}"$
 web reinforcing needed to

$$\frac{21}{2} - \frac{(40)(16)(0.938)(30)}{7746} = 7.9' \text{ from support}$$

length of compression rods for bond

$$\frac{(616.1)(15)(0.875)}{(4)(80)} = 25.2", \text{ use } 26"$$



Rein. G_2 same as G_1 but long rods have hooks at one end.

Equivalent loading per ft. length G_2 7746lbs.

$$V = 56322$$

$$M = \frac{(7746)(21)^2(12)}{16} \quad 9.9 = 3,689,245 \text{ in lbs.}$$

area for shear 536.4 sq. in.

$$b = 18", \quad d = 30", \quad d' = 32", \quad bd = 540 \text{ sq. in.}$$

weight of stem 600 lbs. $c = 64"$

$$\frac{M}{bd^2} = \frac{3689245}{(54)(30)^2} = 78.9$$

$$\frac{f}{d} = \frac{4.5}{30} = 0.15 \quad f_c = 725 \quad j = 0.937$$

$$a_s = \frac{3689245}{(16000)(0.937)(30)} = 8.203 \text{ sq. in.}$$

use 12 - 15/16" round rods at 8.28 sq. in.

$$u = \frac{56322}{(12)(2.945)(0.937)(30)} = 56.7 \text{ lbs}$$

$$\frac{d'}{d} = \frac{1.5}{30} = 0.050$$

$$p = \frac{8.28}{(18)(30)} = 0.0153 = p'$$

$$L = 0.348$$

$$K = 0.0141$$

$$f_c = \frac{3689245}{(18)(30)^2(0.348)} = 654.4 \text{ lbs.}$$

$$f_s = \frac{3689245}{(18)(30)^2(0.0141)} = 16151 \text{ lbs}$$

$$\text{at support } \frac{V}{j d} = 2000 \text{ lbs linear inch}$$

$$\text{at one third point } V = 1850 \text{ lbs linear inch}$$

diagonal tension to be taken by rods

$$\frac{(2000 + 1850)}{2}(7)(12)(0.666) = 107800 \text{ lbs.}$$

$$\frac{(12)(56.7)}{(1.5)(80)} = 5.7 \text{ rods, run 6 rods straight to end}$$

$$\text{tensile value of 6 rods } \frac{(6)(0.69)(16000)}{0.7} = 94630 \text{ lbs.}$$

107800 - 94630 = 13170 lbs to be taken by stirrups
 $(0.12)(20) = 0.36$, use 3/8" rods bent, double loop
 spacing of stirrups same as G¹

$$\text{bend two rods at } \frac{21}{2} \left(1 - \sqrt{\frac{(8)(2)}{(12)(10)}} \right) = 6.72', 6'8\frac{1}{2}"$$

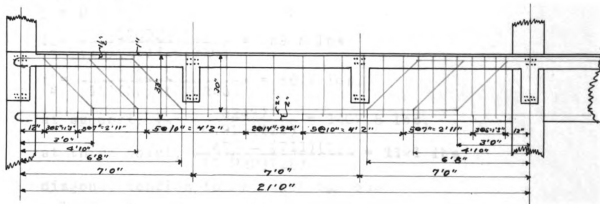
spacing must not be greater than three fourths d or 22 $\frac{1}{2}"$

$$\text{bend next 2 at } 6'8\frac{1}{2}" - 1'10" = 4'10\frac{1}{2}" \text{ from support}$$

$$\text{Bend next 2 at } 4'10\frac{1}{2}" - 1'10" = 3' \frac{1}{2}" \text{ from support}$$

web reinforcing needed to 7.9' from support.

Lap for bond 26"



Girder, G_s.

Reaction from concentrated loads 25011 lbs.
 assumed load of stem 400 lbs.
 wall board on side 283 lbs.
 10 percent added for window 28 lbs.
 equivalent load per linear foot

$$\frac{(25011)(3)}{21} + 711 = 4284 \text{ lbs.}$$

$$V = (25011) + (10.5)(711) = 32477 \text{ lbs.}$$

$$M = \frac{(4284)(21)^2(12)}{12} 0.9 = 1700320 \text{ in. lbs.}$$

$$\text{Area, } \frac{32477}{105} = 309.3 \text{ sq. in. for shear.}$$

testing out for economical d

$$b = 13", \quad d = 25" \quad d' = 27" \quad b' = 13 + (8)(4.5) = 49"$$

$$\text{wgt of beam, } \frac{(13)(27)(150)}{144} = 365.6 \text{ lbs.}$$

$$\frac{M}{b'd^2} = \frac{1700320}{(49)(25)^2} = 55.3 \quad \frac{t}{d} = \frac{4.5}{25} = 0.18$$

$$f_c = 475 \quad j = 0.923$$

$$a_s = \frac{1700320}{(16000)(0.923)(25)} = 4.605$$

use 6, 15/16" round rods at 4.14 sq. in.

$$u = \frac{32477}{(6)(2.945)(0.923)(25)} = 79.6 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{27}{25} = 1.08 \quad p' = \frac{4.605}{(13)(25)} = 0.014 = \frac{p}{2}$$

$$\frac{(6)(79.6)}{(1.5)(80)} = 4, \text{ run straight thru, bend 2}$$

$$L = 9.370 \quad K = 0.0243$$

$$f_c = \frac{1700320}{(13)(25)^2(0.370)} = 889.5 \text{ lbs.}$$

$$f_s = \frac{1700320}{(13)(25)^2(0.0243)} = 8612 \text{ lbs.}$$

$$\text{at support, } \frac{V}{jd} = \frac{32477}{(0.923)(25)} = 1407.5 \text{ lbs.}$$

$$\text{at third point, } \frac{32477 - (711)(7)}{(0.923)(25)} = 1191 \text{ lbs.}$$

diagonal tension to be taken by rods

$$\frac{1407.5 + 1191}{2} (7)(12)(0.66) = 72786 \text{ lbs.}$$

$$\text{value of rods, } \frac{(2)(0.66)(16000)}{0.7} = 31843 \text{ lbs.}$$

$$72786 - 31843 = 40943 \text{ lbs., use 3/8" stirrups}$$

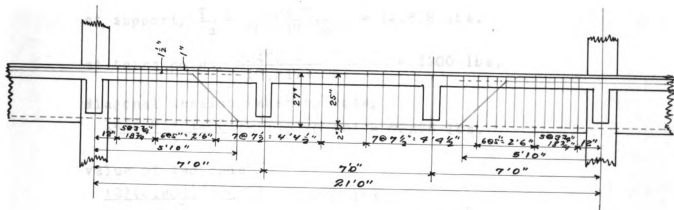
$$\text{stirrups spacing, } \frac{(3)(2)(0.11)(0.823)(25)(16000)}{(2)(32477)} = 3.9"$$

$$\text{at } \frac{1}{8} \text{ spacing, } \frac{4}{3} \times 3.9 = 5.2" \quad \text{at } \frac{1}{4}, (2)(3.9) = 7.8"$$

$$\text{bend 2 rods, } \frac{21}{2} (1 - \sqrt{\frac{(8)(2)}{(12)(6)}}) = 5.9' \text{ from support.}$$

$$\text{web reinforcing needed } \frac{21}{2} - \frac{(40)(13)(0.823)(25)}{4284} = 7.7' \text{ from support}$$

$$\text{length for bond, } \frac{(559.5)(15)(0.875)}{(4)(80)} = 22.9"$$



GIRDER, G₄

reactions from concentrated loads 25011 lbs.

assumed load of stem 400 lbs.

wall load 283 lbs.

10 per cent added for window 28 lbs.

equivalent load per linear foot

$$\frac{(25011)(3)}{21} + 711 = 3284 \text{ lbs.}$$

$$V = (25011) + (10.5)(811) = 32477 \text{ lbs.}$$

$$M = \frac{(3284)(21)^2(12)}{10} = 2,267,093 \text{ in. lbs.}$$

$$\frac{32477}{105} = 309.3 \text{ sq. in. for area.}$$

$$b = 14", d = 25", d' = 27", b' = 50" \text{ wgt. stem, } 393.7 \text{ lbs.}$$

$$\frac{M}{b'd^2} = \frac{2267093}{(50)(25)^2} = 72.5 \text{ lbs.}$$

$$\frac{t}{d} = \frac{4.5}{25} = 0.18 \quad f_c = 590 \quad j = 0.917$$

$$a_s = \frac{2267093}{(16000)(0.917)(25)} = 5.83 \text{ sq. in.}$$

use 8 - 1 $\frac{1}{4}$ " rods at 5.96 sq. in. area

$$u = \frac{32477}{(6)(3.534)(0.917)(25)} = 66.8 \text{ lbs.}$$

$$\frac{d'}{d} = \frac{2.0}{25} = 0.08 \quad p' = \frac{5.96}{(14)(25)} = 0.017 = p$$

$$\frac{(6)(66.8)}{(1.5)(80)} = 3.3, \text{ run 4 rods thru, bend 2}$$

$$d = 0.356 \quad K = 0.0162$$

$$f_c = \frac{2267093}{(14)(25)(0.356)} = 727.7 \text{ lbs.}$$

$$f_s = \frac{2267093}{(14)(25)(0.0162)} = 15800 \text{ lbs.}$$

$$\text{at support, } \frac{V}{j d} = \frac{32477}{(0.917)(25)} = 1416.9 \text{ lbs.}$$

$$\text{at third point, } \frac{32477 - (711)(7)}{(0.917)(25)} = 1200 \text{ lbs.}$$

diagonal tension taken by rods,

$$\frac{1416.9 + 1200}{2} (7)(12)(0.66) = 73276 \text{ lbs.}$$

value of two rods

$$\frac{(2)(0.994)(16000)}{0.7} = 45440 \text{ lbs}$$

$$73276 - 45440 = 27836 \text{ lbs to be taken by stirrups}$$

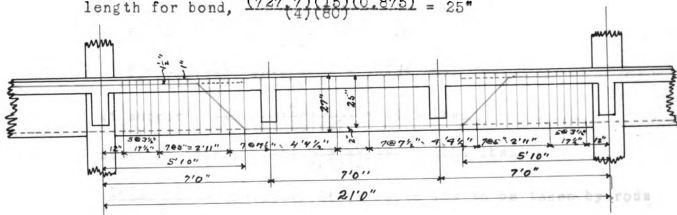
$$(0.012)(25) = 9.3, \text{ use } 7/16" \text{ stirrups}$$

$$\text{spacing } \frac{(3)(2)(0.15)(0.917)(25)(16000)}{(2)(32477)} = 3.8" \text{ c-c}$$

at one eighth point, $\frac{4}{3} \times 3.8 = 5"$, at one quarter, $(2)(3.8) = 7.6"$
bend two rods at 5.9' from support.

$$\text{web rein. needed } \frac{21}{2} - \frac{(40)(14)(25)(0.917)}{4284} = 7.6' \text{ from support}$$

$$\text{length for bond, } \frac{(727.7)(15)(0.875)}{(4)(80)} = 25"$$



GIRDER, G_s

Reaction from concentrated loads, floor	25011
Reaction from concentrated loads, platform	11460
Total concentrations	36471 lbs.
Assumed load of stem	500 lbs.

Equivalent uniform load per linear foot

$$\frac{(36471)(3)}{21} + 500 = 5750 \text{ lbs.}$$

$$V = 36471 + (10.5)(500) = 41721 \text{ lbs.}$$

$$M = \frac{(5750)(21)^2(12)}{12} 0.9 = 2282175 \text{ in. lbs.}$$

$$\frac{41721}{105} = 397.3 \text{ sq. in. area} \quad t = 4.5, \quad r = 60$$

$$b = 15", \quad d = 27", \quad b' = 51", \quad d' = 20"$$

$$\text{wgt. of stem, } \frac{(15)(29)(150)}{144} = 454 \text{ lbs}$$

$$\frac{M}{bd^2} = \frac{2282175}{(51)(27)^2} = 60.1 \quad \frac{t}{d} = \frac{4.5}{27} = 0.16$$

$$f_c = 520 \quad j = 0.925$$

$$a_s = \frac{2282175}{(16000)(0.925)(27)} = 5.7 \text{ sq. in.}$$

use $y - 1\frac{1}{2}"$ round rods at 5.96 sq. in. area.

$$u = \frac{41721}{(6)(3.534)(0.925)(27)} = 78.7 \text{ lbs}$$

$$\frac{d}{d'} = \frac{1.5}{27} = 0.055 \quad p = \frac{5.96}{(15)(27)} = 0.0147$$

$$\frac{(6)(78.7)}{(1.5)(80)} = 3+, \text{ let 4 run thru.}$$

$$L = 9.347 \quad K = 0.0138$$

$$f_c = \frac{2282175}{(15)(27)^2(0.348)} = 602 \text{ lbs.}$$

$$f_s = \frac{2282175}{(15)(27)^2(0.0138)} = 15124 \text{ lbs.}$$

$$\text{at support, } \frac{V}{jd} = \frac{41721}{(0.925)(27)} = 1671 \text{ lbs.}$$

$$\text{at third point, } \frac{41721 - (500)(7)}{(0.925)(27)} = 1531 \text{ lbs.}$$

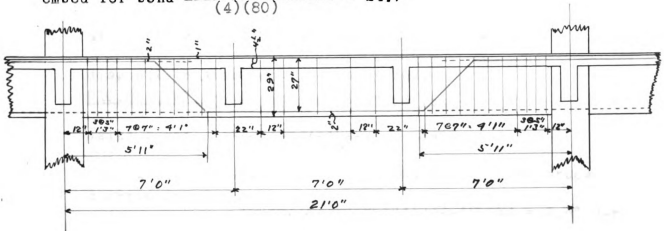
$$\frac{1671 + 1531}{2} (7)(12)(0.66) = 89656 \text{ lbs to be taken by rods}$$

$$\text{value of rods } \frac{(2)(0.994)(16000)}{0.7} = 45440, \text{ two rods}$$

$$89656 - 45440 = 44216 \text{ lbs to be taken by stirrups}$$

$$\text{spacing, } \frac{(3)(2)(0.1963)(0.925)(27)(16000)}{(2)(41721)} = 5.6" \text{ c-c}$$

at one eighth, $\frac{4}{3}(5.6) = 7.4"$, at one fourth $(2)(5.6) = 11.2"$
 bend 2 rods at $\frac{21}{2} \left(1 - \sqrt{\frac{(8)(2)}{(12)(8)}} \right) = 5.9'$ from support.
 wrt rein. needed $\frac{21}{2} - \frac{(40)(15)(27)(0.925)}{5750} = 7.9'$ from support.
 embed for bond $\frac{(602)(15)(0.875)}{(4)(80)} = 24.7"$

GIRDER, G_{18}

Concentrated loads 11460 lbs.

Assumed load of stem 250 lbs

Equivalent load per ft. $\frac{(11460)(3)}{21} + 250 = 1887$ lbs.

$V = 11460 + (10.5)(250) = 14085$ lbs.

$M = \frac{(1887)(21)^2(12)}{12} = 809,067$ in lbs.

$\frac{14085}{105} = 134.1$ sq. in. area for shear

$b = 8"$, $d = 17"$, $b' = 44"$, $d' = 18.5"$

wgt. stem, $\frac{(8)(18.5)(150)}{144} = 155$ lbs.

$M_{d2} = \frac{809067}{(44)(17)^2} = 63.6$ $\frac{t=4.5}{d \ 17} = 0.26$
 $f_c = 475$ $j = 0.905$

$a_s = \frac{809067}{(16000)(0.91)(17)} = 3.269$ sq. in.

use 4 - $1\frac{1}{8}"$ round rods at 3.98 sq. in. area

$u = \frac{14085}{(4)(3.534)(0.91)(17)} = 66.8$ lbs.

$\frac{d}{d'} = \frac{18.5}{17} = 0.088$ $p' = \frac{3.98}{(8)(17)} = 0.029 = p$

$\frac{(9)(66.8)}{(4.5)(80)} = 2.2$, bend two rods

$$L = 9.514 \quad K = 0.0267$$

$$f_c = \frac{809067}{(8)(17)^2(0.514)} = 681 \text{ lbs.}$$

$$f_s = \frac{809067}{(8)(17)^2(0.0267)} = 13107 \text{ lbs.}$$

$$\text{at support, } \frac{V}{j_d} = \frac{14085}{(0.91)(17)} = 910.5 \text{ lbd.}$$

$$\text{at one third, } \frac{14085 - (250)(7)}{(0.91)(17)} = 797.4 \text{ lbs.}$$

diagonal tension to be taken by rods.

$$\frac{910.5 + 797.4}{2} (7)(12)(0.66) = 47824 \text{ lbs.}$$

$$\text{value 2 rods, } \frac{(2)(0.994)(16000)}{0.7} = 45440$$

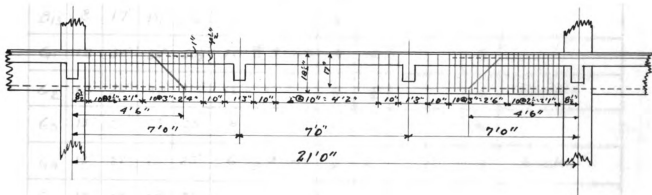
$(0.012)(17) = 0.204"$, use one quarter inch stirrups
stirrup spacing

$$\frac{(3)(2)(0.15)(0.91)(17)(16000)}{(2)(14085)} = 2.6" \text{ c-c}$$

at one eighth, $\frac{4}{3}(2.6) = 3.4"$, at one fourth $(2)(2.6) = 5.2"$
bend 2 rods, $\frac{21}{2} \left(1 - \sqrt{\frac{(8)(2)}{(12)(4)}}\right) = 4.5'$ from support.

web rein. needed, $\frac{21}{2} - \frac{(40)(8)(0.91)(17)}{1887} = 7.9'$ from support.

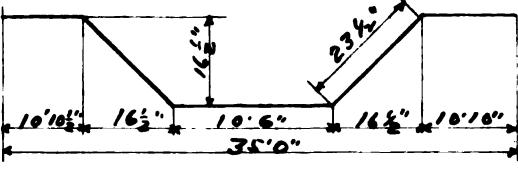
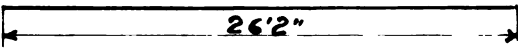
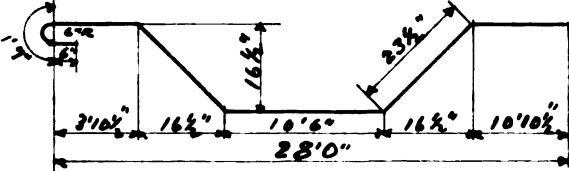
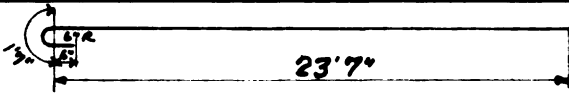
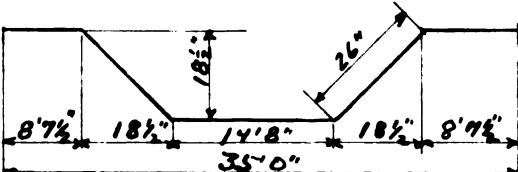
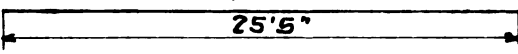
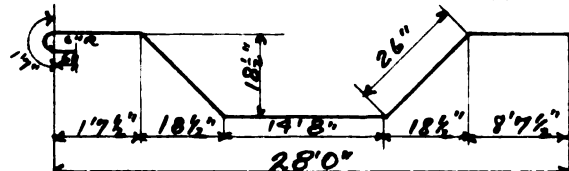
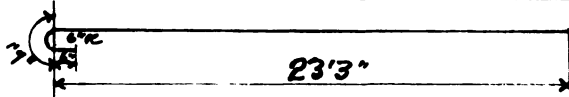
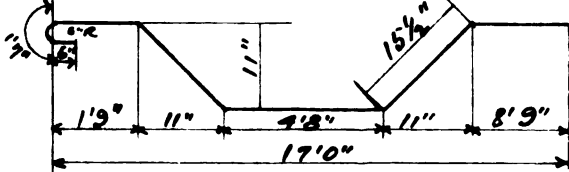
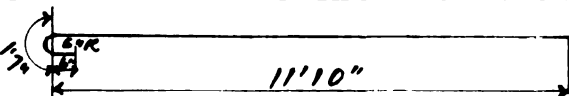
length for bond, $\frac{(681)(15)(0.875)}{(4)(80)} = 28"$



GIRDER, G_{14}

Same as G_{13} , but with hooks on reinforcement.

Beam & Girder Rods.
1st, 2nd, 3rd Floors.

Bends	Size	length	B or G		No. 5 or 6	Total Wanted.
			Mk.	No.		
	$\frac{3}{4}" \phi$	36'2"	B ₁	1	2	416
			B ₂	1	2	180
	$\frac{3}{4}" \phi$	26'2"	B ₁	2	6	1224
			B ₂	2	6	340
	$\frac{3}{4}" \phi$	31'3"	B ₃	1	2	156
			B ₄	1	2	72
	$\frac{3}{4}" \phi$	25'8"	B ₃	2	6	468
			B ₄	2	6	216
	$\frac{13}{16}" \phi$	36'3"	B ₅	1	2	60
	$\frac{13}{16}" \phi$	25'5"	B ₅	2	2	60
	$\frac{13}{16}" \phi$	31'4"	B ₆	1	2	12
	$\frac{13}{16}" \phi$	25'4"	B ₆	2	2	12
	$\frac{3}{4}" \phi$	19'10"	B ₇	1	2	24
			B ₁₃	1	2	4
	$\frac{3}{4}" \phi$	13'11"	B ₇	2	4	48
			B ₁₃	2	4	8

*Beam & Girder,
1st, 2nd, 3rd Floors.*

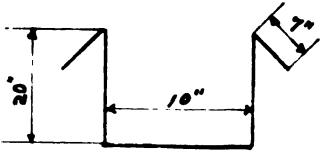
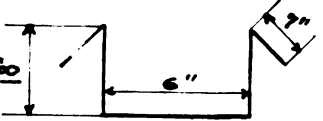
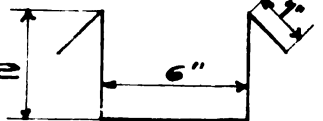
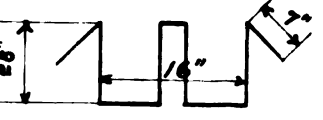
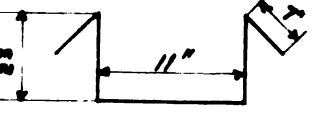
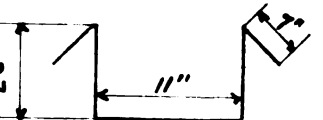
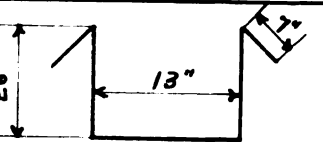
<i>Bends</i>	Size	Length	B or G		No. needed	Total No. Order
			Mk	No.		
	$\frac{3}{4}$ "	14'11"	B ₂	1	2	4
	$\frac{3}{4}$ "	14'2"	B ₂	2	4	8
	$\frac{15}{16}$ "	39'0"	G ₁	1	2	144
	$\frac{15}{16}$ "	39'0"	G ₁	1	2	144
	$\frac{15}{16}$ "	25'4"	G ₁	1	6	432
	$\frac{15}{16}$ "	31'6"	G ₂	1	2	60
	$\frac{15}{16}$ "	31'6"	G ₂	1	2	60
	$\frac{15}{16}$ "	32'0"	G ₂	1	2	60
	$\frac{15}{16}$ "	25'3"	G ₂	1	6	180
	$\frac{15}{16}$ "	36'7"	G ₃	1	2	46
	$\frac{15}{16}$ "	25'5"	G ₃	2	4	92

Beam & Girder
1st, 2nd, 3rd Floor.

Bends	Size	Length	B or G		No Needed	Total No.
			MK	No.		
	$15\frac{3}{8}\phi$	31'8"	G ₂	1	2	12
	$15\frac{3}{8}\phi$	25'2"	G ₂	2	4	24
	$1\frac{1}{8}\phi$	36'8 1/2"	G ₅	1	2	8
	$1\frac{1}{8}\phi$	25'2"	G ₅	2	4	16
	$1\frac{1}{8}\phi$	36'0"	G ₁₃	1	2	6
	$1\frac{1}{8}\phi$	25'8"	G ₁₃	2	2	6
	$1\frac{1}{8}\phi$	31'1"	G ₁₄	1	2	4
	$1\frac{1}{8}\phi$	25'5"	G ₁₄	2	2	4

Stirrup Schedule

1st, 2nd, 3rd Floors.

Bends	Size	Length	B or G		No Each	Total Number
			Mk	No.		
	$\frac{7}{16}\phi$	5'4"	B ₁	1	58	11832
			B ₂	1	58	3220
			B ₃	1	58	4524
			B ₄	1	58	2088
	$\frac{7}{16}\phi$	4'8"	B ₅	1	16	480
			B ₆	1	16	96
	$\frac{1}{4}\phi$	3'4"	B ₇	1	24	288
			B ₁₂	1	24	48
			B ₁₃	1	24	48
	$\frac{3}{8}\phi$	11'10"	G ₁	1	29	2088
			G ₂	1	29	870
	$\frac{3}{8}\phi$	5'11"	G ₃	1	40	920
	$\frac{7}{16}\phi$	5'11"	G ₄	1	40	240
	$\frac{1}{2}\phi$	6'7"	G ₅	1	26	104
	$\frac{1}{4}\phi$	4'2"	G ₁₃	1	61	183
			G ₁₄	1	61	122

COLUMNS.

All interior columns.

3rd story columns.

Live load on roof, (21)(21)(30)=	13230 lbs.
Dead load on slab, (21)(21)(121)=	53361 lbs.
Dead load of slab, (21)(21)(44) =	19404 lbs.
Dead load, 3 beams, (3)(21)(172) =	10836 lbs.
Dead load, 1 girder, (21)(329) =	6909 lbs.
Total on column	103740 lbs.

Concrete $l = l_d = 3$ $f_c = 560$ lbs allowable, $f_c = 800$ lbs allowable with hoops,

$$E_c = \frac{1}{15} E_s \quad n = 26$$

Compute column for 2' less than story height, use octagon shape.

$$\text{short diameter of column } d = \sqrt{\frac{W}{2f_c \tan \alpha}}$$

$$d = \sqrt{\frac{103740}{(2)(800)(0.414)}} = 12.3", \text{ use } d = 12.5", \text{ total } d = 15.5"$$

$$A = 2d^2 \tan \alpha = (2)(12.5)^2 (0.414) = 129.375 \text{ sq. in. area}$$

use 1 per cent of steel, 1.29 sq. in.

8 - $\frac{1}{4}$ " rods give 1.57 sq. in.

$$\text{weight of column, } \frac{(2)(15.5)^2 (0.414)}{144} (11)(150) = 1139.65 \text{ lbs.}$$

use one quarter inch spiral hoops at 2" c-c

2nd story columns.

live load on floor, (21)(21)(250) =	110250 lbs.
dead load of slab, (21)(21)(56) =	24696 lbs.
dead load, 3 beams, (3)(21)(238) =	14994 lbs.
dead load one girder, (21)(500) =	12600 lbs.
Column head load	162540 lbs.
Load from 3rd floor,	104880 lbs.
Total on column	267420 lbs.

$$d = \sqrt{\frac{267420}{(2)(800)(0.414)}} = 20.0"$$

use 21", make total $d = 24"$

$$A = (2)(21)^2 (0.414) = 365.148 \text{ sq. in.}$$

use 1 per cent steel, 3.65 sq. in.

steel, 8 - $\frac{13}{16}$ " round rods.

$$\text{wgt. of column, } \frac{(2)(24)^2 (0.414)(11)(150)}{144} = 5466 \text{ lbs.}$$

use one quarter inch hoops at 2 $\frac{1}{4}$ " c-c

1st Floor columns.

Floor load 162540 lbs.
 Load from above 272886 lbs
 Load on column 435426 lbs.

$$\text{short diameter } \sqrt{\frac{435426}{(2)(800)(0.414)}} = 25.6"$$

use 26", total d = 29"

$$A = (2)(26)^2(0.414) = 559.728 \text{ sq. in.}$$

use 1 per cent steel 5.597 sq. in. , 8 - 1" rods.

$$\text{wgt. of column } \frac{(2)(29)^2(0.414)(11)(150)}{144} = 7979 \text{ lbs.}$$

use one quarter inch hooping at 2 $\frac{1}{2}$ " spacing

Basement Columns.

Floor load 162540 lbs.
 Load from above 443405 lbs.
 Load on column 605945 lbs.

$$\text{short diameter, } \sqrt{\frac{605945}{(2)(800)(0.414)}} = 30.2"$$

use 30.5", total d = 33.5"

$$A = (2)(30.5)^2(0.414) = 770.25 \text{ sq. in.}$$

use 1 per cent steel, 7.70 sq. in., use 8 - 1 $\frac{1}{2}$ " rods

$$\text{wgt. of column } \frac{(2)(33.5)^2(0.414)(9)(150)}{144} = 8712 \text{ lbs.}$$

use one quarter hooping at 2 $\frac{1}{2}$ " spacing.

COLUMN SCHEDULE, Interior

Story	Kind of load	Load	Dia. col.	Vert. rods	Spiral 3" less than diameter
Third	Roof	103740	15.5"	8 - $\frac{1}{2}$ "	$\frac{1}{4}$ " hooping
	Col.	--1140	octa.		spaced 8"
	Total	104880			
Second	Floor	162540	24"	8 - $\frac{1}{2}$ "	$\frac{1}{4}$ " hooping
	Column	--5466	octa.		spaced 2 $\frac{1}{2}$ "
	Total	272886			
First	Floor	162540	29"	8 - 1"	$\frac{1}{4}$ " hooping
	Col.	--7979	octa.		spaced 2 $\frac{1}{2}$ "
	total	443405			
Basement	Floor	162540	33.5"	8 - 1 $\frac{1}{2}$ "	$\frac{1}{4}$ " hooping
	Column	--8712	octa.		spaced 2 $\frac{1}{2}$ "
	Total	614657	on footing.		

Exterior Columns, C², C³, C⁴, C⁵, C⁶, C⁷

Third Story.

Live load from roof $\frac{(21)(21)(30)}{2} = 6615 \text{ lbs.}$ Dead load on slab, $\frac{(21)(21)(121)}{2} = 26681 \text{ lbs.}$ Dead load of slab, $\frac{(21)(21)(44)}{2} = 9702 \text{ lbs.}$ Dead load, one B_s (21)(172) = 3612 lbs.Dead load, one B_s (21)(103) = 2163 lbs.Dead load, $\frac{1}{2}$ pf G₂ $\frac{(21)(329)}{2} = 3455 \text{ lbs}$ Wall load, (21)(283) = ~~5846~~ lbs.

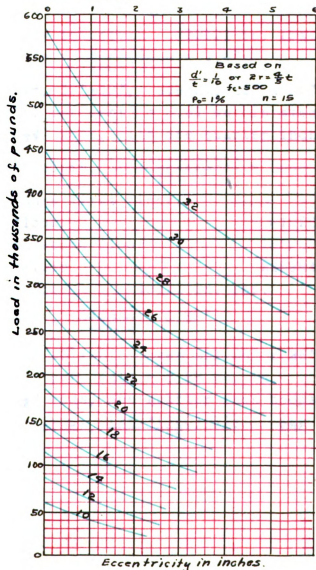
Total load on column 58074 lbs.

Total load on one roof beam, (1537)(21) = 32277 lbs.

Eccentric moment due to girder $\frac{(32277)(7)(12)(2)}{10} = 560734 \text{ in. lbs.}$

Eccentric moment in the column taken at one fourth of moment produced by eccentric load in girder framing into column.

Eccentric moment in column 140183 in lbs

Eccentricity, $\frac{140183}{58074} = 2.5"$ 

Column size 14" x 14", use 1 $\frac{1}{2}$ " fire protection 17"x17"
 $a_s = (17)^2(1 \text{ per cent}) = 2.89 \text{ sq. in.}$, use 8 - $\frac{1}{4}$ " rods
 use one quarter inch bands spaced at 12" c-c

$$\text{wgt. of column } \frac{(17)(17)(150)(11)}{144} = 3212 \text{ lbs.}$$

Second floor, C², C³, C⁴, C⁵, C⁶, C⁷,

$$\text{Live load, } \frac{(21)(21)(250)}{2} = 55105 \text{ lbs.}$$

$$\text{Dead load floor slab, } \frac{(21)(21)(69)}{2} = 15215 \text{ lbs.}$$

$$\text{Dead load, one B}_s \quad (21)(238) = 4998 \text{ lbs.}$$

$$\text{Dead load, one B}_s \quad (21)(137) = 2877 \text{ lbs.}$$

$$\text{Dead load, } \frac{1}{2} \text{ pf G}_2 \quad \frac{(21)(600)}{2} = 6300 \text{ lbs.}$$

$$\text{Walls and windows, } (21)(311) = \underline{\underline{6531 \text{ lbs.}}}$$

$$\text{Total load on floor} \quad 91026 \text{ lbs.}$$

$$\text{Load from third floor} \quad \underline{\underline{59386 \text{ lbs.}}}$$

$$\text{Load on column} \quad 150412 \text{ lbs.}$$

$$\text{Total load on one floor beam, } (2405)(2) = 50595 \text{ lbs.}$$

$$\text{Moment in girder, } \frac{(50505)(7)(12)(2)}{10} = 848484 \text{ in. lbs.}$$

$$\text{Eccentricity in column} \quad 212121 \text{ in. lbs.}$$

$$\text{Eccentricity} \quad \frac{212121}{150412} = 1.4"$$

Column size, 19"x19", overall 22" x 22"

$$a_s = (19)^2(1 \text{ per cent}) = 3.61 \text{ sq. in steel}$$

use 8 - $\frac{1}{4}$ " rods, with one quarter inch bands at 12" c-c

$$\text{weight of column, } \frac{(22)^2(150)(11)}{144} = 5546 \text{ lbs.}$$

Same columns thru first floor.

$$\text{Load on floor} \quad 91026 \text{ lbs.}$$

$$\text{Load from second floor} \quad 155958 \text{ lbs.}$$

$$\text{Load on column} \quad 246984 \text{ lbs.}$$

$$\text{eccentric moment} \quad 212121 \text{ in. lbs.}$$

$$\text{eccentricity, } \frac{212121}{246984} = 0.89"$$

column size 23"x23", over all 26" x 26"

$$a_s = (23)^2(1 \text{ per cent}) = 5.29 \text{ sq. in.}$$

use 8 - $\frac{1}{4}$ " rods, one quarter inch hoops at 12" c-c

$$\text{weight of column, } \frac{(26)^2(150)(11)}{144} = 7826 \text{ lbs.}$$

Same columns thru basement.

Load on floor 91026 lbs.
 Load on first floor, 254730 lbs.
 Load on column, 345756 lbs.
 Eccentric moment in column 212121 in. lbs.

Eccentricity, $\frac{212121}{345756} = 0.62"$

From diagram on page 151

column size, 26"x26" overall 29" x 29"

weight of column, $\frac{(29)^2(159)(9)}{144} = 7885 \text{ lbs.}$

Exterior Columns C₁₆, C₂₄, C₃₂

Third floor,

Live load from roof, $\frac{(21)(21)(30)}{2} = 6615 \text{ lbs.}$

Dead load of slab, $\frac{(21)(21)(44)}{2} = 9702 \text{ lbs.}$

Dead load on slab, $\frac{(21)(21)(121)}{2} = 26681 \text{ lbs.}$

Dead load one B₃ and $\frac{1}{2}$ B₄, $(21)(172)(\frac{3}{2}) = 5408 \text{ lbs.}$

Dead load, one G₃, $(21)(230) = 4830 \text{ lbs.}$

Wall load, $(21)(283) = 5846 \text{ lbs.}$

Total load, 59082 lbs.

Load on one roof beam 32277 lbs.

eccentric moment in girder,

$\frac{(32277)(3)(12)(19.5)}{2 \times 2} = 101672 \text{ in. lbs., in col. 254181 in. lbs.}$

eccentricity $\frac{254181}{59082} = 4.3"$

from table page 151 column size 15"x15", overall 18"x18"

$a_s = (15)^2 \frac{4150}{10000} (1 \text{ per cent}) = 2.25 \text{ sq. in., } 8 - \frac{1}{4}" \text{ rods}$

weight of column, $\frac{(18)^2(150)(11)}{144} = 3728 \text{ lbs.}$

Same columns, second floor.

Live load, 55106 lbs.

Dead load floor 15216 lbs.

1-B₃ and $\frac{1}{2}$ B₄, $(21)(238)(1.5) = 7497 \text{ lbs.}$

1 - G₃, $(21)(366) = 7686 \text{ lbs.}$

Walls and windows 6531 lbs.

Total on floor, 92034 lbs.

From third floor, 62810 lbs.

Total on column, 154844 lbs.

Load on floor beam, 50505 lbs.

Eccentric moment in girder, $\frac{(50505)(12)(10.5)}{2 \times 2} = 1590908 \text{ in. lbs.}$

Eccentric moment in column, 397728 in. lbs.

Eccentricity, $\frac{397728}{154882} = 4.6"$

From table page 151, column 21"x21", overall 24"x24"

$a_s = (21)^2(1 \text{ per cent}) = 4.41 \text{ sq. in.}, 8 - \frac{7}{8}" \text{ rods.}$

weight of column, $\frac{(24)^2(150)(11)}{144} = 6600 \text{ lbs.}$

Same columns thru first floor.

Load on floor, 92034 lbs.

Load from 2nd floor, 161444 lbs.

Load on column, 253478 lbs.

Eccentric moment in column, 397728 in. lbs.

Eccentricity, $\frac{397728}{253478} = 1.5"$

From table page 151, column size 24.5"x24.5", overall 27.5"x27.5"

$a_s = (24.5)^2(1 \text{ per cent}) = 6.00 \text{ sq. in.}, 8 - 1" \text{ rods.}$

weight of column, $\frac{(27.5)^2(150)(11)}{144} = 8665 \text{ lbs.}$

Same columns thru basement.

Load on floor, 92034 lbs.

Load from 1st floor, 262143 lbs.

Load on column, 354177 lbs.

Eccentric moment 397728 in. lbs.

Eccentricity, $\frac{397728}{354177} = 1.2"$

From page 151m column size 27.5"x27.5" overall 30.5"x30.5"

$a_s = (27.5)^2(1 \text{ per cent}) = 7.56 \text{ sq. in.}, \text{ use } 8 - 1\frac{1}{8}" \text{ rods}$

weight of column, $\frac{(30.5)^2(150)(9)}{144} = 8722 \text{ lbs.}$

Columns C_9 , C_{17} , C_{25}

1st, 2nd and 3rd floor same as C_{16} , C_{24} and C_{32}

Thru basement.

Load on floor inside 92034 lbs.

Load from platform 35520 lbs.

Load from above, 262143 lbs.

Total load 389697 lbs.

Live load on floor beam, $(21)(250)(7) = 36750 \text{ lbs.}$

Eccentric moment in girder, $(36750)(3.5)(12) = 1543500 \text{ in. lbs.}$

Eccentric moment in column, 385875 in. lbs.



$$\text{Eccentricity } \frac{385875}{389697} = 0.99"$$

From page 151m column size, 28.5"x28.5", overall 31.5"x31.5"

$$a_s = (28.5)^2(1 \text{ per cent}) = 8.12 \text{ sq. in.}, \text{ use } 8 - 1\frac{1}{4}" \text{ rods}$$

$$\text{weight of column } \frac{(31.5)^2(150)(9)}{144} = 9363 \text{ lbs.}$$

Platform posts. C_{35}, C_{36}, C_{37}

Floor load, 35520 lbs.

Live load on one beam, (7)(10)(250) 17500 lbs.

Eccentric moment in girder, (17500)(3.5)(12) = 735000 in. lbs.

Eccentric moment in post, 183750 in. lbs.

$$\text{eccentricity, } \frac{183750}{35520} = 5.2"$$

From page 151, column size, 14"x14", overall 17"x17".

$$a_s = (14)^2(1 \text{ per cent}) = 1.96 \text{ sq. in.}, \text{ use } 4 - 1\frac{1}{4}" \text{ rods}$$

$$\text{weight of post, } \frac{(17)^2(150)(6)}{144} = 1806 \text{ lbs.}$$

Exterior column schedule, $C_2, C_3, C_4, C_5, C_6, C_7$.

Story	Kind	Load	Dia.	Vert.	Spacing.
	Load		Col.	rods.	
Third	Roof	88074	17"x17"	8 - $1\frac{1}{4}"$	$\frac{1}{4}" - 12"$ c-c
	Col.	3312			
	Total	61386			
Second	Floor	91026	22"x22"	8 - $1\frac{1}{4}"$	$\frac{1}{4}" - 12"$ c-c
	Col.	2246			
	Total	107988			
First	Floor	91026	28"x28"	8 - $1\frac{1}{4}"$	$\frac{1}{4}" - 12"$ c-c
	Col.	7746			
	Total	286730			
Basement	Floor	91026	29"x29"	8 - $1\frac{1}{4}"$	$\frac{1}{4}" - 12"$ c-c
	Col.	7226			
	Total	355641 on footing			

PLATFORM POSTS, C_{35}, C_{36}, C_{37}

Floor 35520 17"x17" 4 - $1\frac{1}{4}"$ $\frac{1}{4}" - 12"$ c-c

Exterior Column Schedule

C₁₆, C₂₄, C₃₂

Story	Kind Load	Load	Dia. Col.	Vent. Rods	Spacing. hoops
Third	Roof	59082	18"×18"	8-5/8"	4"-12" c-c
	Column	<u>3728</u>			
	Total	62810			
Second	Floor	92034	24"×24"	8-7/8"	"
	Column	<u>6600</u>			
	Total	161444			
First	Floor	92034	27.5"×27.5"	8-1"	"
	Column	<u>8665</u>			
	Total	262143			
Basement	Floor	92034	30.5"×30.5"	8-1 1/4"	"
	Column	<u>8722</u>			
	Total	362899 on footing.			

Columns C₉, C₁₇, C₂₅

Third	Roof	59082	18"×18"	8-5/8"	4" - 12" c-c
	Column	<u>3728</u>			
	Total	62810			
Second,	Floor	92034	24"×24"	8-7/8"	"
	Column	<u>6600</u>			
	Total	161444			
First	Floor	92034	27.5"×27.5"	8-1"	"
	Column	<u>8665</u>			
	Total	262143			
Basement	Floor	127554	31.5"×31.5"	8-1 1/4"	"
	Column	<u>9303</u>			
	Total	399000 on footing			

Interior footing, exclusive of those at
elevators and stairs.

Use 1" round rods,

Allowable soil pressure 2 tons per sq. ft.

$$f_c = 650 \quad f_s = 16000 \quad n = 15$$

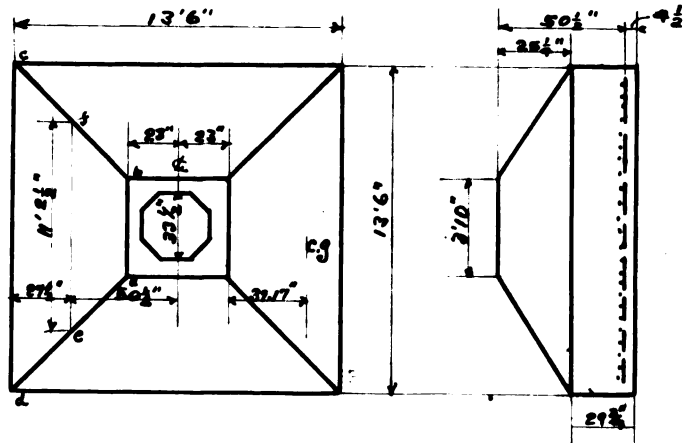
Load on footing 614657 lbs.

Allow 15 per cent for footing 92200 lbs.

Pressure on soil 706857 lbs.

$$\text{area required for pressure } \frac{706857}{4000} = 176.7 \text{ sq. ft.}$$

Use footing 13.5' x 13.5'



Area of column base 771 sq. in., 5.35 sq. ft.

$$\text{Area trapezoid abcd } \frac{182.25 - 5.35}{4} = 44.225 \text{ sq. ft. net.}$$

Upward pressure of soil $(44.225)(4000) = 176900$ lbs.

Distance c.g. of trapezoid from edge of column.

$$\frac{64.25 \times 33.5 + (2)(162)}{3 \times 33.5 + 162} = 39.17"$$

Bending moment at edge of column

$$176900 \times 39.17 = 6,929,173 \text{ in. lbs.}$$

$$\text{depth for moment } \sqrt{\frac{6929173}{(107.4)(33.5)}} = 43.5"$$

depth required for punching shear

$$\frac{176900}{(33.5)(105)} = 50.3"$$

use 50.5" with total depth 55"

$$a_s = \frac{6929173}{(16000)(0.875)(50.5)} = 9.8 \text{ sq. in.}$$

$$\Sigma_o = \frac{176900}{(80)(0.875)(50.5)} = 50.4"$$

space reinforcement over width of column plus twice depth
of footing at column plus one half remaining distance.

$$\text{Area of column base } \frac{(29)^2}{144} = 5.083 \text{ sq. ft.}$$

$$\text{Area trapazoid abcd } \frac{110.25 - 5.083}{4} = 26.292 \text{ sq. ft.}$$

$$\text{Upward pressure of soil } (26.292)(4000) = 105168 \text{ lbs.}$$

distance c.g. of trapazoid from edge of column

$$\frac{42.5}{3} \left(\frac{29 + (2)(126)}{29 + 126} \right) = 25.7 "$$

bedding moment at edge of column

$$(105168)(25.7) = 2,702,817.6 \text{ in. lbs.}$$

$$\text{depth needed for moment } \sqrt{\frac{2702817.6}{(107.4)(29)}} = 29.7 "$$

depth required for punching shear

$$\frac{105168}{(29)(105)} = 34.5 " , \text{ make total depth } 39 "$$

$$a_s = \frac{2702817.6}{(16000)(0.875)(34.5)} = 5.6 \text{ sq. in.}$$

$$\Sigma_o = \frac{105168}{(80)(0.875)(34.5)} = 43.5 "$$

$$\text{space reinforcing over } 29 + (2)(34.5) + \frac{28}{2} = 112 "$$

$$\frac{42.5}{3.142} = 14+, 1 " \text{ rods needed}$$

$$14, 1 " \text{ rods give } 43.988 \text{ ins. perimeter}$$

$$14, 1 " \text{ rods give } 11.00 \text{ sq. in. area}$$

top of footing 4 " square

$$\text{depth } \frac{d}{2} = \frac{34.5}{2} = 17.25 "$$

depth of footing at 34.5" from side of column

$$17.25 + \left(\frac{7}{42.5} \right) (17.25) = 20.09 "$$

$$\text{distance ef, } 126 - (2)(14) = 98 ", 8' 2 "$$

$$\text{area efda, } \frac{110.25 - (8.16)(8.16)}{4} = 10.91 \text{ sq. ft.}$$

$$\text{upward pressure } (10.91)(4000) = 43640 \text{ lbs.}$$

$$\text{shear on ef, } \frac{43640}{(98)(20.09)(0.875)} = 25.3 \text{ lbs.}$$

weight of footing,

$$(10.5)(10.5)(1.8)(150) = 29932.5 \text{ lbs.}$$

$$\frac{(4)(3.54)(1.44)(3.41)(150)}{2} = 5214.8 \text{ lbs}$$

$$\frac{(1.44)(7.1)(7.1)(150)}{3} = 3629.5 \text{ lbs}$$

$$(3.41)(3.41)(1.44)(150) = 2511.6 \text{ lbs.}$$

$$\text{total } 41288.8 \text{ lbs.}$$

WALL FOOTINGS, F_{16} , F_{24} , F_{32}

Load on footing 362899 lbs.

15 per cent for footing 54435 lbs.

soil pressure 417334 lbs.

area required, $\frac{417334}{4000} = 104.33$ sq. ft.

use 10.5' x 10.5' footing, area 110.25

Make these footings same as F_2 , F_3 , F_4 , F_5 , F_6 , F_7 WALL FOOTINGS, F_9 , F_{17}

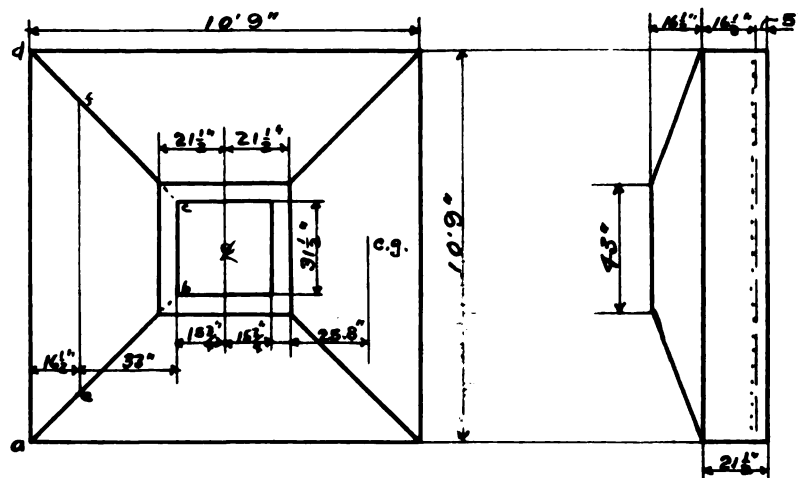
Load on footing 399000 lbs.

15 per cent for footing 59850 lbs

soil pressure 458850 lbs.

area required, $\frac{458850}{4000} = 114.7$ sq. ft.

use 10.75' x 10.75' footing, area 115.56 sq. ft.

area column base, $\frac{(31.5)^2}{144} = 6.89$ sq. ft.area abcd, $\frac{115.56 - 6.89}{4} = 27.17$ sq. ft.Upward soil pressure $(27.17)(4000) = 108680$ lbs.

Distance c.g. trapezoid from edge of footing head

$$\frac{43}{3} \left(\frac{31.5 + (2)(129)}{31.5 + 129} \right) = 25.8"$$

bending moment at edge of column, $(108680)(25.8) = 2803944$ in.lbs.depth for moment $\sqrt{\frac{2803944}{(107.4)(31.5)}} = 28.8"$

depth for punching shear

$$\frac{108680}{(31.5)(105)} = 32.8", \text{ make total depth } 38"$$

 $a_s = \frac{2803944}{(16000)(0.875)(38)} = 6.07$ sq. in. steel $\Sigma_o = \frac{108680}{(80)(0.875)(38)} = 47"$

space reinforcing over

$$31.5 + (2)(33) + \frac{32.5}{2} = 113.75"$$

$$\frac{47}{3.142} = 14+, 1" \text{ rods needed, use 15}$$

$$\frac{113.75}{14} = 8" \text{ c-c}$$

15 rods give 47.13" perimeter, 11.78 sq. in area.

top of footing 43" square,

depth $33/2 = 16.5"$

depth at 33" from side of column

$$16.5 + \left(\frac{8.25}{43}\right)(16.5) = 19.66"$$

distance ef, $129 - 32.5 = 96.5"$, or 8.04'

$$\text{area efca, } \frac{115.56 - (8.04)^2}{4} = 12.73 \text{ sq. ft.}$$

upward soil pressure, $(12.73)(4000) = 50920 \text{ lbs.}$

$$\text{shear on ef, } \frac{50920}{(96.5)(19.66)(0.875)} = 30.4 \text{ lbs sq. in.}$$

FOOTINGS UNDER PLATFORM

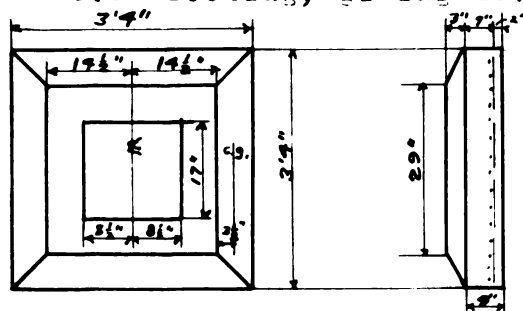
Load on footing 37326 lbs.

15 per cent for footing 5598.9 lbs.

Soil pressure 42924.9 lbs.

$$\text{area required } \frac{42924.9}{4000} = 10.8 \text{ sq. ft.}$$

use 3.3' x 3.3' footing, giving 10.89 sq. ft.



$$\text{area column base } \frac{(17)^2}{144} = 2.01 \text{ sq. ft.}$$

$$\text{area abcd, } \frac{10.89 - 2.01}{4} = 2.22 \text{ sq. ft.}$$

upward soil pressure, $(4000)(2.22) = 8800 \text{ lbs.}$

distance c.g. trapezoid from edge of footing head,

$$\frac{3.6(17 + (2)(33.6))}{3(17 + 33.6)} = 3.15"$$

bending moment, $(8800)(3.15) = 27608 \text{ in. lbs.}$

$$\text{depth for moment } \sqrt{\frac{27608}{(107.4)(17)}} = 3.8"$$

$$\text{depth for punching shear } \frac{8800}{(17)(105)} = 4.9"$$

make total depth 9"

$$\Sigma_o = \frac{8800}{(30)(0.375)(5)} = 25.14" \text{ per factor}$$

$$\frac{25.14}{3.142} = 8.01" \text{ rods needed}$$

$$\frac{17 + (2)(5) + 12.6}{(2)(8)} = 2.4", \text{ spacing c-c}$$

depth of footing at 5" from side of column, 9"

make footing full rectangle

$$\text{weight of footing } (3,3)(3,3)(0,75)(150) = 1225 \text{ lbs.}$$

COLUMN C₁

Third floor column, rectangular section.

$$\text{Live load from roof, } \frac{(21)(21)(30)}{4} = 3207.5 \text{ lbs.}$$

$$\text{Dead load slab, } \frac{(21)(21)(121)}{4} = 13340.5 \text{ lbs}$$

$$\text{dead load of slab } \frac{(21)(21)(44)}{4} = 4881.0 \text{ lbs}$$

$$\text{dead load, one half } E_s, \frac{(21)(172)}{2} = 1806.0 \text{ lbs}$$

$$\text{dead load, one half } E_s, \frac{(21)(108)}{2} = 1081.5 \text{ lbs}$$

$$\text{dead load, one half } C_s, \frac{(21)(230)}{2} = 2415.0 \text{ lbs}$$

$$\text{wall load, } (21)(288) = 5848.0 \text{ lbs}$$

$$\text{Total load on column, } 32647.5 \text{ lbs.}$$

$$\text{Total load on one roof beam at wall } \frac{(1537)(21)}{2} = 16138.5 \text{ lbs.}$$

$$\text{eccentric moment of girder, } \frac{(16138.5)(7)(12)(2)}{10} = 271126.8 \text{ in.lbs.}$$

$$\text{eccentricity in column, } 67781.7 \text{ in. lbs.}$$

$$\text{eccentricity, } \frac{67781.7}{32647.5} = 2.1"$$

Column size, from design on page, 151, 11" x 11",

use 1½" cover for fire protection, 14" x 14"

$$a_s = (14)^2(1 \text{ per cent}) = 1.96 \text{ sq. in., use 8, } 9/16" \text{ rods.}$$

use ½" band at 12" c-c

$$\text{weight of column, } \frac{(14)(14)(150)(11)}{144} = 2245 \text{ lbs.}$$

SECOND FLOOR,

$$\text{dead load of slab, } 7606.5 \text{ lbs.}$$

$$\text{live load } 27552.5 \text{ lbs.}$$

$$\text{One half, } E_s, 2499.0 \text{ lbs.}$$

$$\text{One Half, } C_s, 4137.0 \text{ lbs}$$

$$\text{One half, } E_s, 1438.5 \text{ lbs.}$$

Wall load, (21)(311) 6531.0 lbs.
 Total load on floor, 49765.5 lbs.
 From third floor, 34894.0 lbs.
 Total load on column 84659.5 lbs.
 Load on floor beam at wall, 25253 lbs.
 Moment in wall girder, $\frac{(25253)(7)(12)(2)}{10} = 424242 \text{ in.lbs.}$
 eccentricity in column, 106060 in. lbs.
 eccentricity, $\frac{106060}{84659.5} = 1.25"$

from diagram on page 151
 column size 14.5" x 14.5", overall size 17.5" x 17.5"
 $a_s = (14.5)^2(1 \text{ per cent}) = 2.1 \text{ sq. in.}$
 use 8 # 5/8" rods,
 weight of column, $\frac{(17.5)^2(150)(11)}{144} = 3510 \text{ lbs.}$

FIRST FLOOR.

Load on floor, 49766 lbs.
 Load from second floor, 88170 lbs.
 Load on column 137936 lbs.
 Eccentric moment, 106060 in. lbs.
 eccentricity, $\frac{106060}{137936} = 0.77"$

From diagram on page 151.
 Column size 17" x 17", overall size 20" x 20"
 $a_s = (17)^2(1 \text{ per cent}) = 2.89 \text{ sq. in.}$
 use 8, 11/16" rods.
 weight of column $\frac{(20)^2(150)(11)}{144} = 4584 \text{ lbs.}$

BASEMENT.

Load on floor, 49766 lbs.
 Load from 1st floor 142520 lbs.
 Load on column, 192286 lbs.
 eccentric moment, 106060 in. lbs.
 eccentricity, $\frac{106060}{192286} = 0.55"$

From diagram on page 15
 column size 19.5" x 19.5", overall 22.5" x 22.5"
 $a_s = (19.5)^2(1 \text{ per cent}) = 3.80 \text{ sq. in.}$
 use 8, 13/16" rods.
 weight of column, $\frac{(22.5)^2(150)(9)}{144} = 4747 \text{ lbs.}$

CORNER COLUMN C₁

Story	Kind of load	Load	Dia. of column	Vert. rods	Spacing
Third	Roof	32648			
	Column	<u>2246</u>	14"×14"	8-9/16"	1/4"-12" c-c
	Total	34894			
	Floor	49766			
Second	Column	<u>3510</u>	17.5"×17.5"	8-5/8"	1/4" - 12" c-c
	Total	88170			
	Floor	49766			
First	Column	<u>4584</u>	20"×20"	8-11/16"	1/4" - 12" c-c
	Total	142520			
	Floor	49766			
Basement	Column	<u>4747</u>	22.5"×22.5"	8-13/16"	1/4" - 12" c-c
	Total	197033	on footing.		

FOOTING, F₁

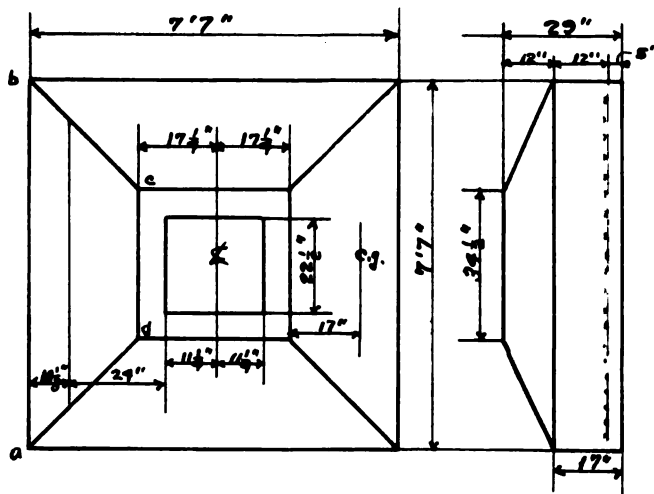
Load 197033 lbs.

15 per cent for footing 29555 lbs.

Soil pressure 226588 lbs.

area required $\frac{226588}{4000} = 56.65$ sq. ft.

use 7.6'×7.6' footing at 59.76 sq. ft.



Area col. base 3.51 sq. ft.

area trapezoid abcd

$$\frac{59.76 - 3.51}{4} = 14.06 \text{ sq. ft.}$$

upward soil pressure,

$$(14.06)(4000) = 56240 \text{ lbs.}$$

distance c.g. from edge of column.

$$\frac{28.35}{3} \left(\frac{22.5 + (2)(91.2)}{22.5 + 91.2} \right) = 17.01"$$

bending moment at column

$$(56240)(17.01) = 956642.4 \text{ in. lbs.}$$

depth for moment

$$\sqrt{\frac{956642.4}{(107.4)(22.5)}} = 19.8"$$

depth for punching shear

$$\frac{56240}{(22.5)(105)} = 23.8"$$

make total depth 29"

$$\Sigma_o = \frac{56240}{(80)(0.875)(24)} = 33.4" \text{ perimeter.}$$

$$\frac{33.4}{3.142} = 10+, 1" \text{ rods needed}$$

11 one inch rods give 8.64 sq. in. area.

11 one inch rods give 34.56 in. perimeter.

top of footing 34.5" sq. depth 12"

depth of footing at 24" from side of column

$$12 + \left(\frac{10.35}{18.35}\right)(12) = 18.7"$$

distance ef, $91.2 - 20.70 = 70.5"$, or 5.87'

$$\text{area efda, } \frac{59.76 - (5.87)^2}{4} = 6.325 \text{ sq. ft.}$$

upward pressure, $(6.325)(4000) = 25300 \text{ lbs.}$

$$\text{shear on ef, } \frac{25300}{(70.5)(18.7)(0.875)} = 24.7 \text{ lbs.}$$

weight of footing,

$$(7.6)^2(150)(1.42) = 12302.88$$

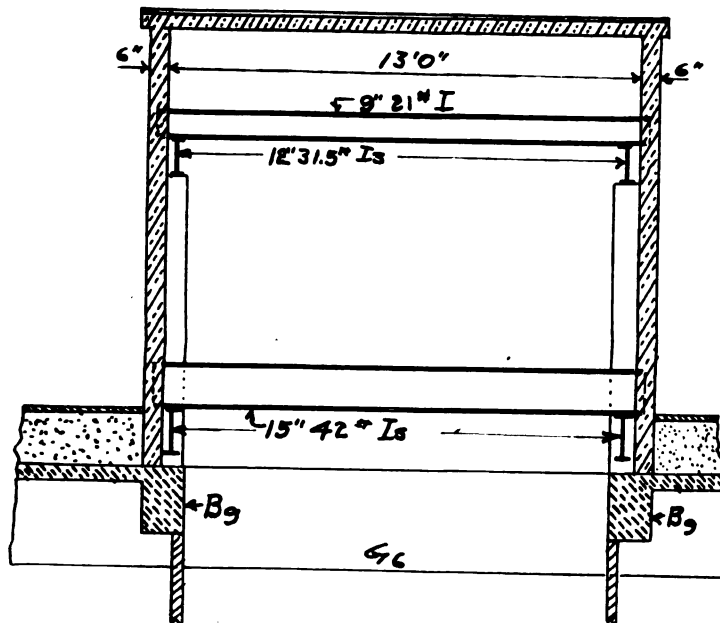
$$\frac{(4)(12)(34.5)(28.35)(150)}{(2)(1728)} = 2031.88$$

$$\frac{(12)(56.7)(56.7)(150)}{(3)(1728)} = 1116.29$$

$$\frac{(34.5)(34.5)(12)(150)}{1728} = 1239.84$$

Total weight of footing 16690.9 lbs.

Problem 28. Elevator shaft and Pent House.



Roof 14'6"×15'2"

Snow load 30 lbs. sq. ft.

$(14.5)(15.17)(30) = 6600$ lbs.

From table 6, use 4" slab,
total depth 5"

Slab load, 62 lbs. sq. ft.

$a_s = 0.37$ sq. in. per ft. slab.

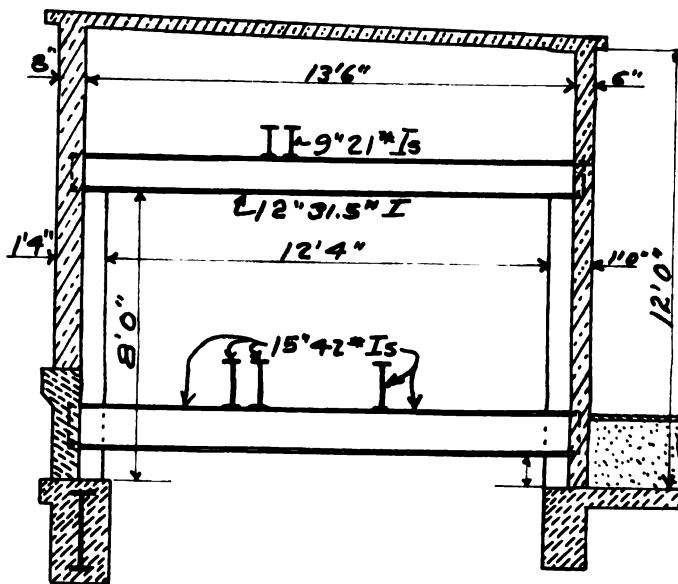
use $\frac{1}{4}$ " rods, both directions

6" c-c

Use 6" walls at sides, 8"

walls at front and rear.

12'×12' clear shaft inside.



[illegible]

Roof Beam b9.

Snow load from pent house 1650.0 lbs.
 Load from roof slab, 3410.0 lbs.
 Side wall, $(0.5)(14.66)(12.25)(150)=13469.0$ lbs.
 Load from main roof, $(195)(1.5)(14.66)=4288.0$ lbs.
 Live load from elevators

28600 lbs + 100 per cent impact,

one-half to b9

28600.0 lbs.

Total load on b9

51417.0 lbs.

Assumed beam load, $(400)(14.66)$

5864.0 lbs.

Total weight

57281.0 lbs.

$$V = 57281 + 2 = 28640 \text{ lbs.}$$

$$M = \frac{(57281)(14.66)(12)}{8} = 1,259,586 \text{ in. lbs.}$$

Area for shear $28640 \div 105 = 272.7$ sq. in.

if $d = 20"$, with total $d = 22.5"$, $b = 14"$

designed as a rectangular beam.

$$K = \frac{M}{bd^2} = \frac{1259586}{(14)(20)^2} = 224.9$$

From table 3, $p = 0.017$ $k = 0.503$ $j = 0.833$

$f_c = 1080$

$$a_s = \frac{1250586}{(16000)(0.833)(20)} = 4.8 \text{ sq. in.}$$

use 4 - $1\frac{1}{4}"$ rd. rods at 5.06 sq. in. 3" c-c

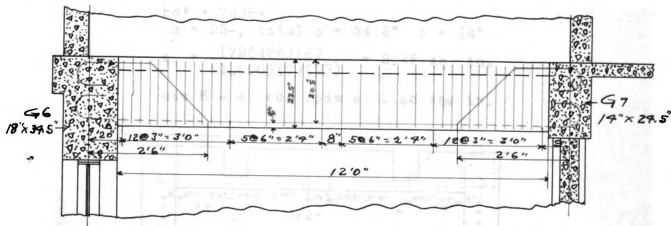
$$U = \frac{28640}{(4)(4.5)(0.833)(20)} = 90 \text{ lbs.}$$

$$x_2 = \frac{12[1 - \sqrt{1 - \frac{8 \times 2}{2 \times 22.5}}]}{2} = 2.52, \text{ bend up 2 at } 2'6\frac{1}{2}"$$

use $\frac{3}{8}"$ stirrups

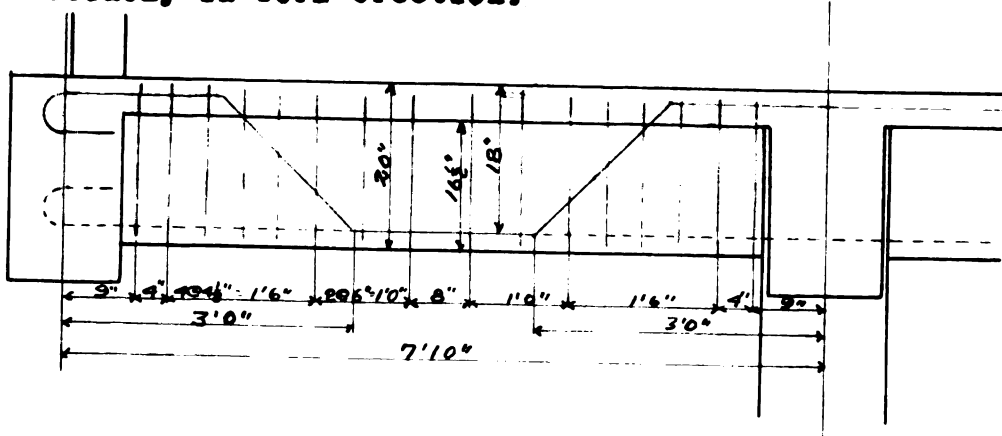
$$s = \frac{(3)(0.11)(2)(16000)(0.85)(20)}{(2)(28640)} = 3.1", \text{ space } 3" \text{ for } 3'$$

6" spacing for rest.

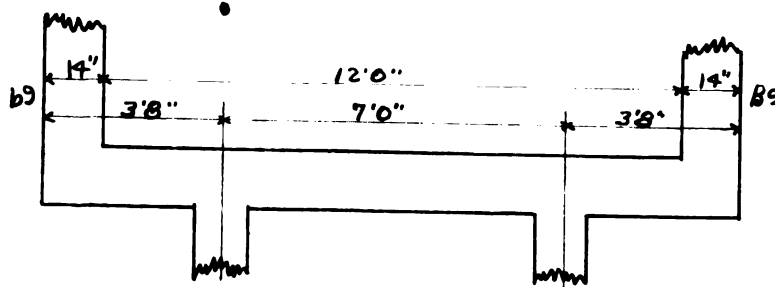


Beam b10.

Make of same dimensions as B3 and B4 for ease and economy in form erection.



Girder G7



Snow load from pent house	1650.0 lbs.
Load from roof slab	3410.0 lbs.
Side wall load	13469.0 lbs.
Main roof load, (19.5)(4.5)(14.0)	12286.0 lbs.
2 B10 concentrations,	1548.0 lbs.
Load on G7	32362.0 lbs.
Assumed stem G7, (300)(14.5)	4350.0 lbs.
Total load,	36712.0 lbs.

$$V = 18356 \text{ lbs.}$$

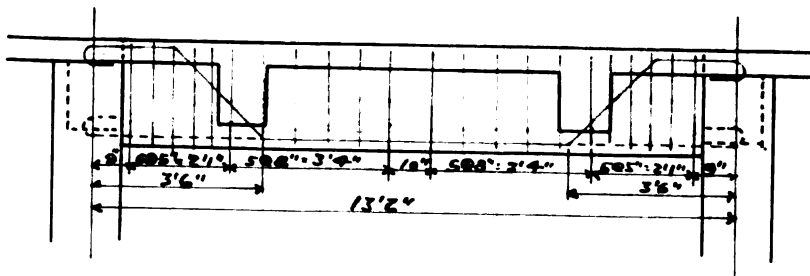
$$M = \frac{(36712)(14.5)(12)}{8} = 798486 \text{ in. lbs.}$$

$$bd^2 = 7435+$$

$$d = 23-, \text{ total } d = 24.5" \quad b = 14"$$

$$a_s = \frac{(798486)(8)}{(1,000)(7)(23)} = 2.48 \text{ sq. in.}$$

use 6 - $\frac{1}{4}$ " rd. rods at 2.48 sq. in.



$$u = \frac{18356}{(6)(2.35)(0.875)(23)} = 64 \text{ lbs.}$$

$$x_2 = \frac{12.8}{2} \left[1 - \sqrt{\frac{8 \times 2}{12 \times 6}} \right] = 3' 5", \text{ bend up 2 rods}$$

use $\frac{3}{4}"$ stirrups

$$s = \frac{(3)(0.11)(2)(16000)(0.875)(23)}{(2)(18356)} = 5.7"$$

use 5" spacing for 2'0", 8" spacing for rest.

Girder G6 for roof.

Pent house roof load	1650.0 lbs.
Roof slab,	3410.0 lbs.
Side wall load,	13469.0 lbs.
2 concentrations, 28640	57280.0 lbs.
Load on G6	75809.0 lbs.
Assume 10 per cent for stem	7580.0 lbs.
Total load	83389.0 lbs.

$$\text{Shear} = 83390 \div 2 = 41695 \text{ lbs.}$$

$$M = \frac{(83390)(21)(12)}{10} = 2,101,428 \text{ in. lbs.}$$

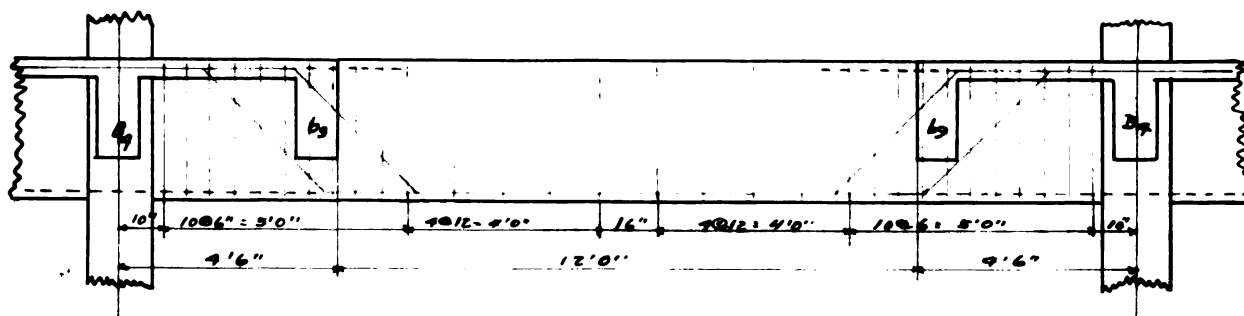
$$bd^2 = \frac{2,101,428}{107.4} = 19566.3$$

assume $b = 18"$

$$d = \sqrt{\frac{19566.3}{18}} = 33", \text{ total } d = 34.5"$$

$$a_s = \frac{2,101,428}{(16000)(0.875)(33)} = 4.54 \text{ sq. in.}$$

use 8 = $\frac{3}{4}"$ rd. rods at 4.81 sq. in.



$$u = \frac{41695}{(8)(2.75)(0.875)(33)} = 65 \text{ lbs.}$$

$$x_2 = \frac{21}{2} \left[1 - \sqrt{\frac{8 \times 2}{12 \times 8}} \right] = 6' 2", \text{ bend 2}$$

$$x_2 = \frac{21}{2} \left[1 - \sqrt{\frac{8 \times 2}{12 \times 8}} \right] = 4' 3", \text{ bend 2 more}$$

use one half inch stirrups

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$$s = \frac{(3)(0.196)(2)(16000)(0.875)(33)}{(2)(41695)} = 6.5"$$

space 6" for 5', 12" spacing for rest.

Beam b9 for floors.

Live load from floor 250 lbs per sq. ft.

$$\text{L.L. from floor } (250)(1.5)(12) = 4500 \text{ lbs.}$$

$$\text{Floor slab, } (56)(1.5)(12) = 1008 \text{ lbs.}$$

$$\text{on beam } 5508 \text{ lbs.}$$

$$\text{Assumed stem of beam } 65 \text{ lbs.}$$

$$\text{Total load } 5573 \text{ lbs.}$$

$$V = 2788 \text{ lbs.}$$

$$M = \frac{(5573)(12)(12)}{12} = 66900 \text{ in. lbs.}$$

assume b at 6"

$$d = \sqrt{\frac{66900}{(107.4)(6)}} = 10", \text{ total } d = 12"$$

$$a_s = \frac{66900}{(16000)(0.875)(10)} = 0.478 \text{ sq. in.}$$

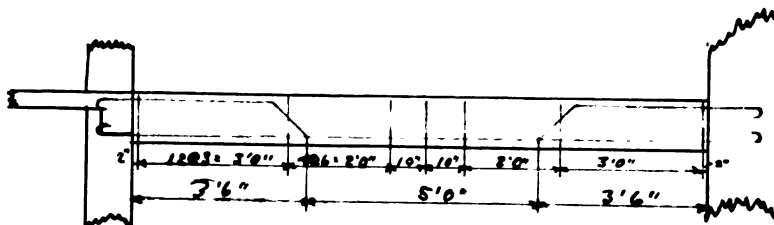
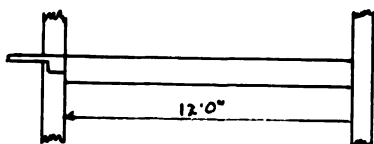
use 2 - $\frac{1}{2}$ " sq. rods.

$$u = \frac{2788}{(2)(4)(0.875)(10)} = 40 \text{ lbs.}$$

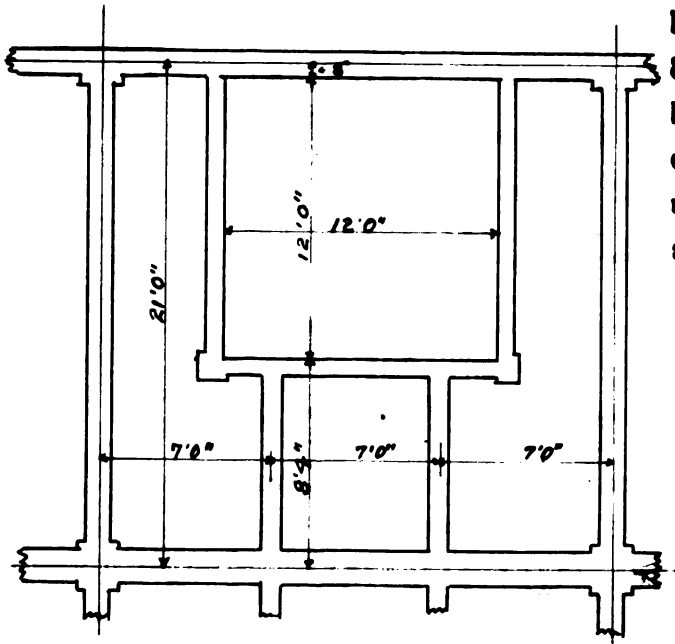
$$x_s = \frac{12}{2} \left[1 - \sqrt{\frac{2 \times 2}{12 \times 2}} \right] = 3.6', \text{ bend 2 rods}$$

use 1/2" stirrups

$$s = \frac{(3)(0.196)(2)(16000)(0.875)(10)}{2 \times 2788} = 2.95", \text{ use 3" c-c}$$



Beam B10 floors.



Make B10 same as B4

b = 12" d = 22.5"

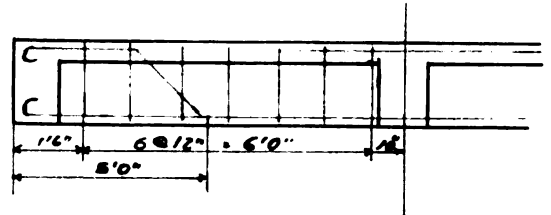
8 - 3/4" rd. rods in 2 rows

bend 2 rods at 4' 0" from support.

carry six straight

use 7/16" stirrups

space 12" c-c entire length B10



Girder G7 floors,

Live Load 250 lbs., 100 per cent impact, 500 lbs.

2 concentrations of 9528 lbs each, 19056 lbs.

additional live load, $250 \times 4 \times 12 = 1000$ lbs.equivalent linear load per foot, $\frac{19056}{12} = 1588$ lbs.

Total load, 2588 per linear foot,

Assume stem 400 lbs.

Load 2988 lbs, say 3000 lbs per linear ft.

$$M = \frac{(3000)(12)(12)}{12} = 36000 \text{ in. lbs.}$$

assume b = 8"

$$d = \sqrt{\frac{(36000)}{(107.4)(8)}} = 6.5", \text{ total } d = 8"$$

$$a_s = \frac{36000}{(16000)(0.875)(6.5)} = 0.396 \text{ sq. in.}$$

use 2 - 1/2" rd. rods

$$s = \frac{(3)(0.15)(16000)(0.875)(6.5)}{(2)(6)(3000)} = 2.25"$$

space 2" for 2', 3" for 2', 6" for 2'

Girder G6 floors.

Concentrated load 2788 lbs.
 Stem 400 lbs.
 Wall and window, 310 lbs.

equivalent load per foot

$$\frac{2788 \times 3}{21} + 710 = 1108 \text{ lbs.}$$

$$V = 2788 + (10.5)(710) = 10249 \text{ lbs.}$$

$$M = \frac{(1108)(21)^2(12)}{12} = 488628 \text{ in. lbs.}$$

Make same dimension as G3

$$b = 13", \quad d = 25", \quad \text{total } d = 27"$$

$$a_s = \frac{488628 \times 8}{(16000)(7)(25)} = 1.39 \text{ sq. in.}$$

use 2 - 15/16" rd. rods at 1.39 sq. in.

use 3/8" stirrups

$$s = \frac{(3)(2)(0.11)(0.875)(25)(16000)}{(2)(10243)} = 11"$$

space 11" for 3', 14" for 3', 18" for 4'

[illegible]

Problem 28.

Make complete details for stair shaft. Beams, girders, etc.,

Snow load on roof	30 lbs. sq. ft.
Slab load on roof,	62 lbs. sq. ft.
Wall loads	75 lbs. sq. ft.
Snow (7)(0.5)(30)	= 2205 lbs.
Slab (7)(3.5)(62)	= 1519 "
Wall (7)(6)(75)	= 3150 "
Stairs (7)(50)	= 350 "

Total 7224 "

$$V = 3612 \text{ lbs.}$$

$$M = \frac{(7224)(7)(12)}{12} = 50568 \text{ in. lbs.}$$

$$bd^2 = \frac{50568}{107.4} = 470.9$$

let $b = \text{one half } d$

$$\frac{d^3}{2} = 470.9 \quad d^3 = 941.8 \quad d = 9.8"$$

if $b = 6"$, then total $d = 12"$.

$$a_s = \frac{50568(8)}{(16000)(7)(9.8)} = 0.362 \text{ sq. in.}$$

use 2 - $\frac{1}{2}$ " round rods at 0.393 sq. in.
run straight thru.

Make G_{10} same dimensions as b_s , rein. same.

Make G_8 same dimensions as G_2 , rein. same.

Make G_6 same dimensions as G_4 , rein. straight

b , same dimensions as b_s .

B₁ floors:

Live load and floor	250 lbs. sq. ft.
Slab load	56 lbs. sq. ft.
Stem	50 lbs. sq. ft.
L.L. and Floor (7)(7)(250)	= 7250 lbs.
Slab,	(7)(7)(56) = 2744 "
Stem,	(7)(50) = 350 "

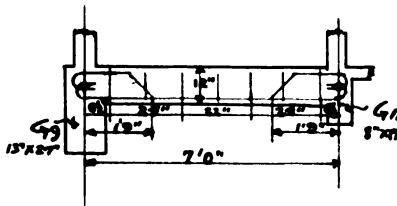
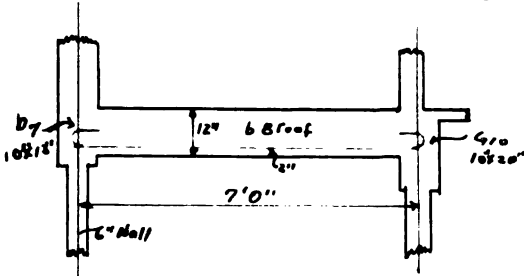
Total 10344 lbs.

$$V = 5172 \text{ lbs.}$$

$$M = \frac{(10344)(7)(12)}{12} = 72408 \text{ in. lbs.}$$

$$bd^2 = \frac{72408}{107.4} = 675.0$$

$b = \text{one half } d$



$$d^3 = 1350 \quad d = 10.9"$$

$$\text{let } b = 6", \text{ total } d = 12"$$

$$a_s = \frac{72408 \times 8}{(16000)(7)(10.9)} = 0.474 \text{ sq. in.}$$

use 3 - $\frac{1}{2}"$ rounds at 0.589 sq. in.

hook ends,

use $\frac{1}{2}"$ stirrups

$$\text{space } \frac{(3)(2)(0.196)(0.875)(10.9)(16000)}{(2)(5172)} = 17.3"$$

make spacing 12" c-c throughout.

G₁₁ for floor.

$$\text{Load concentrations} \quad 5172$$

$$\text{Slab } (7)(3.5)(5) = 1372$$

$$\text{Wall } (14)(13)(50) = 9100$$

$$\text{total} \quad 15644 \text{ lbs.}$$

$$V = 7322 \text{ lbs.}$$

$$M = \frac{(15644)(14)(12)}{12} = 219016 \text{ in. lbs.}$$

$$bd^2 = \frac{219016}{107.4} = 204.0$$

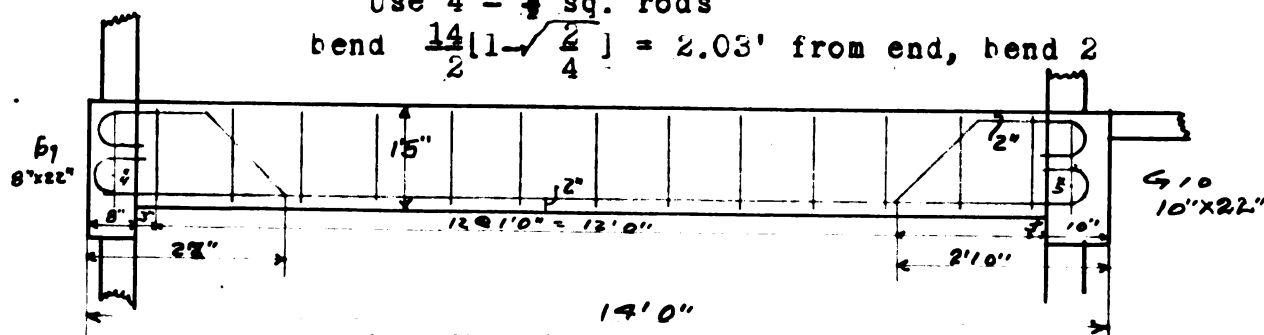
$$d^3 = 4080 \quad d = 15.9" \quad \text{total } d @ 17"$$

$$b = 8"$$

$$a_s = \frac{(219016)(8)}{(16000)(7)(15.9)} = 0.983 \text{ sq. in.}$$

Use 4 - $\frac{1}{2}"$ sq. rods

$$\text{bend } \frac{14}{2} \left[1 - \sqrt{\frac{2}{4}} \right] = 2.03' \text{ from end, bend 2}$$



Use $\frac{1}{2}"$ stirrups

$$\text{space } \frac{(3)(2)(0.196)(0.875)(15.9)(16000)}{(2)(7322)} = 17.8"$$

space stirrups throughout at 12" c-c

G₁₀ for floors,

$$\text{Slab load } (2)(21)(56) \quad 2352 \text{ lbs.}$$

$$\text{Concentration } G_{11} \quad 7322 "$$

$$\text{Wall, } (14)(13)(50) \quad 9100 "$$

$$\text{Total} \quad 18674 "$$

$$\text{add 15 per cent for stem} \quad 2816 "$$

$$\text{Gross total} \quad 21590 "$$

$$V = 19795 \text{ lbs.}$$

$$M = \frac{(21590)(21)(12)}{12} = 453390 \text{ in. lbs.}$$

$$bd^2 = \frac{453390}{107.4} = 4320$$

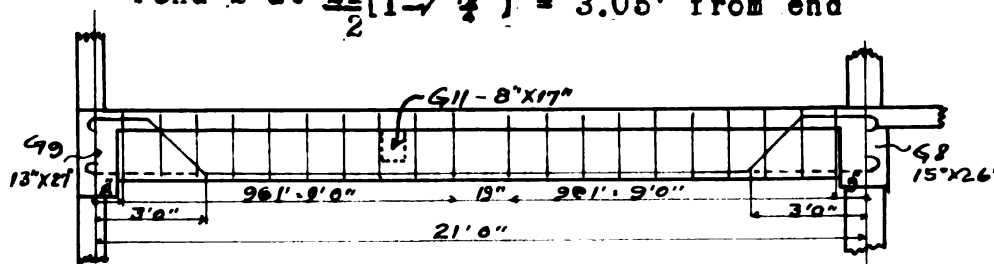
$$d^2 = 8460 \quad d = 29.3"$$

$$b = 10" \quad \text{total } d = 22"$$

$$a_s = \frac{453390 \times 8}{(16000)(7)(20.3)} = 1.59$$

Use 4 - $\frac{3}{4}$ " round rods

$$\text{bend } 2 \text{ at } \frac{21}{2} \left(1 - \sqrt{\frac{2}{3}}\right) = 3.05' \text{ from end}$$



Use $\frac{3}{4}$ " stirrups

space at 12" s-s c-c throughout.

Make B₇ same dimensions as B₅

Make G₉ same dimensions as G₃

Make G₈ same dimensions as G₂

Make G_{1,2} same dimensions as G₂

Columns for stair.

C₃ Third floor, rectangular section.

$$\text{Live load from roof } \frac{(21)(21)(30)}{4} = 3307.5 \text{ lbs.}$$

$$\text{Dead load of slab } \frac{(21)(21)(121)}{4} = 13340.5 "$$

$$\text{Dead load, } \frac{1}{2} b, \frac{(21)(103)}{2} = 1081.5 "$$

$$\text{" " } \frac{1}{2} G_{1,0} \frac{(21)(172)}{2} = 1806.9 "$$

$$\text{" " } b_s \frac{(6)(12)(150)(7)}{(12)(12)} = 525.0 "$$

$$\text{Pent house walls and roof, } \frac{9'' \text{ wall}}{\text{total}} = \frac{20050.0}{40110.5} "$$

$$\text{Parapet wall } (21)(283) = \frac{5946.0}{45956.5} "$$

$$\text{Load on one roof beam at wall } \frac{(1361)(21)}{2} = 14290 \text{ lbs.}$$

$$\text{Eccentric moment of girder } \frac{(14290)(7)(12)(2)}{10} = 240072 \text{ in. lbs.}$$

$$\text{Eccentricity in column } 60018 \text{ in. lbs.}$$

$$\text{Eccentricity } \frac{60018}{45956} = 1.3"$$

Column size, see Page 151, chart, 11" x 11".
 use 1½" cover for fire protection, 14" x 14" out to out.
 $a_s = (14^2)(1 \text{ per cent}) = 1.96 \text{ sq. in.}$

use 8 - 9/16" rods

use ½" bands at 12" c-c

$$\text{wgt. of column} = \frac{(14)(14)(150)(11)}{144} = 2245 \text{ lbs.}$$

Make column same dimensions as C¹

Make footing F₂ same dimension as F₁

Columns C₃₃ and C₃₄ for stairway and elevator penthouse.

$$\text{Live load at top} = \frac{(14)(14)(30)}{4} = 1470 \text{ lbs.}$$

$$\text{Dead load slab} = \frac{(14)(14)(121)}{4} = 5929 "$$

$$\text{Walls} = \frac{(14)(0.5)(150)(6.5)}{2} = 6825 "$$

$$\frac{1}{2}b, \text{ load} = 2788 "$$

$$\frac{1}{2}G, \text{ load} = 18356 "$$

$$\text{Total load on column} = 35368 \text{ lbs}$$

3rd floor,

$$\text{short diameter} = \sqrt{\frac{35368}{(2)(800)(0.414)}} = 7.3"$$

use 1½" cover for fire protection, 10.5" octagonal

$$A = (2)(7.5^2)(0.414) = 46.58 \text{ sq. in.}$$

use 1 per cent steel, 0.4658 sq. in.

4 - ½" round rods at 0.78 sq. in.

$$\text{weight of column} = \frac{(2)(10.5)(0.414)(11)(150)}{144} = 1125 \text{ lbs.}$$

use ½" spirals at 2" c-c

2nd floor,

$$\text{3rd floor column load at roof} = 35368 \text{ lbs.}$$

$$\text{3rd floor column} = 1125 "$$

$$\text{live load at floor} = \frac{110250}{8} = 13780 "$$

$$\text{Dead load of floor} = \frac{24696}{8} = 3087 "$$

$$\frac{1}{2}b, \text{ stem} = 525 "$$

$$\frac{1}{2}G, \text{ stem} = 2040 "$$

$$\text{total load} = 55925 "$$

$$\text{Short diameter} = \sqrt{\frac{55925}{(2)(800)(0.414)}} = 9.5"$$

1½" cover for fire protection, 12.5" out to out.

$$A = (2)(9.5^2)(0.414) = 74.73 \text{ sq. in.}$$

1 per cent steel, 0.74, use 4 - $\frac{1}{2}$ " rods

$$\text{weight of column, } \frac{(2)(12.5^2)(0.414)(11)(150)}{144} = 1500.0 \text{ lbs.}$$

use $\frac{1}{4}$ " spiral bands at 2" c-c

1st floor column,

Load from second floor 55570 lbs.

2nd floor column 1500 "

Floor loads 19432 "

Total load 76502 "

$$\text{Short diameter, } \sqrt{\frac{76500}{662.4}} = 11.4"$$

1 $\frac{1}{2}$ " for fire protection, 14" out to out

$$A = (2)(11^2)(0.414) = 100.184 \text{ sq. in.}$$

1 per cent for steel, 1.00 sq. in.

use 4 - 1" sq. rods

$$\text{Weight column } \frac{(2)(14^2)(0.414)(11)(150)}{144} = 1887 \text{ lbs.}$$

use $\frac{1}{4}$ " spirals at 2" c-c

Basement column.

Load at first floor, 76500 lbs.

1st Floor column 1890 "

Floor load 19432 "

Total 97822 "

$$\text{Short diameter } \sqrt{\frac{97822}{662.4}} = 12.1"$$

Use 1 $\frac{1}{2}$ " cover for fire protection, 15" out to out

$$A = (2)(12^2)(0.414) = 119.23 \text{ sq. in.}$$

1 per cent for steel, 1.19 sq. in.

use 4 - 1 $\frac{1}{2}$ " sq. rods

$$\text{weight of column, } \frac{(2)(15^2)(0.414)(9)(150)}{144} = 1772 \text{ lbs.}$$

use $\frac{1}{4}$ " spirals at 2" c-c

Weight on footing.

Load at top of basement column 97822 lbs.

Basement column 1772 "

total on footing 98594 "

Footings F_{33} and F_{34}

Load at top of footing 98600 lbs.

15 per cent for footing 14790 "

Soil pressure 113390 "

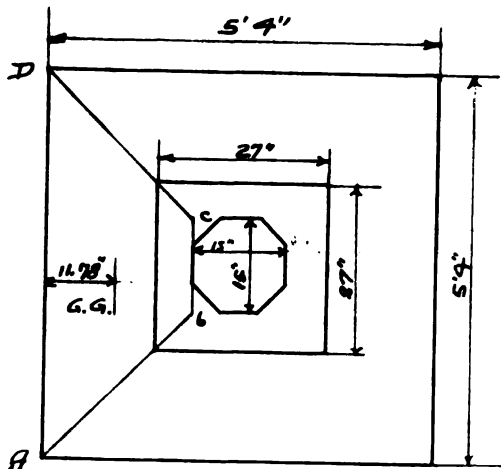
area required $\frac{113390}{4000} = 28.4$ sq. ft.use $5'4" \times 5'4" = 28.4$ sq. ft.

area of column base, 119.23 sq. in. or 0.83 sq. ft.

Area abcd = 28.4 sq. ft.

0.834) 27.57

6.89 sq. ft.



upward pressure

 $(4000)(6.89) = 27650$ lbs.

distance c.g. from edge of footing,

 $\frac{18.5(27+(2)(64))}{3(17+64)} = 11.78"$ $M = (27560)(11.78) = 324657$ in. lbs.depth for M $\sqrt{\frac{324657}{(107.4)(27)}} = 19.5"$

Depth for punching shear,

 $\frac{27560}{(27)(105)} = 9.9"$

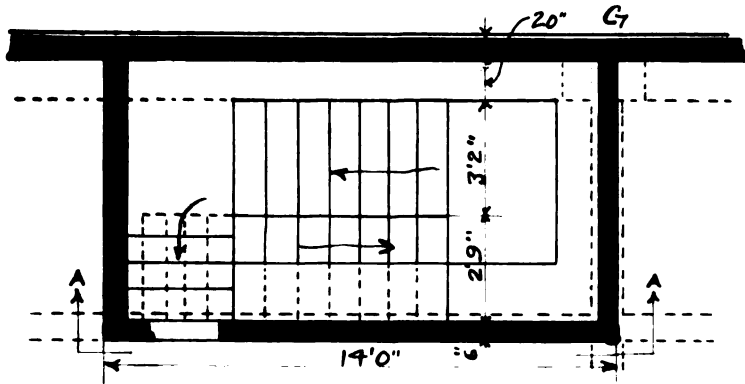
total depth 12"

 $\Sigma_o = \frac{27560}{(80)(0.875)(10.5)} = 37.5"$ $\frac{37.5}{3.142} = 12, 1"$ rods neededspacing $\frac{27+(2)(10.5)+13}{12} = 5.+" c-c$ weight of footing $(5.25)(5.25)(1)(150) = 4260$ lbs.

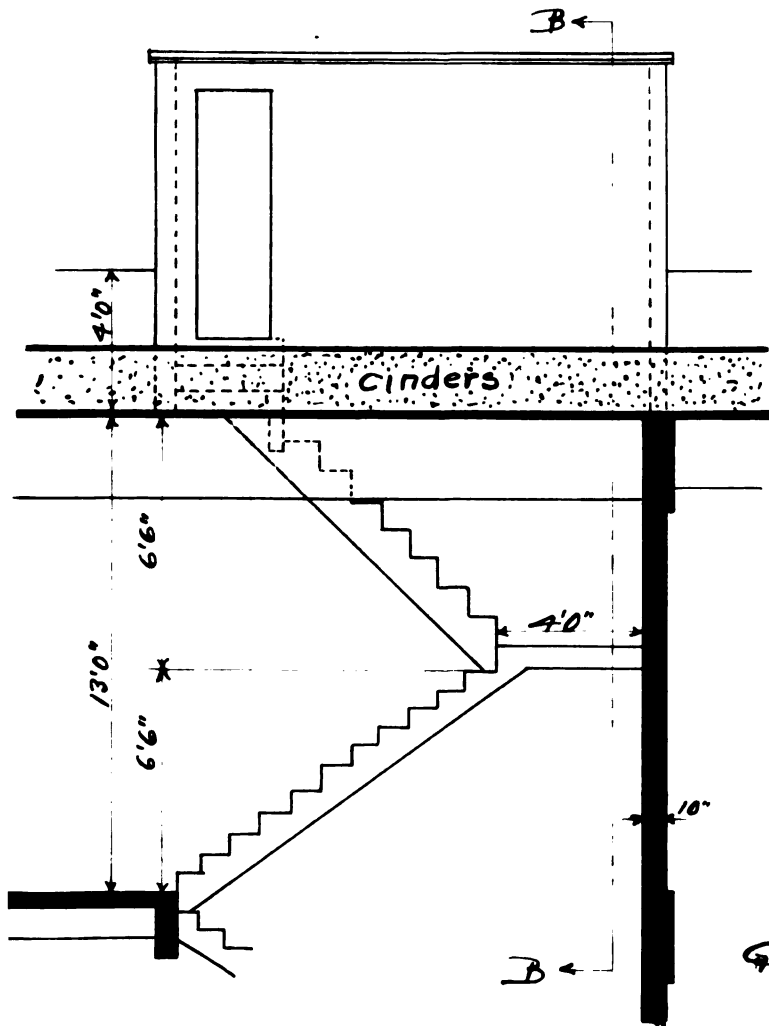
FOOTING SCHEDULE.

Mark	Size	No. Footings	Steel			
			No.	Size	Length	Total
INTERIOR	13'6" X 13'6"	36	16	1" ϕ	13'0"	576
F ₂ F ₃ F ₄ F ₅ F ₆ F ₇	10'6" X 10'6"	12	14	1" ϕ	10'0"	168
F ₁₆ , F ₂₄ , F ₃₂	10'6" X 10'6"	6	14	1" ϕ	10'0"	84
F ₉ , F ₁₇	10'9" X 10'9"	4	14	1" ϕ	10'3"	56
P	3'4" X 3'4"	6	8	1" ϕ	3'0"	48
F ₂₅	10'6" X 10'6"	2	14	1" ϕ	10'0"	28
F ₁ , F ₈	7'7" X 7'7"	4	10	1" ϕ	7'0"	40
F ₃₃ , F ₃₄	5'4" X 5'4"	4	12	1" ϕ	5'0"	48

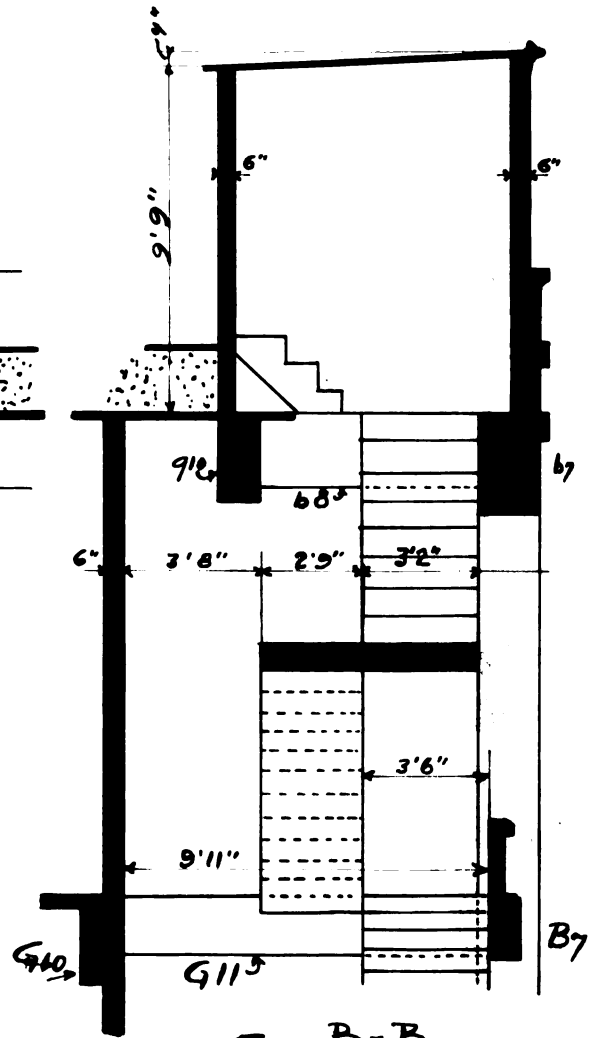
Stair Details.



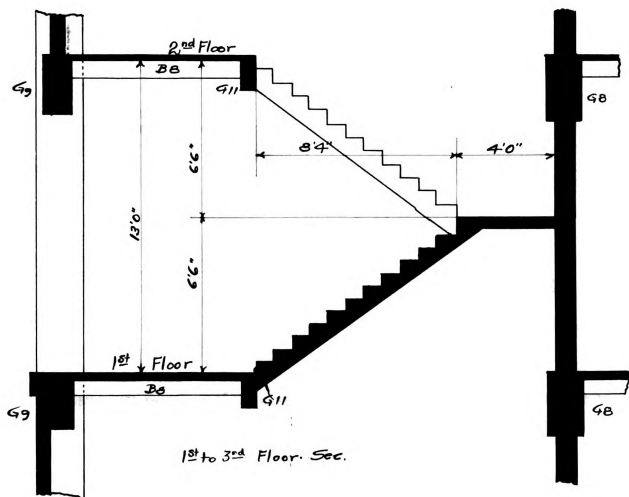
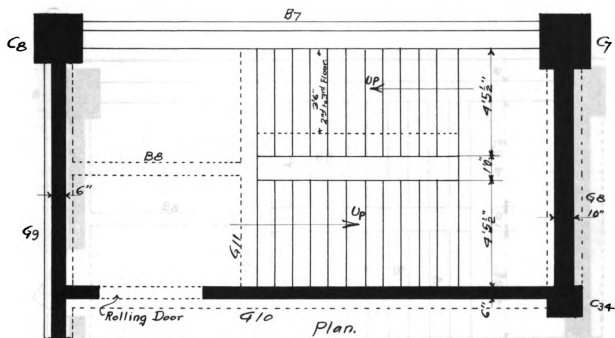
Section thru Stair House
& Parapet.



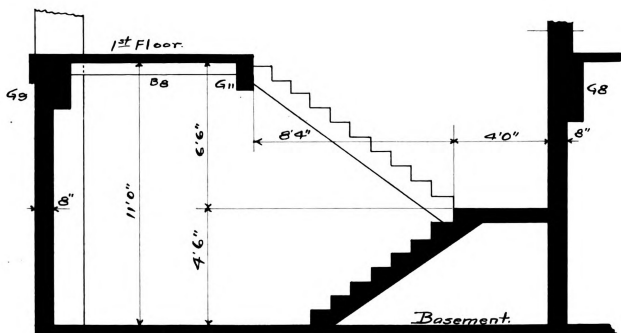
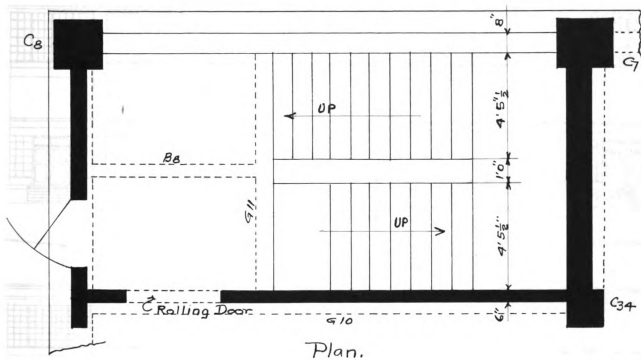
Sec A-A.

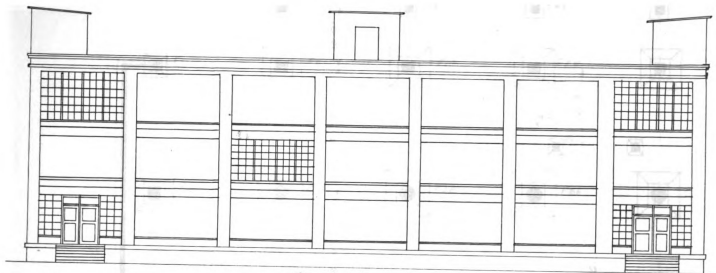


Sec B-B.

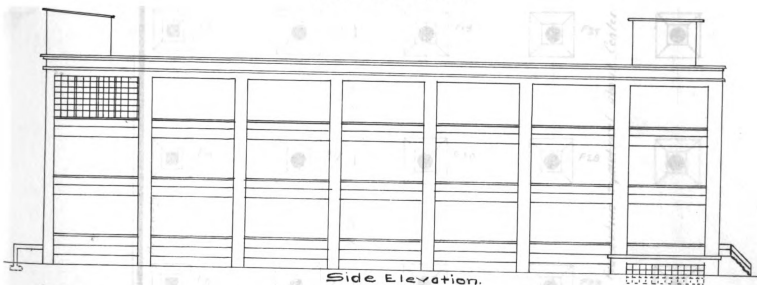


Stair Details

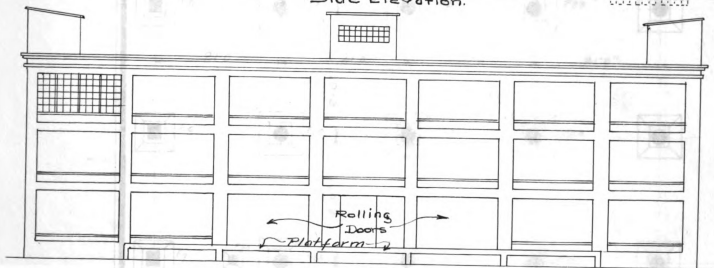




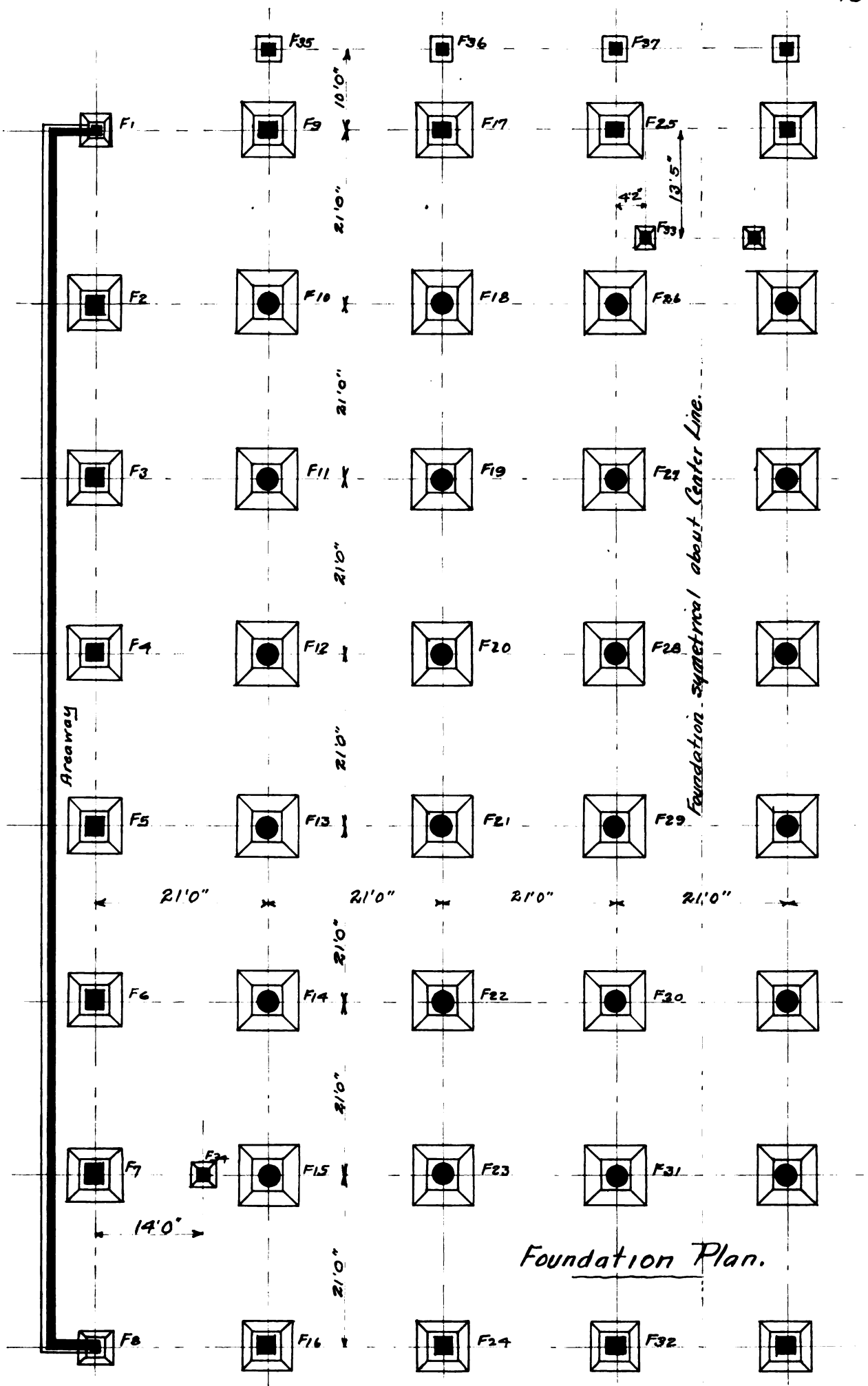
Front Elevation.



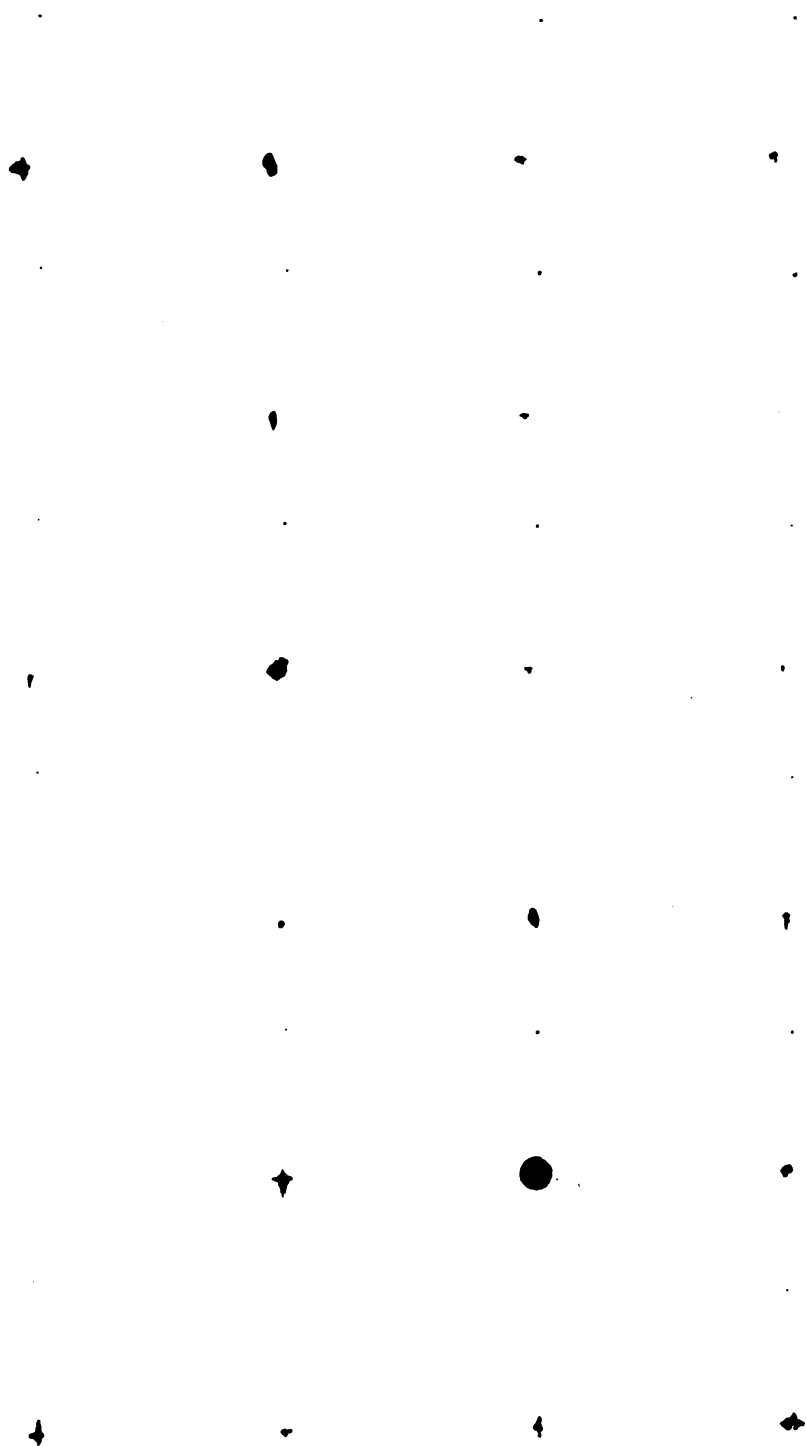
Side Elevation.

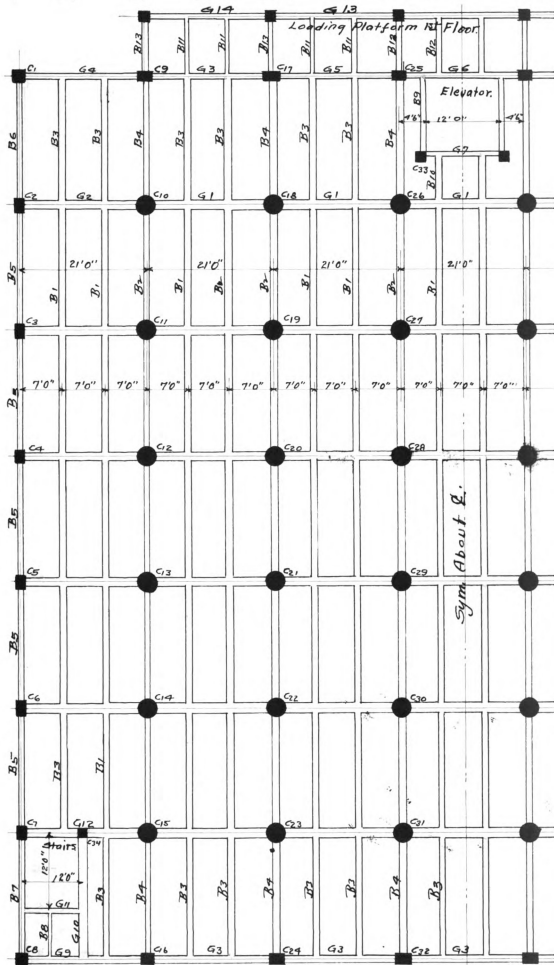


Rear Elevation.

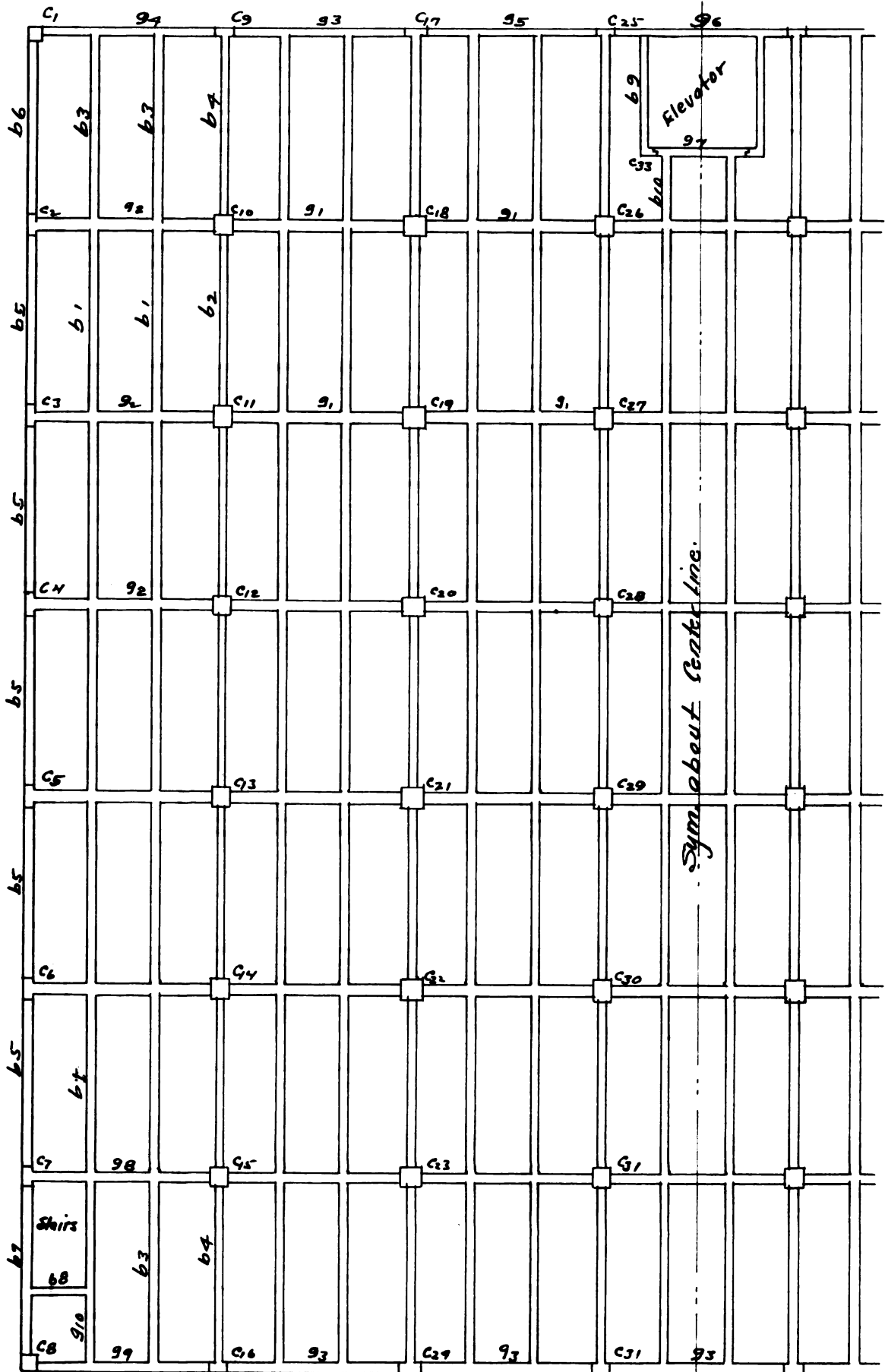


Foundation Plan.

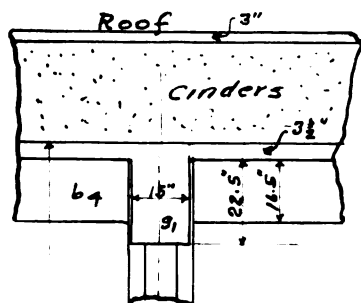




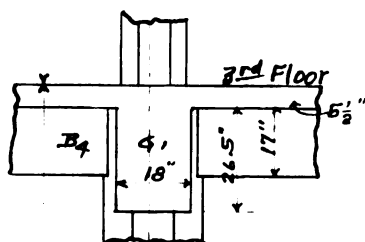
Floor Framing Plan.



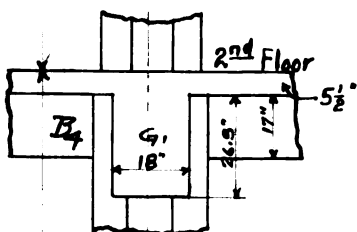
Roof Framing Plan.



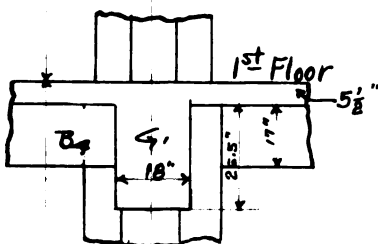
13'0"
12'5"
8-1/2" ϕ Vert.
1/4" Spiral 2" c-c



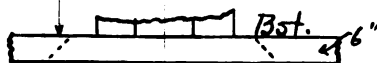
13'0"
12'1"
8-13/16" ϕ Vert.
1/4" Spiral 2 1/2" c-c



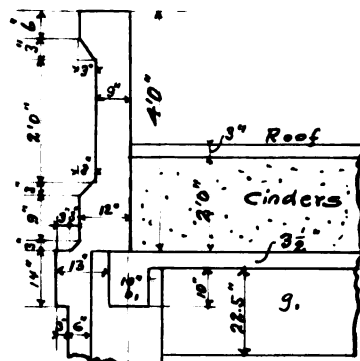
13'0"
12'4"
8-1" Vert.
1/4" Spiral 2 1/2" c-c



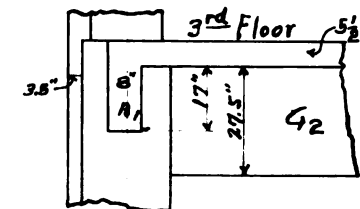
11'0"
10'5"
8-1 1/4" ϕ Vert.
1/4" Spiral 2 1/2" c-c



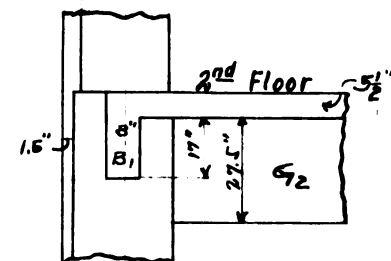
Interior Cols.
Excepting C33, C34



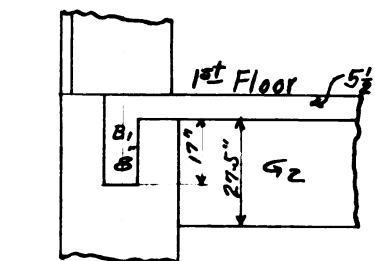
13'0"
12'5"
8-11/16" ϕ Vert.
1/4" bands - 12" c-c



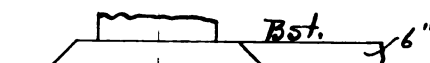
13'0"
12'1"
8-13/16" ϕ Vert.
1/4" bands - 12" c-c



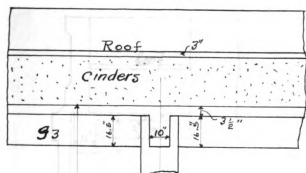
13'0"
12'4"
8-15/16" ϕ Vert.
1/4" bands - 12" c-c



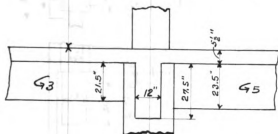
11'0"
10'5"
8-1 1/8" ϕ Vert.
1/4" bands - 12" c-c



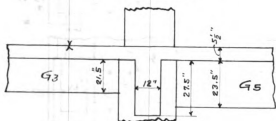
Exterior Cols.
C2, C3, C4, C5, C6, C7



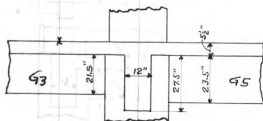
13'0"
13"
16"
8- $\frac{5}{8}$ " ϕ vert.
 $\frac{1}{4}$ " hoops 12"cc



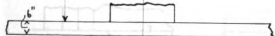
13'0"
21"
24"
8- $\frac{3}{8}$ " ϕ vert.
 $\frac{1}{4}$ " hoop 12"cc



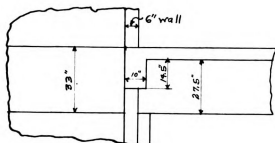
13'0"
24.5"
24.5"
8-1" ϕ vert.
 $\frac{1}{4}$ " hoops 12"cc



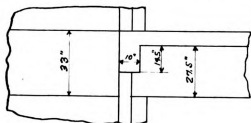
11'0"
27.5"
30.5"
8-1 $\frac{1}{8}$ " ϕ vert.
 $\frac{1}{4}$ " hoops 12"cc



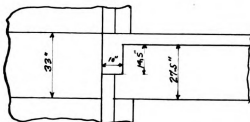
G9, C17, C25.
Basement section
for C16, C24, C32
21' x 31.5'



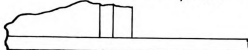
6" wall
33"
10"
18.5"
27.5"
4- $\frac{1}{2}$ " ϕ vert.
 $\frac{1}{4}$ " spiral 2"cc



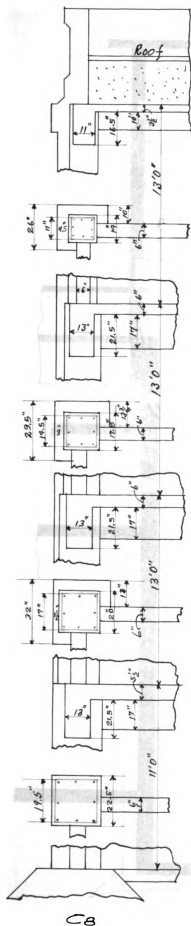
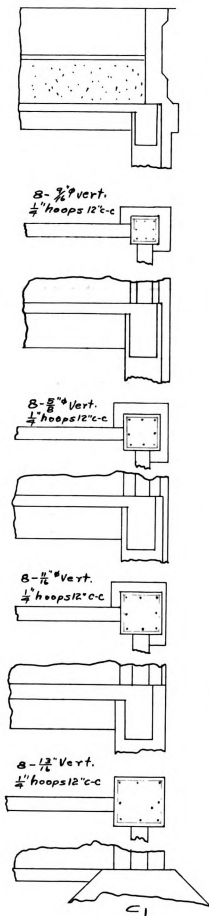
33"
10"
18.5"
27.5"
4-1" ϕ vert.
 $\frac{1}{4}$ " spiral 2"cc



33"
10"
18.5"
27.5"
4-1 $\frac{1}{8}$ " ϕ vert.
 $\frac{1}{4}$ " spiral 2"cc



C34

 C_8 

8- $\frac{9}{16}$ " vert.
 $\frac{1}{4}$ " hoops 12" c-c

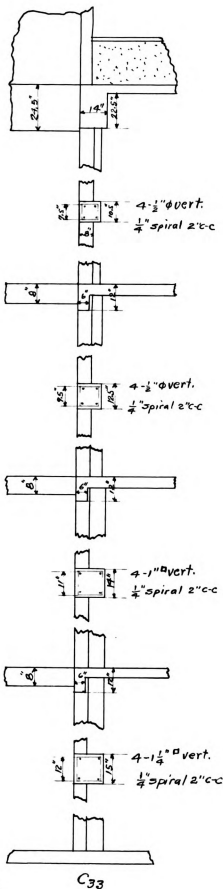
$\frac{1}{4}$ " hoops 12" c-c

8-5/8" ϕ Vert.

$\frac{1}{4}$ " hoops 12" c-c

$$B = \frac{n}{k} \cdot \theta \cdot V_{\text{ext.}}$$
 $\frac{1}{2} \text{ h } 000512^{\circ} \text{ C-C}$

g - 13th Vert.

 $\frac{1}{2}$ " hoops 12" c-c

4- $\frac{1}{8}$ " ϕ vert.

$\frac{1}{2}$ " spiral 2'x-c

4 - 1" ϕ vert.

$\frac{1}{2}$ " "Zinc / 2" C.C.

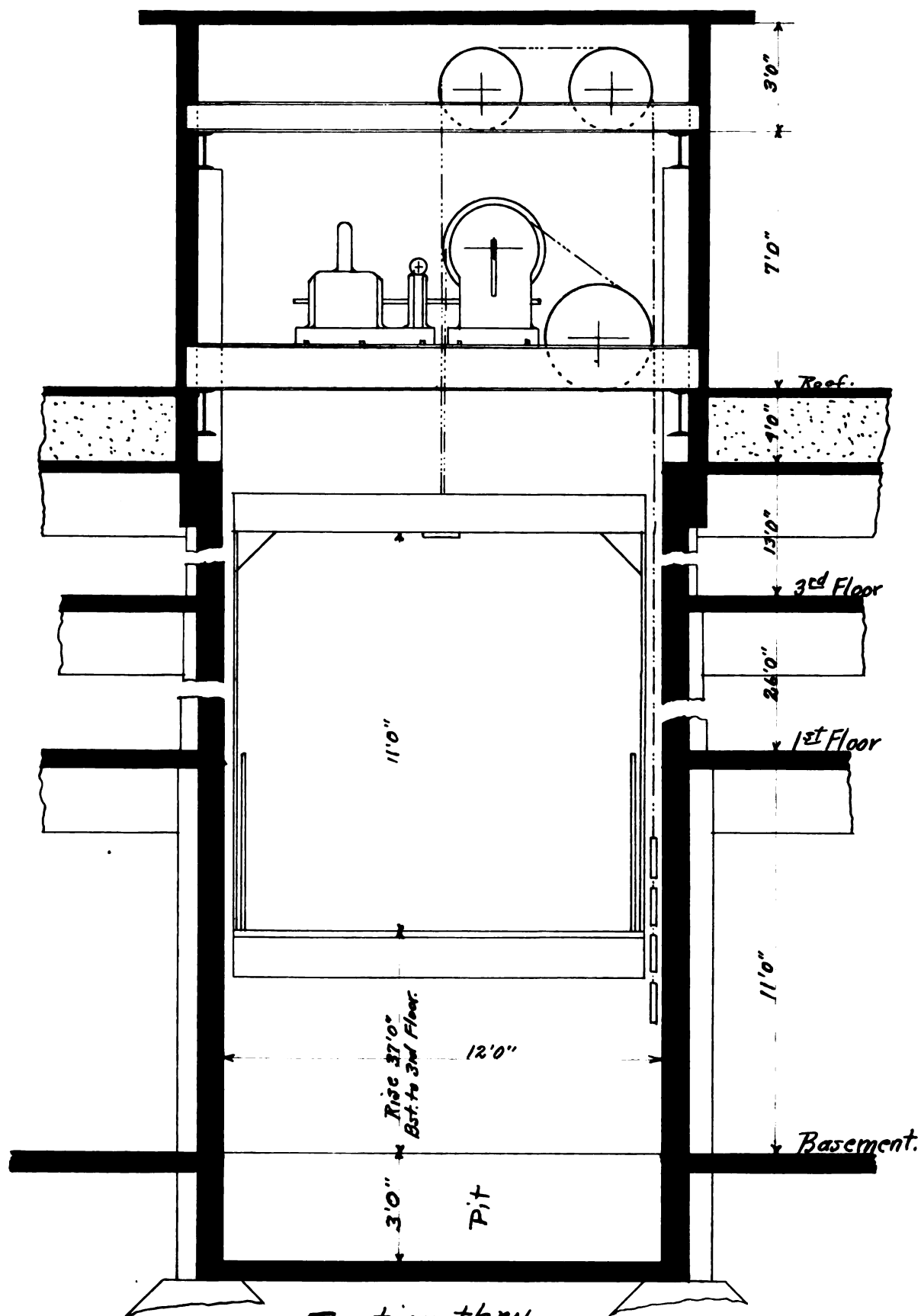
4-1" $\square$ vert.

$\frac{1}{2}$ " spiral 2" c-c

4-1 $\frac{1}{2}$ " $\square$ vert.

19

C33



Section thru
Elevator Shaft.

Page location of Details for the
building design of Prob. 29.

ROOF.

Roof Slab.

Slab details	Page 119
Slab steel.....	" 119

Roof Beams.

b ₁ , b ₂ , b ₃ , b ₄	" 121
b ₅	" 122
b ₆	" 123
b ₇	" 173-123
b ₈	" 173
b ₉	" 167
b ₁₀	" 168

Roof Girders.

g ₁	" 123
g ₂	" 125
g ₃ , g ₄ , g ₅	" 126
g ₆	" 169
g ₇	" 168
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g ₉	" 173-126
g ₁₀	" 173-121

Steel Schedules.

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FLOOR

Slab Details.....	" 130
-------------------	-------

Floor beams.

B ₁	" 130
B ₂	" 131
B ₃ , B ₄	" 132
B ₅ , B ₆	" 133
B ₇	" 175-170
B ₈	" 173
B ₉	" 170
B ₁₀	" 171
B ₁₁ , B ₁₂ , B ₁₃	" 134

Floor Girders.

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G ₁₂	"	175-137
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Steel Schedules.

Slab steel.....	"	130
Beam and Girder.....	"	144-145- 146-147- 148

COLUMNS.

Details	"	188-189- 190
Steel scheduled with column details.		

FOOTINGS.

F ₁	"	164
F ₂ , F ₃ , F ₄ , F ₅ , F ₆ , F ₇	"	158
F ₈	"	164
F ₉	"	160
F ₁₀ , F ₁₁ , F ₁₂ , F ₁₃ , F ₁₄ , F ₁₅	"	157
F ₁₆	"	158
F ₁₇	"	160
F ₁₈ , F ₁₉ , F ₂₀ , F ₂₁ , F ₂₂ , F ₂₃	"	157
F ₂₄ , F ₂₅	"	158
F ₂₆ , F ₂₇ , F ₂₈ , F ₂₉ , F ₃₀ , F ₃₁	"	157
F ₃₂	"	158
F ₃₃ , F ₃₄	"	178

Steel Schedules.

For all column footings.....	"	179
Elevator house details.....	"	166
Elevator shaft details.....	"	191
Stair details.....	"	180-181- 182

The chapter on Building design and specifications gives a fairly comprehensive idea of the requirements of a building design from the architectural point of view and from the point of view of the man who places or figures on the inside fixtures and equipment of the building. These things, however, are a function of each individual designing office, each unit as it works, having its own methods and systems of planing and detailing. There are certain basis things which are needed upon a drawing, beyond this point the amount of detail which is placed upon a sheet is determined by the personal equation of the office.

The specifications there stated are but general. In every case of a building design, the specifications written must conform to the building code of the locality in which the structure is to be erected. For this reason, no set of specifications presented can claim to be more than a form which may be followed with more or less degree of success.

The chapter on building materials is but a toiled down repetition of what has been presented in many other works on cement and concrete.

The chapter on forms and form construction is well written, it lacks in giving the necessary equations to be used in determining the strength of the various timbers used in the form construction. The author evidently assuming that the book would not be used as a reference book by any but those who would be familiar with the mathematics necessary for testing the strength of a piece of timber which is to be used in the forming.

The use of the steel form has advanced at so rapid a rate during the last few years, in fact since the time of the publication of this work on concrete, that the information here given on steel forming is very meager. Steel forms are undoubtedly the coming form for all classes of concrete work where the different units can be standardized. The use of the steel form is more economical, more satisfactory and turns out a much better looking job.

Proposition 30. Page 816h.

Determine the maximum spacing of 2x6 joists which are to span 6' and support a 6" slab having a construction live load of 75 pounds per square foot.

$$M = \frac{wl^2}{8}$$

Allowable fiber stress in joists and sheathing 1200 pounds per square inch. Deflection limited to 1/4"

$$M = \frac{(150)(6)^2(12)}{8} = 8100 \text{ per ft. of width.}$$

$$M = \frac{bhd^2}{6s}, \quad s = \frac{bhd^2}{6M}$$

$$s = \frac{(1200)(2)(6)^2}{(6)(8100)} = 1.72' \text{ or } 1' 8\frac{1}{2}" \text{ Spacing.}$$

$$D = \frac{5}{384} \frac{wl^4}{EI}$$

$$D = \frac{(5)(150)(1.72)(72)^3(12)}{(384)(120000)(2)(6)^3} = \frac{278.64}{12000} = 0.02" \text{ deflection.}$$

Proposition 31.

A wall form is filled with concrete at the rate of five vertical feet per hour. What is the required spacing of studs, assuming 1 1/2" sheathing and a temperature of 70°. Using same M and same unit stresses as in 30.

70° temperature is equivalent of 990 lbs. pressure.

$$M = \frac{(990)(5)^2(12)}{8} = 37125 \text{ in. lbs.}$$

allowable deflection 1/4"

$$D = \frac{8}{384} \frac{wl^4}{EI}$$

$$\frac{1}{4} = \frac{8}{384} \frac{(990)(s)(5)^4(12)}{(120000)(2)(6)^3}$$

$$s = \frac{(384)(120000)(2)(6)^3}{(4)(8)(990)(5)^4(12)(12)} = 1.4' \text{ or } 1' 5" \text{ spacing.}$$

Proposition 32.

Find the maximum spacing of 4x4 posts to support the forms for a 19"x28" beams, 7' c-c with 5½" slab.

Use 500 lbs. per sq. in. allowable stress in posts.

Construction load 75 lbs. sq. ft.

$$\begin{array}{rcl}
 5\frac{1}{2}" \text{ slab weight, } (\frac{5}{12})(7)(150) & = & 481.25 \text{ lbs. per ft. B} \\
 10 \times 28 \text{ beam, } (\frac{10}{12})(\frac{28}{12})(150) & = & 291.66 \text{ lbs. per ft. B} \\
 \text{Construction load, } (75)(7) & = & 525.00 \text{ lbs. per ft. B} \\
 \hline
 \text{load per ft. of beam} & & 1297.91 \text{ lbs.}
 \end{array}$$

a 4"x4" post will carry.

$$4 \times 4 \times 500 = 8000 \text{ lbs. compression.}$$

call load per ft. of beam 1300 lbs.

$$\text{post spacing } \frac{8000}{1300} = 6.1', \text{ space 6' c-c}$$

Proposition 33.

Determine the maximum spacing of joists and stringers supporting forms for an 8" slab floor, flat slab. Construction live load of 75 lbs. sq. ft. Assume 1½" sheathing and 2"x8" joists. Assume that the sheathing and joists are partially continuous.

Allowable timber stress 1200 lbs. sq. in.

Maximum deflection allowable ¼".

$$E = \frac{wl^2}{10}$$

$$\text{Weight 8" slab } (\frac{8}{12})(150) = 100 \text{ lbs. sq. ft.}$$

$$\text{Live load per ft. } 75 \text{ lbs.}$$

$$\text{Total load per ft. } 175 \text{ lbs.}$$

4"x4" post will carry 8000 lbs.

$$175l^2 \neq 8000 \quad l^2 = \sqrt{\frac{8000}{175}} = 6.7', \text{ space 6'6" c-c}$$

$$E = \frac{(175)(6.7)^2(12)}{10} = 8872.5 \text{ in. lbs.}$$

$$s = \frac{(1200)(2)(6.7)^2}{(8)(8872.5)} = 4' \text{ p-c}$$

space the joists 3'6" to center with posts.

The Chapter on Bending and Placing of Reinforcement contains nothing new; the things there stated have been stated many times before in never every treatise or volume which has been written having to do with reinforced concrete work. For the purpose of the student who is reading the volume, it would have been well to have here stated in such a way as to make a lasting impression, that all commercial sizes of steel should be bought in straight pieces, in as large multiple lengths as are commercially rolled, to be cut and bent upon the job. Only the larger sizes of steel are ordered to exact length and bent by the mills, these sizes being of such stiffness and weight that it would be impractical to bend them on the job.

The Chapter on Mixing and Placing of Concrete is what has been repeated over and over in all works on written literature on Concrete Construction. Is or should be common knowledge to all who are working in concrete.

The Chapter on Finishing of Concrete surfaces is probably as near up to date as it could be made at the time the volume being used was sent to press, namely 1911. Since that time considerable advance has been made in the methods of finishing concrete surfaces, principally in the use of electrically driven small one man cutting machines, for both floors and walls.

The Chapter on Waterproofing of Concrete might, to the writers idea, have well been omitted. The subject and the methods of waterproofing is still under much discussion and there seems to be a consensus of opinion that the adding of foreign substances to the concrete mass is very bad for the mix, unless the same is done with great care. The addition of foreign substances having the tendency to weaken the mix. To the writers idea the best water proofing is done by a well graded and well mixed concrete of sand and gravel or stone, well roamed and compacted in the form.

The Chapter on Construction Plans, is again, but every individual case of construction requires its own study for construction plans, therefore it is well that the chapter be limited to but a few general remarks.

Proposition 34.

What items should be charged up against construction work for the plant involved?

First cost of plant.

Breight on plant.

Cartage and hauling to site.

Exection at site.

Piping for water.

Cost of water.

Runways, platforms, etc.,

Towers,

Temporary builddings,

Bins,

Canvas,

Fuel,

Operation costs,

Maintenance costs,

Moving plant to new locations on site,

Interest on investment,

Insurance on plant.

Depreciation,

The estimated salvage value of the plant should be considered when adjusting the percentages to be used in carrying the plant costs into the estimate figures.

Proposition 35.

Enumerate the various operations involved in the erection of the reinforced-concrete skeleton of an office building.

All preliminary investigations having been made, specifications written, contract let, contractor upon the ground.

1. Excavation of basement, foundations, footings, elevator pits, boiler rooms, etc.

2. Erection of concreting plant at a position of maximum utility. Where the greatest amount of concrete can be placed with the least effort and loss of time.

3. Pour footings.

4. Erection of outside curtain wall forms.

5. Erection of basement column forms,

6. Place wall reinforcement.

7. Pour outside walls.

8. Lay forms for first floor.

9. Place column reinforcing.

10. Place floor system reinforcing.

11. Pour concrete columns and floor slabs.

12. Place forms, steel and pour stairways.

13. Place first floor column forms.

14. Place first floor outside column forms.

15. Place second floor floor forms.

16. Place reinforcing steel for first floor columns.

17. Place second floor steel.

18. Pour first floor columns,

19. Pour second floor slabs.

20. Place forms, steel and pour stairways for first to second floor.

Continue similarly for each floor to roof. To save form material, column spacing and floor slabs of the various floors should be standardized so far as possible. Then within the proper limit of time for the setting of concrete, the floor forms, beam forms, girder forms, and column forms, may be stripped and used again on floors above.

Proposition 36.

Stock piles should be so located that material will be available to the workmen, and to the plant with the least amount of labor. The materials passing from stock pile to mixing plant, carpenter plant, or bending plant; then on to the building always with progressive motion, no retracements of doubling back.

The stock piles should be easy of access to the trucks, or other units bringing in the materials. The proper study for placing of stock piles with reference to the job as a whole will result in cutting considerable from the cost of handling.

Proposition 37.

The considerations which should govern the actual laying out of a construction plant should be; the shape and size of the area over which work is to be done; the contour and slope of the ground surface; the means of ingress and egress to and from the site for the hauling units; the distance from the receiving points of shipments; the type of construction to be done, whether compact, or spread out over a considerable area; a compact construction job would be served by derricks, or by tower and short chutes; a spread area, by tower and chutes, or by a cableway unit. The total garbage must be considered. The time limit for the work, and the bonus or penalty. The time of year must be considered, especially in the north section, where there is very apt to be unfavorable weather conditions.

Proposition 38

The sole purpose of the design and use of the construction plant, is economy of operation, maximum of output at the minimum of cost; all things affecting the work being considered.

Proposition 39.

Economic plant design requires that the plant best suited to the work, regardless of cost, be the one installed. First cost should not be the deciding factor. Cost of installation, maintenance, operation, removal, interest on investment, all these should be considered for each type of machine and plant layout; these together with the differences in the saving in labor and the salvage value of the plant when the contract is completed. The higher priced plant, which under the conditions of operation will deliver more yardage - though it be at a somewhat higher cost per yard - may due to the greater salvage value of the units be the more economic plant for use on the contract. The plant should be of a size and capacity which will permit of continuous operation, for only by the maximum of operation can the yardage costs be reduced to a minimum.

Proposition 40.

The essentials of proper mixing are; exact measurement of gravel or stone, sand and cement; thorough mixture of the mass; proper amount of water; care in dumping the concrete into place. The proper ramming or puddling of the concrete is of importance.

With hand mixing there should be a proper mixing of the sand on the cement before the gravel or stone and the water are added. Constant vigilance in the mixing is here a necessity.

With machine mixing, proper proportioning, proper supply of water, proper number of turns of the drum to insure thorough mixing are necessities.

In mass work minute care need not be taken in the placing. In thin walls, great care must be exercised not to displace the reinforcing in the pouring or by the use of too large aggregate. In columns, care should be exercised to keep the coarse aggregate from lodging between the steel and face of the column, where it would be exposed upon stripping the forms. In floor systems care should be exercised that no pockets of coarse material be formed in beams and girders, especially that none be formed at the joints of beams, girders and columns.

In bad weather the mix must be protected, from rains when freshly laid in summer; from freezing and frost action in fall and winter.

Section 3, deals with estimating but only in a very light way. Chapters 15, 16, and 17 give the reader some slight idea of the methods and ways of estimating unit costs and preparing the estimate of quantities and costs. The art of estimating can only be acquired by long practice, for though the process of taking off the main quantities is relatively a simple matter, the great danger of underestimating comes, not from those things which can be readily seen to be a part of the job, but from the oversight and non-estimation of the many smaller miscellaneous items, which in the aggregate make up an amount of money which under the contract may be the entire percentage of gain and profit that the contractor has been figuring upon. Construction plant, overhead, contingencies, insurance, taxes, premiums, accidents, protracted runs of bad weather, are some of the things which the unexperienced estimator is prone to miss or underestimate.

The units of cost which will be used in preparing the estimate of the building which is called for in Proposition 41, on the building designed as a part of this thesis, see page 118, are given below. These prices are based upon material prices, labor prices, and construction costs as of December 1st, 1920, Washington, D.C.

Concrete.

Cement at mill, Bbl.	4.08
Frieght	0.10
Cloth sacks	0.40
Per bbl. f.o.b. job	4.58
Deduct credit for sacks	0.40
	4.18
Testing	0.04
Unloading	0.04
Teaming	0.02
Bundling and tying, bags	0.03

Net cost per bbl. f.o.b. job 4.45

Sand, 1.40 per ton = 1.30 per cu. yd.

Crushed stone, 1.75 per ton = 2.36 per cu. yd.

Sand 2.00 per cu. yd. on job.

Crushed stone 2.50 per cu. yd. on job.

Mass concrete, footings, etc.

Cement, 1.66 bbl. at 4.45	7.41
Sand, one-half cu. yd. at 2.00	1.00
Stone, one cu. yd. at 2.50	2.50
Labor, per yd.	3.00
Plant, per yd.	1.50
	15.41 per cu. yd.
Walls, columns, floors,	16.00 per cu. yd.

FORMS.

Forms for floor slabs:

Lumber, nails, oil, etc.	0.06
Labor, making panels,	0.02
Labor, erecting studs and bracing,	0.04
Labor, laying panels,	0.04
Stripping,	0.02
	0.18 per sq. ft.

Forms for beams and girders.

Lumber, nails and oil, etc.,	0.06
Labor making,	0.04
Labor erecting,	0.08
Labor stripping,	0.02
	0.20 per sq. ft.

Forms for columns:

Lumber, nails, oil, etc.	0.06
Labor, making panels,	0.03
Labor, erecting, plumbing, bolting,	0.14
Labor stripping,	0.03
	0.26 per sq. ft.

Forms for footings,

Lumber, nails, oil, etc.,	0.06
Labor, making and erecting,	0.10
Labor stripping,	0.02
	0.18 per sq. ft.

Forms for walls,

Lumber, nails, oil, etc.,	0.06
Labor, making,	0.04
Labor, erecting and plumbing,	0.10
Labor, stripping,	0.02
	0.22 per sq. ft.

Steel prices.

1/4" to 5/16"	4.13 per 100 lbs.
3/8" to 7/16"	3.88 per 100 lbs.
1/2" to 9/16"	3.73 per 100 lbs.
5/8" to 11/16"	3.68 per 100 lbs.
3/4" and up	3.63 per 100 lbs.

Unloading, hauling and piling steel at job,	1.00 ton
Bending and placing steel in bldg.- floors	10.00 ton
Bending and placing steel in bldg.- beams, girders	13.00 ton
Bending and placing steel in bldg.- columns,	15.00 ton
Placing steel in walls,	13.00 ton,
Average price steel in place,	14.00 ton.

Surface finish.

1" Granolithic floor per 100 sq. ft.	
1 bbl. cement	4.45
4 cu. ft. sand,	0.32
400 lbs. fine crushed stone	0.40
	5.17
Labor, mixing and placing,	1.20
Finisher troweling surface,	1.50
	7.87 per 100 sq. ft.

Estimate for a three story and basement building,
 147' x 147'. Factory window. Cinder and concrete roof.
 Area, $4 \times 147 \times 147 = 86,436$ sq. ft.
 Cube, $147 \times 147 \times 52.75 = 1,129,675$ cu. ft.

Concrete, exterior footings, (1:3:6 mix)

$20 \times 10.5 \times 10.5 \times 3.25 = 7166$ cu. ft.
 $4 \times 7.6 \times 7.6 \times 2.4 = 5552$
 $4 \times 10.75 \times 10.75 \times 3.2 = 1464$
 $6 \times 3.33 \times 2.22 \times 1 = 67$

Interior footings.

$36 \times 13.5 \times 13.5 \times 4.6 = 30071$
 $4 \times 5.33 \times 5.33 \times 1 = 114$

 $44441 = 1646 \text{ c.y. at } 15.40 = 16348.00$

Concrete foundation walls, (1:2:4)

$147 \times 0.66 \times 10 = 980$ south cu. ft.
 $147 \times 0.66 \times 10 = 980$ north
 $147 \times 0.66 \times 10 = 980$ east
 $147 \times 0.66 \times 3 = 284$ west
 $147 \times 1 \times 1 = 147$ cap west wall

 $3381 = 148 \text{ c.y. at } 16.00 = 2368.00$

Concrete retaining wall (1:3:6)

$2 \times 1 \times 8 \times 155 = 2480$ cu. ft.
 $1 \times 0.5 \times 155 = 77.5$

 $2557.5 = 95 \text{ c.y. at } 15.40 = 1463.00$

Concrete, exterior columns, (1:2:4)

Base. $2.4 \times 2.4 \times 11 \times 12 = 668$
 1st. $2.2 \times 2.2 \times 13 \times 12 = 792$
 2nd $1.9 \times 1.9 \times 13 \times 12 = 584$
 3rd $1.4 \times 1.4 \times 13 \times 12 = 312$
 Base $2.5 \times 2.5 \times 11 \times 6 = 413$
 1st $2.3 \times 2.3 \times 13 \times 6 = 425$
 2nd $2 \times 2 \times 13 \times 6 = 312$
 3rd $1.5 \times 1.5 \times 13 \times 6 = 176$
 Base $2.6 \times 2.6 \times 11 \times 6 = 470$
 $2.3 \times 2.3 \times 13 \times 6 = 425$
 2nd $2 \times 2 \times 13 \times 6 = 312$
 3rd $1.5 \times 1.5 \times 13 \times 6 = 176$
 Base $2 \times 2 \times 11 \times 4 = 176$
 forward -----
 5122

12178.00

Forward from page 203, 5122 cu. ft. \$ 12179.00

1st	1.6×1.6×13×4	=	145
2nd	1.5×1.5×13×4	=	117
3rd	1.2×1.2×13×4	=	71

5455 cu. ft.
292 c.y. at 16.00 3232.00

Concrete, interior columns (1:2:4)

Base	2.75×2.75×11×36	=	2995 cu. ft.
1st	2.4 ×2.4 ×13×36	=	2744
2nd	2×2×13×36	=	1872
3rd	1.25×1.25×13×36	=	731

Stairway columns,

Base	1.25×1.25×11×2	=	35
1st	1.16×1.16×13×2	=	36
2nd	1×1×13×2	=	26

Elevator columns,

Base	1.25×1.25×14×2	=	44
1st	1.16×1.16×13×2	=	36
2nd	1.1×1.1×13×2	=	31
3rd	0.9×0.9×13×2	=	22

5572 = 316 c.y. at 16.00 5093.00

Platform posts, (1:2:4)

1.4×1.4×8×6	=	72 = 3 c.y. at 16.00	48.00
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Concrete floors (1:2:4)

3 floors, 147×147×0.2×3 = 32418 cu.ft.

roof

147×147×0.33	=	7203
--------------	---	------

147×147×0.26	=	5402
--------------	---	------

45015

duct openings

780

44235

Platform 10×0, 2×2×21	=	820
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44755 = 280 c.y. at 16.00 26564.00

Concrete beams, Roof.

0.9×1.4×140×20	=	3805
----------------	---	------

0.9×0.9×140×2	=	194
---------------	---	-----

Floors

1×1.4×140×20×3	=	11900
----------------	---	-------

0.7×1.4×140×2×3	=	793
-----------------	---	-----

0.7×1×10×16	=	107
-------------	---	-----

12109 = 304 c.y. at 16.00 26564.00
Forward 63755.00

Concrete Girders (1:2:4)

Forward 63755.00

Roof.

1.2×1.9×140×6	=	1878 cu. ft.
0.9×1.4×140×2	=	363
1.2×1.8×14	=	29

Floors

1.5×2.3×140×6×3	=	8820
1.1×1.8×20×25	=	993
1.3×2×40	=	100
1.1×1.8×20×3	=	119
0.7×0.8×14×3	=	7
0.8×1.4×8×3	=	28
0.7×1×12×3	=	24
0.7×1.1×20×5	=	72

 12432 = 481 c.y. at 16.00

7376.00

Roof parapet (1:2:4)

147×4×4×1	=	2352 cu. ft.	=	87 c.y. at 16.00	1392.00
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Concrete pilaster facing (1:2:4)

2.5×0.8×38×32	=	1040 cu. ft.	=	40 c.y. at 16.00	640.00
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Concrete walls, (1:2:4)

Elevator shaft and house

0.5×13×52×2	=	808
0.5×13×3×2	=	39
0.5×13.5×14×4	=	273

Elevator roof

14.5×13.2×0.25	=	45
----------------	---	----

 1001 = 40 c.y. at 16.00

640.00

Stairway, wall and house

0.7×10×9	=	60
0.7×10×1×3	=	210
0.5×14×10×2	=	140
0.5×10×10×2	=	100
14.5×12×0.25	=	43

 533 = 21 c.y. at 16.00

336.00

Concrete window sills,

0.75×1×140×4×3	=	1260 cu. ft.
----------------	---	--------------

deduct

 45

 1215 = 48 c.y. at 16.00

 768.00

Forward

 74875.00

Forward from page 207

74875.00

Concrete stairways, (1:2:4)

Stair nosings

$$4.6 \times 5 \times 0.7 = 16$$

$$4.6 \times 7 \times 0.7 = 21$$

$$4.6 \times 8 \times 0.7 \times 4 = 96$$

$$4.6 \times 4 \times 0.7 = 12$$

Landings

$$4 \times 10 \times 0.5 \times 4 = 80$$

$$\underline{\underline{224}} = 2:1 \text{ c.y. at } 16.00$$

3584.00

Concrete, plain

$$147 \times 151 \times 0.5 = 11088 \text{ cu ft.} = 411 \text{ c.y. at } 15.40 \quad 6330.00$$

Granolithic floor finish

$$146 \times 146 \times 3 = 63948 \text{ sq ft.}$$

$$\text{deduct } 13 \times 13 \times 6 = 1014$$

$$\underline{\underline{62934}} \text{ sq ft. at } 0.08 \text{ per sq. ft.} \quad 5035.00$$

Cinder fill on roof.

$$(146 \times 146 \times 2) - (13 \times 13 \times 2 \times 2) = 41956, 1584 \text{ c.y. at } 1.00 \quad 1584.00$$

Carborundum rubbing, exterior
columns and parapet only.

$$4 \times 147 \times 4 = 2352$$

$$2.5 \times 39 \times 32 = 3120$$

$$0.75 \times 140 \times 4 \times 3 = 1260$$

$$\underline{\underline{6732}} \text{ sq. ft. at } 0.10 \text{ per ft.} \quad 673.00$$

Forms, footings.

$$20 \times 10.5 \times 4 \times 2.25 = 2050 \text{ sq. ft.}$$

$$4 \times 7.6 \times 4 \times 2.4 = 368$$

$$4 \times 10.75 \times 4 \times 3.1 = 540$$

$$6 \times 8.25 \times 4 \times 1 = 80$$

$$36 \times 13.5 \times 4 \times 1.6 = 3916$$

$$4 \times 5.5 \times 4 \times 1 = 88$$

$$\underline{\underline{11848}} \text{ sq ft at } 0.18 \text{ per ft} \quad 2133.00$$

Forms, foundation walls.

$$147 \times 10 \times 2 = 2940 \text{ sq. ft at } 0.20 \text{ per ft.} \quad 588.00$$

$$\text{west wall } 147 \times 3 \times 2 = 882 \text{ sq ft. at } 0.22 \text{ per ft.} \quad 194.00$$

Retaining wall

$$155 \times 3 \times 2 = 930 \text{ sq. ft at } 0.50 \text{ per ft.} \quad 465.00$$

Forward

82460.00

Forward from page 208

92450.00

Exterior Column forms.

12×11×2.4×4	=	1276
12×13×2.2×4	=	1332
12×13×1.4×4	=	1144
12×13×1.1×4	=	884
6×11×2.5×4	=	660
12×13×2.3×4	=	1456
12×13×2×4	=	1248
12×13×1.5×4	=	936
6×11×2.7×4	=	704
4×11×2×4	=	352
4×13×1.7×4	=	350
4×13×1.5×4	=	312
4×13×1.2×4	=	248

 10317 sq. ft. at 0.23 per sq ft. 2388.00
Interior Column forms.

36×11×2.7×4	=	4356
30×13×2.4×4	=	4024
36×13×2×4	=	3744
36×13×1.5×4	=	2840
2×11×1.25×4	=	110
2×13×1.2×4	=	121
2×13×1×4	=	104
2×14×1.25×4	=	140
2×13×1.2×4	=	121
2×13×1.1×4	=	112
2×13×0.9×4	=	94

 13708 sq ft at 0.29 per sq ft 4100.00
Platform posts.

6×6×1.4×4	=	104 sq ft. at 0.46 per sq ft.	53.00
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Floor and roof slab, forms

140×140	=	13600
10×20×3	=	1000

 deduct openings 13600

 19002 sq. ft. at 0.15 per sq. ft. 3426.00

Forward

 102867.00

Forward from Page 208

102207.00

Beam and Girder Brims.

20x140x8.7	=	10288	sq. ft.
2x140x2.8	=	700	
60x140x8.5	=	32200	
6x140x8.5	=	1040	
18x10x2.7	=	427	
8x140x8	=	4200	
2x140x8.8	=	1040	
1x14x4.7	=	88	
18x140x8.2	=	18840	
28x20x4.8	=	2376	
2x20x5.8	=	210	
8x20x4.8	=	288	
8x14x1.2	=	49	
8x8x8.7	=	88	
8x12x2.7	=	88	
8x10x2.8	=	18	

 70618 sq. ft. at 0.20 per sq. ft. 14104.00

Parquet

4x147x4	=	2382	
2x147x4	=	1191	
1x147x4	=	595	
2x147x4	=	1191	

 4360

4360 sq. ft. at 0.26 per sq. ft. 1134.00

Elevator and Stairway.

1x8x18x2	=	188	
2x8x18x2	=	376	
4x14x14x2	=	1101	
14.8x18.2x2	=	100	
1x8x10x2	=	181	
2x13x10x2	=	360	
2x10x14x2	=	280	
2x10x10x2	=	200	
14.8x12.8x2	=	111	

 3187 sq. ft. at 0.34 per sq. ft. 1083.00

Aircon Units.

8x4x140x2.8	=	1120	sq. ft. at 0.15 per sq. ft.	168.00
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 Forward:

 120820.00

Carry over from Page 210

120828.00

Stairway steps

4.5x8	=	36 sq. ft.
65x4.5x0.7	=	185
4.5x7	=	31
4.5x8x4	=	144
3.5x8x2	=	16
3.5x27x0.7	=	65
3.5x4	=	14
4x10x3	=	110
4x8	=	24

884 sq. ft. at 0.80 per sq. ft. 208.00

Reinforcing steel.

Roof.

200 pcs. 13/16" rd. x 36'1.5"	=	12735 lbs
320 pcs. 13/16" rd. x 38' 0"	=	17858
144 pcs. 13/16" rd. x 31'2.5"	=	7324
40 pcs. 13/16" rd. x 30' 1"	=	2122
20 pcs. 13/16" rd. x 35'5.5"	=	1280
4 pcs. 13/16" rd. x 30'3.5"	=	218
120 pcs. 1 1/8" rd. x 36'3"	=	14800
60 pcs. 1 1/8" rd. x 38'0"	=	7080
40 pcs. 1 1/8" rd. x 31'7"	=	4278
20 pcs. 1 1/8" rd. x 30'12"	=	2084

70480 x 0.80 per 1000 281.00

38 pcs. 11/16" rd. x 38'5"	=	3018
38 pcs. 11/16" rd. x 36'10"	=	1130
8 pcs. 11/16" rd. x 31'0"	=	318
8 pcs. 11/16" rd. x 30'11"	=	307

5463 x 0.80 per 1000 120.00

2400 pcs. 3/8" rd. x 36'10"	=	21600
1248 pcs. 3/8" rd. x 38'10"	=	11808

33408 x 0.80 per 1000 1872.00

Stirrups for roof

4488 pcs. 1/8" rd. x 36'10"	=	2144
388 pcs. 1/8" rd. x 38'10"	=	144
388 pcs. 3/8" rd. x 36'10"	=	334
2240 pcs. 7/16" rd. x 36'10"	=	2128

14821 x 0.80 per 1000 118.00

Carry over

120898.00

Forward from Page 111

125075.00

Misc steel.

8186 pcs. 3/8" rd. x 17'0" = 10882 lbs.

1819 pcs. 3/8" rd. x 24'0" = 18671

29553 lbs at 2.50 cwt 1805.00

Beam and girder.

888 pcs. 3/4" rd. x 36'2" = 32878

1784 pcs. 3/4" rd. x 26'2" = 29338

228 pcs. 3/4" rd. x 31'3" = 10902

284 pcs. 3/4" rd. x 26'6" = 28388

28 pcs. 3/4" rd. x 13'10" = 834

28 pcs. 3/4" rd. x 18'11" = 1171

4 pcs. 3/4" rd. x 14'11" = 90

3 pcs. 3/4" rd. x 14'2" = 170

80 pcs. 1 1/8" rd. x 30'3" = 3130

72 pcs. 1 1/8" rd. x 28'3" = 3127

12 pcs. 1 1/8" rd. x 31'4" = 388

288 pcs. 1 1/8" rd. x 32'0" = 28010

482 pcs. 1 1/8" rd. x 28'4" = 28688

132 pcs. 1 1/8" rd. x 31'8" = 3009

60 pcs. 1 1/8" rd. x 32'0" = 1417

302 pcs. 1 1/8" rd. x 31'1" = 17330

46 pcs. 1 1/8" rd. x 30'7" = 3910

8 pcs. 1 1/8" rd. x 30'8" = 998

18 pcs. 1 1/8" rd. x 28'2" = 1381

6 pcs. 1 1/8" rd. x 30'0" = 728

8 pcs. 1 1/8" rd. x 28'3" = 131

4 pcs. 1 1/8" rd. x 31'1" = 421

4 pcs. 1 1/8" rd. x 28'7" = 314

31750 lbs at 2.50 cwt 7705.00

Stirrups

23864 pcs. 7/16" rd. x 3'4" = 64482

576 pcs. 7/16" rd. x 4'8" = 1878

240 pcs. 7/16" rd. x 5'11" = 726

2888 pcs. 3/8" rd. x 11'10" = 13130

920 pcs. 3/8" rd. x 5'11" = 1048

81710 lbs at 2.50 cwt 8171.00

104 pcs. 1/2" rd. x 6'3" = 387 lbs at 2.50 cwt 17.00

308 pcs. 1/4" rd. x 4'5" = 111

384 pcs. 1/2" rd. x 3'4" = 311

327 lbs at 2.50 cwt 18.00

Forward

127865.00

Forward from page 212

137385.00

Column Steel.

Interior Columns

288 pcs. 1/2" rd. x 15'0"	= 2882 lbs at 3.73 cwt	108.00
288 pcs. 13/16" rd. x 15'0"	= 7617	
288 pcs. 1" rd. x 15'0"	= 11035	
288 pcs. 1 1/8" rd. x 15'0"	= 12651	
	<u>31803</u> lbs at 3.63 cwt	1154.00
36 pcs. 1/4" rd. 12.5" dia spiral 2" c-c, 13'0"	= 2237	
36 pcs. 1/4" rd. 21" dia. spiral 2.5" c-c, 13'0"	= 3281	
36 pcs. 1/4" rd. 26" dia. spiral 2.5" c-c, 13'0"	= 3857	
36 pcs. 1/4" rd. 30.5" dia. spiral 2.5" c-c, 11'0"	= 3663	
	13033 lbs at 4.13 cwt	539.00

Exterior columns.

128 pcs. 11/16" rd. x 15'0"	= 2425	
128 pcs. 5/8" rd. x 15'10"	= <u>2003</u>	
	4428 lbs at 3.63 cwt	163.00
96 pcs. 13/16" rd. x 15'10"	= 2532	
32 pcs. 13/16" rd. x 13'0"	= 734	
96 pcs. 15/16" rd. x 13'0"	= 3350	
192 pcs. 1 1/8" rd. x 13'0"	= 8430	
96 pcs. 7/8" rd. x 15'0"	= 2944	
96 pcs. 1" rd. x 15'0"	= <u>3332</u>	
	21339 lbs at 3.63 cwt	775.00

Bands.

144 pcs. 1/4" rd. x 8'0"	= 120	
144 pcs. 1/4" rd. x 8'6"	= 160	
144 pcs. 1/4" rd. x 9'0"	= 192	
120 pcs. 1/4" rd. x 9'6"	= 180	
144 pcs. 1/4" rd. x 10'0"	= 148	
144 pcs. 1/4" rd. x 10'6"	= 177	
144 pcs. 1/4" rd. x 11'0"	= 203	
120 pcs. 1/4" rd. x 11'6"	= 190	
48 pcs. 1/4" rd. x 4'0"	= 32	
48 pcs. 1/4" rd. x 6'2"	= 41	
48 pcs. 1/4" rd. x 6'0"	= 46	
40 pcs. 1/4" rd. x 8'10"	= <u>28</u>	
	1740 lbs at 4.13 cwt	64.00

Forward 140197.00

Forward from page 213

140187.00

Elevator and stair columns

24 pcs. 1/2" rd. x 15'0"	=	240 lbs at 3.75 cwt	9.00
16 pcs. 1" sq. x 15'0"	=	616	
16 pcs. 1 1/4" sq. x 13'0"	=	<u>1105</u>	
		1921 lbs at 3.63 cwt	70.00
4 - 1/4" spirals 9.5" dia. 2" c-c, 13'0"	=	249	
4 - 1/4" spirals 11" dia. 2" c-c, 13'0"	=	249	
4 - 1/4" spirals 12" dia. 2" c-c, 11'0"	=	211	
2 - 1/4" spirals 7.5" dia, 2" c-c, 13'0"	=	<u>125</u>	
		834 lbs at 4.13 cwt	34.00

Steel around elevator and stairs.

Elevator house beam and girders.

48 pcs. 1/2" rd. x 13'6"	=	403 lbs.	
28 pcs. 1/2" rd. x 7'3"	=	<u>136</u>	
		534 lbs at 3.75 cwt	21.00
34 pcs. 3/8" rd. x 8'0"	=	64	
16 pcs. 3/8" rd. x 4'9"	=	29	
21 pcs. 3/8" rd. x 3'3"	=	<u>42</u>	
		135 lbs at 3.69 cwt	5.00
4 pcs. 1 1/8" rd. x 13'6"	=	210	
4 pcs. 1 1/8" rd. x 14'10"	=	201	
8 pcs. 3/4" rd. x 15'0"	=	180	
8 pcs. 3/4" rd. x 13'3"	=	126	
4 pcs. 3/4" rd. x 19'0"	=	114	
2 pcs. 3/4" rd. x 17'4"	=	52	
4 pcs. 7/8" rd. x 23'2"	=	214	
4 pcs. 7/8" rd. x 22'10"	=	<u>262</u>	
		1482 lbs at 3.63 cwt	52.00

Floor at elevator shaft.

6 pcs. 1/2" sq. x 18'2"	=	83	
6 pcs. 1/2" sq. x 18'0"	=	97	
198 pcs. 1/2" rd. x 2'10"	=	374	
6 pcs. 1/2" rd. x 17'2"	=	68	
78 pcs. 1/2" rd. x 2'3"	=	<u>117</u>	
		730 lbs at 3.75 cwt	28.00
3 pcs. 15/16" rd. x 31'0"	=	<u>240</u>	
3 pcs. 15/16" rd. x 36'7"	=	<u>253</u>	
12 pcs. 3/4" rd. x 15'0"	=	<u>271</u>	
36 pcs. 3/4" rd. x 14'6"	=	<u>724</u>	
		1588 lbs at 3.63 cwt	55.00

Forward

140481.00

215.

Forward from page 214

140461.00

42 pcs. 7/16" rd. stirrups x 4'9" = 102 lbs 3.88 cwt 4.00
 27 pcs. 3/8" rd. stirrups x 4'9" = 59 lbs 3.88 cwt 2.00

Beams and girders at Stairway.

3 pcs. 1/2" rd. x 12'2" = 25
 3 pcs. 1/2" rd. x 11'2" = 60
 24 pcs. 1/2" rd. x 2'10" = 46
 120 pcs. 1/2" rd. x 4'7" = 366
 45 pcs. 1/2" rd. x 3'7" = 108
 6 pcs. 1/2" sq. x 17'2" = 83
 6 pcs. 1/2" sq. x 19'10" = 86

 782 lbs at 3.73 cwt 30.00
 6 pcs. 3/4" rd. x 25'2" = 217
 6 pcs. 3/4" rd. x 26'6" = 232
 466 lbs at 3.63 cwt 17.00

Footing steel.

876 pcs. 1" rd. x 13'6" = 20762
 280 pcs. 1" rd. x 10'6" = 7860
 86 pcs. 1" rd. x 10'9" = 1608
 48 pcs. 1" rd. x 8'4" = 437
 40 pcs. 1" rd. x 7'7" = 310
 48 pcs. 1" rd. x 8'4" = 336

 32140 lbs at 3.63 cwt 1167.00

Bowels

288 pcs. 1 1/4" rd x 2'0" = 2104
 192 pcs. 1 1/8" rd x 2'0" = 1238
 16 pcs. 1 1/4" rd. x 2'0" = 150
 32 pcs. 13/16" rd x 2'0" = 110

 3598 lbs at 3.63 cwt 145.00

Steel in stair treads.

9 pcs. 1/2" rd x 19'6" = 117
 9 pcs. 1/2" rd x 22'6" = 135
 36 pcs. 1/2" rd x 14'0" = 336
 64 pcs. 1/2" rd x 23'6" = 448

 1436 lbs at 3.73 cwt 54.00

Forward

141580.00

Forward from page 215

141880.00

General excavation, (scraper and hand		
work, leveling of site, no earth removed,		
22500 sq. ft. at 0.05		1125.00
Excavation of basement, 8000 cu. yds at 0.50		4800.00
Footing excavation, (hand work, loan)		
Exterior footings,	372 cu. yds.	
Interior footings,	1200	
Platform footings,	3	
Elevator shaft,	25	
Foundation wall trench,	25	
	<u>1625</u> cu. yds. at 1.50	2437.00
Backfill, rehandle and grade,	1625 cu. yds. at 0.50	812.00
Masonry, brick curtain walls,	3440 cu. ft. at 1.00	3440.00
Steel sash, erected and pointed, 20 percent pivot vent.		
South, 7x5'4"x18'2" =	681	
21x7'4"x18'0" =	2925	
East, 18x7'4"x18'0" =	2507	
North, 21x7'4"x18'0" =	2925	
West, 18x7'4"x18'0" =	2646	
4x7'4"x10'0" =	283	
	<u>11077</u> sq. ft. at 0.45	5380.00
Glass and glazing, factory rubber glass, (12x12)		
set including putty, 20 per area of sash,		
10782 sq. ft. at 0.30		3235.00
Doors, frames, hardware, etc		
Dbl. swing, wood exterior		
doors, wood frame, transom,		
C.I. sill 2 at 8'0" by 11'0, No 1 at 175.00 ea		350.00
Single swing, tin clad doors		
in stair halls, angle iron		
frames, C.I. sills, 2 - 8'0" by 7'0, No 2 at 10.00 ea.		450.00
Rolling doors,	3 at 200.00 ea	1200.00
Light iron and steel		
2" pipe hand rail (2 lines at stair) 112' at 2.50 ft.		280.00
2" pipe hand rail (1 line on wall) 112' at 1.50 ft.		168.00
3 1/2" safety stair treads,	30 at 1.50 ea.	120.00
Slotted inserts in ceiling, 3000 lbs. at 2.50 lb.		750.00
C.I. scuppers,	12 at 5.00 each	60.00
Miscellaneous iron, sundries		<u>300.00</u>
Forward		166817.00

Forward from page 216.

186817.00

Roofing and flashings;;

5 ply tar and gravel roof

147x147 = 216 squares at 18.00 per square

3888.00

Tongue metal base and cap flashing,

4x145.66 = 583 ft. at 0.40 ft.

233.00

Painting;

Walls and ceilings (2coats mill white)

9600 sq. ft. at 0.40 per sq. ft.

3840.00

Sash and doors, 2 coats

1800 sq. ft. at 0.30 per sq. ft.

540.00

Miscellaneous iron painted

50.00

Placing 282 tons of reinforcing steel at 14.00 ton

4038.00

Engineering, plans, details, etc.

1800.00

Clean up job at completion, clean glass,

300.00

Liability insurance,

1000.00

Watchman, 24 weeks at 25.00 per week

600.00

Superintendence, job overhead, office stationery,

miscellaneous office, 24 weeks at 200.00 per

4800.00

Sundries,

800.00

Net

187918.00

Contractors profit, 10 per cent

22192.00

TOTAL ESTIMATE

200728.00

Following items not included in estimate,

Separate Contracts.

- (1) Heating and boilers
- (2) Plumbing, conductors, drains
- (3) Electric wiring
- (4) Sprinkler system
- (5) Elevators.
- (6) Land.

APPENDIX No. 1.

Reports 1 to 18 inclusive.

To be used for the design and review
of rectangular gears, tee gears, and
sloes.

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for Appendix No. 1.

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TABLE 1.

Equivalent Fluid Pressure.

$\frac{t}{h}$	w
0.30	12.6
0.35	16.6
0.40	21.1
0.45	26.1
0.50	31.6
0.55	37.6
0.60	44.1
0.65	51.2

 t = width of base in feet. h = total height in feet. w = pounds per foot.

TABLE 2.

Data for design of rectangular beams.

Formulas needed

$$M = Kbd^2, \text{ or } bd^2 = \frac{M}{K}$$

$$a_3 = pbd$$

Formulas used for tables.

$$P = \frac{\frac{f_s}{f_c} \left(\frac{f_s}{f_c} + 1 \right)}{\frac{f_s}{f_c} \left(\frac{f_s}{f_c} + 1 \right) + 1}$$

$$K = \sqrt{2pn + (pn)^2} - pn \quad j = 1 - \frac{K}{P}$$

$$K = pf_s j \text{ or } \frac{1}{2} f_c k j$$

Working Stresses		Ratio of Moduli $n=12$				Ratio of Moduli $N=15$			
f_s	f_c	k	j	P	K	k	j	P	K
12,000	580	0.332	0.889	0.0069	73.6	0.389	0.872	0.0080	83.7
	550	0.354	0.882	0.0081	85.7	0.407	0.864	0.0093	96.9
	600	0.375	0.875	0.0094	98.4	0.428	0.857	0.0107	110.0
	650	0.394	0.869	0.0107	111.3	0.448	0.851	0.0121	123.6
	700	0.412	0.863	0.0120	124.4	0.467	0.844	0.0136	138.0
	750	0.429	0.857	0.0134	137.8	0.484	0.839	0.0151	152.0
	800	0.444	0.852	0.0148	151.3	0.501	0.833	0.0167	166.9
14,000	500	0.300	0.900	0.0054	67.5	0.348	0.884	0.0062	76.7
	550	0.320	0.893	0.0063	78.6	0.372	0.876	0.0073	89.5
	600	0.340	0.888	0.0073	90.6	0.391	0.870	0.0084	102.0
	650	0.358	0.881	0.0083	102.6	0.410	0.863	0.0095	114.8
	700	0.375	0.875	0.0094	114.8	0.428	0.857	0.0107	128.3
	750	0.391	0.870	0.0105	127.6	0.446	0.851	0.0120	142.3
	800	0.407	0.864	0.0116	140.4	0.462	0.846	0.0132	156.3
15,000	500	0.286	0.905	0.0048	64.7	0.334	0.889	0.0056	74.1
	550	0.306	0.898	0.0056	75.4	0.355	0.882	0.0065	86.1
	600	0.325	0.892	0.0065	86.7	0.375	0.875	0.0075	98.3
	650	0.343	0.886	0.0074	98.4	0.393	0.869	0.0085	111.3
	700	0.360	0.880	0.0084	110.3	0.411	0.863	0.0096	124.2
	750	0.376	0.875	0.0094	123.1	0.429	0.857	0.0107	137.9
	800	0.391	0.870	0.0105	135.7	0.445	0.852	0.0118	151.2
16,000	500	0.273	0.909	0.0043	62.0	0.319	0.894	0.0050	71.3
	550	0.292	0.903	0.0050	72.2	0.339	0.887	0.0058	82.3
	600	0.310	0.897	0.0058	83.2	0.358	0.881	0.0067	94.4
	650	0.328	0.891	0.0067	95.0	0.378	0.874	0.0077	107.4
	700	0.344	0.885	0.0075	106.2	0.397	0.868	0.0087	120.6
	750	0.361	0.880	0.0085	119.1	0.414	0.862	0.0097	133.8
	800	0.375	0.875	0.0094	131.3	0.429	0.857	0.0107	146.7
20,000	500	0.230	0.923	0.0029	53.1	0.272	0.909	0.0034	61.8
	550	0.248	0.917	0.0034	62.4	0.292	0.903	0.0040	72.2
	600	0.264	0.912	0.0040	72.2	0.311	0.897	0.0047	83.7
	650	0.280	0.907	0.0046	82.4	0.328	0.891	0.0053	94.4
	700	0.295	0.902	0.0052	93.3	0.344	0.885	0.0060	106.2
	750	0.309	0.897	0.0058	103.9	0.359	0.880	0.0067	117.9
	800	0.324	0.892	0.0065	115.6	0.374	0.875	0.0075	130.9

TABLE 3.

Data for reviewing rectangular beams.

n = 18

Formulas needed:

$$p = \frac{a_s}{bd}$$

$$\frac{M}{bd^2} = K$$

Formulas used in preparing table:

$$k = \sqrt{2pn + (pn)^2} - pn$$

$$j = 1 - \frac{1}{2}k$$

$$f_c = \frac{2f_s p}{k}$$

$$K = p f_s j \text{ or } \frac{1}{2} f_c k j$$

p	k	j	f _s = 650 lb. per square inch		f _s = 16000 lb. per square inch	
			f _s	K	f _c	K
.0020	0.217	0.924	25300	65.5	285	22.7
.0022	0.226	0.925	33400	68.0	310	32.4
.0024	0.235	0.922	31800	70.4	330	35.8
.0026	0.243	0.919	30400	72.6	340	38.0
.0028	0.251	0.916	29100	74.6	360	41.4
.0030	0.258	0.914	28000	76.8	370	43.6
.0032	0.265	0.912	26900	78.5	390	47.1
.0034	0.273	0.909	26100	80.7	400	49.6
.0036	0.279	0.907	25200	82.1	410	51.9
.0038	0.285	0.905	24400	83.9	430	55.1
.0040	0.292	0.903	23700	85.6	440	58.2
.0042	0.298	0.901	23100	87.4	450	60.4
.0044	0.303	0.899	22400	88.6	465	63.3
.0046	0.309	0.897	21800	89.8	480	66.5
.0048	0.314	0.895	21100	90.6	490	68.8
.0050	0.320	0.893	20800	92.8	500	71.4
.0052	0.326	0.892	20300	94.2	510	73.9
.0054	0.330	0.890	19900	95.6	520	76.4
.0056	0.334	0.889	19400	96.6	540	80.1
.0058	0.339	0.887	19000	97.7	550	82.7
.0060	0.344	0.885	18600	98.8	560	85.2
.0062	0.348	0.884	18300	100.3	570	87.7
.0064	0.353	0.882	18000	101.6	580	90.6
.0066	0.357	0.881	17600	102.3	590	92.8
.0068	0.361	0.880	17300	103.5	600	95.3
.0070	0.365	0.878	16900	103.2	610	97.7
.0072	0.369	0.877	16700	105.4	625	101.1
.0074	0.373	0.876	16400	106.3	635	103.7
.0076	0.377	0.875	16100	107.1	645	106.4
.0078	0.381	0.873	15900	108.3	655	108.1
.0080	0.384	0.872	15600	108.8	670	112.2
.0082	0.388	0.871	15400	110.0	680	114.9
.0084	0.392	0.869	15200	110.9	690	117.5
.0086	0.395	0.868	15000	112.0	700	120.0
.0088	0.398	0.867	14700	112.2	710	122.5
.0090	0.402	0.866	14500	113.0	720	125.3
.0092	0.403	0.865	14300	113.8	725	127.0
.0094	0.408	0.864	14100	114.5	740	130.4
.0096	0.411	0.863	13900	115.2	750	133.0
.0098	0.415	0.862	13800	115.6	760	135.9
.010	0.418	0.861	13600	117.1	770	138.6
.012	0.446	0.851	12100	123.6	860	163.2
.014	0.471	0.843	11000	129.8	950	188.6
.016	0.493	0.836	10000	133.8	1040	214.3
.018	0.513	0.829	9300	138.8	1120	238.7
.020	0.531	0.823	8600	141.6	1210	264.4

TABLE 4.

Use for designing slabs.

Based on $M = \frac{wL^2}{10}$

For supported ends, $(M = \frac{wL^2}{8})$, deduct 20 per cent from load.For fully continuous, $(M = \frac{wL^2}{12})$, add 20 per cent to load.

$f_c = 650 \text{ sg. in./lb.} \quad f_s = 16000 \text{ lb. sg. in.} \quad n = 15 \quad p = 0.0077$

$k = 0.378 \quad b = 12 \text{ in.}$

$M = p f_s j b d^2 = 1290 d^2 \text{ (safe moment of resistance)}$

$a_s = p b d = 0.0324 d \text{ (steel area)}$

$\frac{wL^2}{10} (12) = p f_s j b d^2, \text{ or } w = 1074 \frac{d^2}{L^2} \text{ (safe load)}$

Depth to Steel in.	Depth below Steel in.	Total depth slab, in.	Total safe load (w) per square foot including weight of slab. For safe live load deduct weight of slab.															Weight of Slab per square foot.	Steel area in section of slab 1 foot wide sq. in.	Safe moment of resistance in lb.
			Span in feet (l)																	
			4	5	6	7	8	9	10	11	12	13	14	15	16	17	18			
1 $\frac{3}{4}$	$\frac{3}{4}$	2 $\frac{1}{2}$	208	132	91	67	51	40						31	0.162	3950				
2	$\frac{3}{4}$	2 $\frac{3}{4}$	268	172	120	88	67	53	43	35				34	0.185	5160				
2 $\frac{1}{4}$	$\frac{3}{4}$	3	339	217	151	111	85	67	54	45				38	0.208	6530				
2 $\frac{1}{2}$	$\frac{3}{4}$	3 $\frac{1}{4}$	419	268	187	137	105	83	67	55	47			41	0.231	8030				
2 $\frac{3}{4}$	$\frac{3}{4}$	3 $\frac{1}{2}$	507	325	226	165	127	100	81	67	57	48		44	0.254	9750				
3	$\frac{3}{4}$	3 $\frac{3}{4}$	605	387	269	197	151	119	97	80	67	57	49	47	0.277	11600				
3 $\frac{1}{4}$	$\frac{3}{4}$	4	709	454	315	232	177	140	113	94	79	67	58	50	0.300	13600				
3 $\frac{1}{2}$	1	4 $\frac{1}{2}$	822	527	366	269	206	162	132	109	91	78	67	58	56	0.323	15800			
4	1	5	1074	688	478	351	269	212	172	142	119	102	88	76	62	0.370	20600			
4 $\frac{1}{2}$	1	5 $\frac{1}{2}$	1360	871	605	444	340	269	218	180	151	129	111	97	69	0.416	26100			
4 $\frac{3}{4}$	1 $\frac{1}{4}$	6	1520	971	675	496	380	300	243	201	169	144	124	108	75	0.438	29100			
5 $\frac{1}{2}$	1 $\frac{1}{4}$	6 $\frac{3}{4}$	1850	1190	825	606	463	366	297	245	206	175	151	132	81	0.485	35600			
5 $\frac{3}{4}$	1 $\frac{1}{4}$	7	2220	1420	990	726	556	438	356	294	247	210	182	158	87	0.531	42700			
6 $\frac{1}{4}$	1 $\frac{1}{4}$	7 $\frac{1}{2}$	2620	1680	1170	858	657	519	420	347	292	248	214	187	94	0.578	50500			
6 $\frac{3}{4}$	1 $\frac{1}{4}$	8	3070	1960	1360	1000	766	605	490	405	340	290	250	218	100	0.624	58900			
7 $\frac{1}{4}$	1 $\frac{1}{4}$	8 $\frac{1}{2}$	3540	2260	1570	1150	884	698	566	467	392	335	289	252	106	0.670	67900			
7 $\frac{3}{4}$	1 $\frac{1}{4}$	9	4040	2580	1800	1320	1010	798	646	534	449	382	330	288	113	0.716	77600			
8 $\frac{1}{4}$	1 $\frac{1}{4}$	9 $\frac{1}{2}$	4580	2930	2030	1490	1140	904	733	605	509	434	374	326	119	0.762	87900			
8 $\frac{3}{4}$	1 $\frac{1}{4}$	10	5150	3300	2290	1680	1290	1020	824	680	572	487	420	366	125	0.808	98800			

TABLE 5.

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Use for reviewing slab designs.

Based on $M = \frac{wl^2}{10}$
 $f_c = \text{or } < 650 \text{ lb./sq. in.}$ For supported ends ($M = \frac{wl^2}{8}$) deduct 20 percent from load
For fully continuous ($M = \frac{wl^2}{12}$) add 20 percent to load $M = \text{lesser of } \begin{cases} M_s = p f_s b d^2 \\ M_c = \frac{1}{2} f_c k_j b d^2 \end{cases} \text{ safe moment}$ $a_s = p b d = 12 \text{ pd. (steel area)}$ $\frac{wl^2}{10} (12) = p f_s b d^2 \quad \text{Using table 3.}$
 $\frac{wl^2}{10} (12 - \frac{1}{2} f_c k_j b d^2)$ For $p = 0.002$, $w = 297 \frac{\text{lb.}}{\text{sq. ft.}}$ (safe load)" $p = 0.004$, $w = 582 \frac{\text{lb.}}{\text{sq. ft.}}$ " "" $p = 0.006$, $w = 852 \frac{\text{lb.}}{\text{sq. ft.}}$ " "" $p = 0.008$, $w = 1088 \frac{\text{lb.}}{\text{sq. ft.}}$ " "" $p = 0.010$, $w = 1171 \frac{\text{lb.}}{\text{sq. ft.}}$ " "

P	Depth to Steel in.	Depth below Steel in.	Total Depth of Slab in.	Total safe load (w) per square foot including weight of slab. For safe live load deduct weight of slab.															Weight of slab per square foot	Steel area in section of slab one foot wide	Safe mom- ent of resistance.
				Span in feet (l)																	
				4	5	6	7	8	9	10	11	12	13	14	15	16.	Sq. in.	in. lb.			
0.002	2 1/4	3/4	3	34	60	41											38	0.054	1800		
	3 1/4	3/4	4	196	125	87	64	49									50	0.078	3760		
	4	1	5	297	190	132	97	74	59	47							62	0.096	5700		
	4 1/4	1 1/4	6	420	268	186	137	103	83	67	55	47					75	0.114	8060		
	5 1/4	1 1/4	7	615	393	273	200	153	121	98	81	68	58	50	44	87	0.138	11800			
	6 1/4	1 1/4	8	847	542	376	276	212	167	135	112	94	80	69	60	100	0.162	16300			
	7 1/4	1 1/4	9	1120	715	496	364	279	220	179	147	124	106	91	79	113	0.186	21400			
0.004	8 1/4	1 1/4	10	1420	910	632	464	356	281	228	188	158	135	116	101	125	0.210	27300			
	2 1/4	3/4	3	184	118	82	60	46									38	0.108	3540		
	3 1/4	3/4	4	384	246	170	125	96	76	61	51						50	0.156	7380		
	4	1	5	582	372	258	190	145	115	93	77	65	55	47			62	0.192	11200		
	4 1/4	1 1/4	6	823	526	366	268	206	162	131	109	91	78	67	58	75	0.228	15800			
	5 1/4	1 1/4	7	1200	770	535	393	301	238	193	159	134	114	98	86	87	0.276	23100			
	6 1/4	1 1/4	8	1660	1060	738	542	415	328	267	219	184	157	135	118	100	0.324	31800			
0.006	7 1/4	1 1/4	9	2190	1400	972	714	547	432	350	289	243	207	178	155	113	0.372	42000			
	8 1/4	1 1/4	10	2790	1780	1240	910	697	551	446	368	310	264	228	198	125	0.420	53500			
	2 1/4	3/4	3	270	173	120	88	67	53	43							38	0.162	5160		
	3 1/4	3/4	4	562	360	250	183	140	111	90	74	62	53				50	0.234	10800		
	4	1	5	852	545	379	278	213	168	136	113	95	81	70			62	0.288	16300		
	4 1/4	1 1/4	6	1200	771	535	393	301	238	193	159	134	114	98	86	75	0.342	23100			
	5 1/4	1 1/4	7	1760	1130	784	575	441	348	282	233	196	167	144	125	87	0.414	33800			
0.008	6 1/4	1 1/4	8	2430	1550	1080	793	607	480	388	321	270	230	198	173	100	0.486	46600			
	7 1/4	1 1/4	9	3200	2050	1420	1090	800	632	512	423	356	303	261	228	113	0.558	61500			
	8 1/4	1 1/4	10	4080	2610	1810	1330	1020	806	653	540	463	386	333	290	125	0.630	78300			
	2 1/4	3/4	3	344	220	153	112	86	68	55	45						38	0.216	6610		
	3 1/4	3/4	4	718	459	319	234	179	142	115	95	80	68	59			50	0.312	13800		
	4	1	5	1090	696	483	355	272	215	174	144	121	103	89	77	62	0.384	20900			
	4 1/4	1 1/4	6	1540	984	682	502	383	303	246	203	171	145	125	109	75	0.456	29500			
0.010	5 1/4	1 1/4	7	2250	1440	1000	735	561	444	360	297	250	213	184	160	87	0.552	43200			
	6 1/4	1 1/4	8	3100	1990	1380	1010	774	612	496	409	344	293	253	220	100	0.648	59500			
	7 1/4	1 1/4	9	4080	2620	1820	1330	1020	807	654	540	454	387	334	291	113	0.743	78500			
	8 1/4	1 1/4	10	5210	3340	2320	1700	1300	1030	834	688	579	493	426	370	125	0.840	100000			
	2 1/4	3/4	3	370	237	165	121	93	73	59	49						38	0.270	7100		
	3 1/4	3/4	4	778	495	344	252	193	153	124	102	86	73	63	55	50	0.390	14800			
	4	1	5	1170	750	520	383	293	231	187	155	130	111	96	83	62	0.480	22500			
0.010	4 1/4	1 1/4	6	1650	1060	734	539	414	326	264	218	184	157	135	118	75	0.570	31800			
	5 1/4	1 1/4	7	2420	1550	1070	790	606	478	388	320	269	229	198	172	87	0.690	46500			
	6 1/4	1 1/4	8	3340	2140	1480	1090	835	659	534	441	371	316	272	237	100	0.810	64100			
	7 1/4	1 1/4	9	4400	2820	1950	1430	1100	869	704	581	489	417	360	313	113	0.930	84500			
	8 1/4	1 1/4	10	5610	3590	2490	1830	1400	1110	897	741	623	531	459	399	125	1.050	107800			

TABLE 6.

Use for continuous Rectangular Beams.

Safe loading and reinforcement for beam 1" in width.

Based on $M = \frac{wl^2}{12}$ $n = 15$ $f_s = 16000$ lb. per sq. in. $f_c = 650$ lb. per sq. in.For $\left\{ \begin{array}{l} M = \frac{wl^2}{10} \\ M = \frac{wl^2}{8} \end{array} \right\}$ deduct $\left\{ \begin{array}{l} 16\frac{2}{3}\% \\ 33\frac{1}{3}\% \end{array} \right\}$ from safe loads using same steel area.

Depth to Steel (d) in.	Total safe load (w) per linear foot for beam one inch wide including weight of beam. (see foot notes)																								Steel area in beam one inch wide (as) sq. in.	Safe m- ment of resistance M. in. lb.
	Span in feet (L)																									
	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24							
6.0	107	79	60	48	39	32	27	23	20																0.046	3870
6.5	126	93	71	56	45	38	31	27	23	20															0.050	4540
7.0	146	107	82	63	52	43	36	31	27	23	21														0.054	5260
7.5	168	123	94	75	60	50	42	36	31	27	24	21													0.058	6040
8.0	191	140	107	85	69	57	48	41	35	31	27	24	21												0.062	6870
8.5	216	158	121	96	78	64	54	46	40	34	30	27	24	21											0.065	7760
9.0	242	178	136	107	87	72	60	51	44	39	34	30	27	24	22										0.069	8700
9.5	269	198	151	120	97	80	67	57	49	43	38	33	30	27	24	22									0.073	9690
10.0	298	219	168	133	107	89	75	64	55	48	42	37	33	30	27	24	22								0.077	10740
10.5	329	242	185	146	118	98	82	70	60	53	46	41	36	33	30	27	24	22							0.081	11890
11.0	361	265	203	160	130	107	90	77	66	58	51	45	40	36	32	29	27	25	22						0.085	13000
11.5	394	290	222	175	142	117	98	84	72	63	55	49	44	39	35	32	29	27	25						0.089	14200
12.0	430	316	242	191	155	128	107	92	79	69	60	53	48	43	39	35	32	29	27						0.092	15470
12.5	466	342	262	207	168	138	116	99	86	74	65	58	52	46	42	38	35	32	29						0.096	16780
13.0	504	370	284	224	182	150	126	107	93	81	71	63	56	50	45	41	37	34	31						0.100	18150
13.5	544	399	306	242	196	161	136	116	100	87	76	68	60	54	49	44	40	37	34						0.104	19570
14.0	585	430	329	260	210	174	146	125	107	93	82	73	65	58	53	48	43	40	36						0.108	21050
14.5	628	461	353	279	226	187	157	133	115	100	88	78	70	62	56	51	46	43	39						0.112	22580
15.0	671	493	378	298	242	200	168	143	123	107	94	83	74	67	60	55	50	46	42						0.115	24160
15.5	716	526	403	318	258	213	179	152	131	114	100	89	79	71	64	58	53	49	45						0.119	25800
16.0	764	561	430	339	275	227	191	163	140	122	107	95	85	76	69	62	57	52	48						0.123	27490
16.5	812	596	457	361	292	241	203	173	149	130	114	101	90	81	73	66	60	55	51						0.127	29240
17.0	863	634	485	384	310	257	216	183	158	138	121	107	96	86	78	70	64	59	54						0.131	31040
17.5	914	672	514	406	329	272	228	195	168	146	128	114	101	91	82	74	68	62	57						0.135	32890
18.0	967	710	544	430	348	288	242	206	177	155	136	120	107	97	87	79	72	66	60						0.139	34800
20.0	1193	877	671	530	430	355	298	254	219	191	168	148	132	119	107	97	89	81	74						0.154	42960
22.0	1449	1060	812	642	520	430	361	308	265	231	203	180	160	144	140	118	107	99	90						0.169	51980
24.0	1718	1262	968	764	619	512	430	366	316	275	242	214	191	171	155	140	127	117	107						0.185	61860
26.0		1481	1134	897	726	600	504	430	370	323	284	251	224	201	181	164	150	138	126						0.200	72600
28.0		1717	1314	1039	842	696	589	498	429	374	328	291	260	233	210	190	173	160	146						0.216	84200
30.0			1511	1193	967	799	671	572	494	428	378	334	297	267	242	219	199	183	168						0.231	96640

Total depth of beam in inches.	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
Weight of Beam 1" wide per linear foot.	8.3	8.3	9.4	10.4	11.5	12.5	13.5	14.6	15.6	16.7	17.7	18.7	19.8	20.8	21.9	22.9	24.0	25.0	26.0	27.1	28.1	29.2	30.2	31.2

For safe load of any width of beam multiply by width in inches.

For area of cross-section of steel for any width of beam multiply sq. in. of steel by width in inches.

The shearing resistance of the concrete at an average value of $(\frac{2}{3})(40) = 35$ lb. per sq. in. is sufficient for all loads to the right of the 31939 line x-x', considering the maximum shear $\frac{1}{2}wl$.

TABLE 7, Part I.

Use for T Beams.

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$$n=15 \quad f_s = \text{or } < 16000 \text{ lb. per sq. in.} \quad f_c = \text{or } < 650 \text{ lb. per sq. in.}$$

$$k = \frac{pn + \frac{1}{2} \left(\frac{t}{d} \right)^2}{pn + \frac{t}{d}}$$

$$M_s = f_s a_s j d = f_s p j b d^2, \text{ or } \frac{M_s}{b d^2} = f_s p j$$

$$j = \frac{6 - 6 \frac{t}{d} + 2 \left(\frac{t}{d} \right)^2 + \left(\frac{t}{d} \right)^3 / (2pn)}{6 - 3 \frac{t}{d}}$$

$$M_c = f_c \left(1 - \frac{t}{2kd} \right) b t j d, \text{ or } \frac{M_c}{b d^2} = f_c \left(1 - \frac{t}{2kd} \right) \frac{t}{d} j$$

$$\frac{f_s}{f_c} = \frac{n(1-k)}{k}$$

$M_r = \text{safe resisting moment} = \text{lesser of } M_c \text{ and } M_s$

p = 0.002							p = 0.004						
	$\frac{t}{d}$	k	j	Maximum fiber stress corresponding to $f_c = 650$ lb. per sq. in.	Maximum fiber stress in concrete corresponding to $f_s = 16,000$ lb. per sq. in.	$\frac{M_r}{b d^2}$		$\frac{t}{d}$	k	j	Maximum fiber stress in steel corresponding to $f_c = 650$ lb. per sq. in.	Maximum fiber stress in concrete corresponding to $f_s = 16,000$ lb. per sq. in.	$\frac{M_r}{b d^2}$
	0.10	0.269	0.954	26500	392	30.5		0.10	0.266	0.954	14200	730	54.3
	0.11	0.257	0.950	28200	369	30.4		0.11	0.388	0.948	15400	675	58.2
	0.12	0.248	0.946	29500	352	30.3		0.12	0.373	0.944	16400	634	60.4
	0.13	0.240	0.943	30800	337	30.2		0.13	0.360	0.940	17300	600	60.2
	0.14	0.234	0.940	31900	326	30.1		0.14	0.349	0.936	18200	572	59.9
	0.15	0.229	0.937	32800	316	30.0		0.15	0.339	0.932	19000	546	59.7
	0.16	0.225	0.935	33600	310	29.9		0.16	0.331	0.928	19700	528	59.4
	0.17	0.222	0.933	34200	304	29.9		0.17	0.323	0.925	20400	509	59.2
	0.18	0.220	0.931	34600	301	29.8		0.18	0.317	0.921	21000	495	59.0
	0.19	0.218	0.929	35000	297	29.7		0.19	0.312	0.918	21500	484	58.8
	0.20	0.217	0.928	35200	296	29.7		0.20	0.308	0.915	21900	475	58.6
	0.21	0.217	0.928	35200	296	29.7		0.21	0.304	0.914	22300	465	58.5
	0.22	0.217	0.928	35200	296	29.7		0.22	0.300	0.912	22700	457	58.4
								0.23	0.298	0.909	22900	452	58.2
								0.24	0.296	0.907	23200	448	58.0
								0.25	0.294	0.906	23400	444	58.0
								0.26	0.293	0.904	23500	442	57.9
								0.27	0.292	0.903	23600	440	57.8
								0.28	0.291	0.903	23800	438	57.8
								0.29	0.291	0.903	23800	438	57.8
Neutral axis in flange							for greater values of $\frac{t}{d}$						
Values of p for neutral axis at lower edge of slab.	Values below this line correspond to values of p in first column.						Values of p for neutral axis at lower edge of slab.	Values below this line correspond to values of p in first column.					
0.0023	0.23	0.230	0.924	32600	319	29.0	0.0043	0.30	0.300	0.901	22700	457	58.1
0.0025	0.24	0.240	0.918	30800	337	26.8	0.0046	0.31	0.310	0.897	21700	474	66.1
0.0028	0.25	0.250	0.916	29300	356	21.1	0.0050	0.32	0.320	0.893	20700	502	71.5
0.0030	0.26	0.260	0.913	27800	375	23.8	0.0054	0.33	0.330	0.890	19800	525	76.9
0.0033	0.27	0.270	0.911	26500	395	28.2	0.0058	0.34	0.340	0.886	18900	550	82.3
0.0036	0.28	0.280	0.908	25000	415	52.3							
0.0039	0.29	0.290	0.904	23800	436	56.5							

TABLE 7, Part 2.
Use for T beams.

$n=15$ $f_s = \text{or } < 16000 \text{ lb. per sq. in.}$ $f_c = \text{or } < 650 \text{ lb. per sq. in.}$

$P = 0.006$							$P = 0.008$						
$\frac{t}{d}$	k	j	Maximum fiber stress in steel corresponding to $f_s = 16000 \text{ lb. per sq. in.}$	Maximum fiber stress in concrete corresponding to $f_c = 650 \text{ lb. per sq. in.}$	M_r $\frac{\text{lb.-ft.}}{\text{bd}^2}$		$\frac{t}{d}$	k	j	Maximum fiber stress in steel corresponding to $f_s = 16000 \text{ lb. per sq. in.}$	Maximum fiber stress in concrete corresponding to $f_c = 650 \text{ lb. per sq. in.}$	M_r $\frac{\text{lb.-ft.}}{\text{bd}^2}$	
0.10	0.500	0.952	9750	1067	55.7		0.10	0.568	0.952	7920	1400	56.4	
0.11	0.480	0.947	10600	984	59.3		0.11	0.547	0.947	8070	1287	60.9	
0.12	0.462	0.943	11300	917	64.0		0.12	0.530	0.943	8650	1200	65.2	
0.13	0.447	0.939	12000	862	67.8		0.13	0.514	0.938	9230	1125	69.2	
0.14	0.434	0.934	12700	818	71.3		0.14	0.499	0.934	9800	1060	73.1	
0.15	0.422	0.930	13400	779	74.5		0.15	0.486	0.930	10300	1008	76.7	
0.16	0.411	0.926	14000	745	77.5		0.16	0.474	0.925	10800	960	79.9	
0.17	0.402	0.923	14500	716	80.4		0.17	0.463	0.921	11300	920	83.0	
0.18	0.394	0.919	15000	693	83.0		0.18	0.453	0.917	11800	884	85.9	
0.19	0.386	0.915	15500	670	85.6		0.19	0.445	0.914	12100	855	88.8	
0.20	0.380	0.912	15900	654	87.4		0.20	0.437	0.910	12500	827	91.2	
0.21	0.373	0.909	16400	635	89.3		0.21	0.430	0.906	12900	805	93.5	
0.22	0.368	0.906	16700	621	91.0		0.22	0.424	0.903	13200	784	95.6	
0.23	0.364	0.903	17000	610	92.7		0.23	0.418	0.899	13600	766	97.4	
0.24	0.360	0.900	17300	600	94.7		0.24	0.413	0.896	13800	750	99.2	
0.25	0.357	0.897	17500	592	96.1		0.25	0.409	0.893	14100	738	100.7	
0.26	0.354	0.895	17800	584	97.9		0.26	0.405	0.890	14300	726	102.1	
0.27	0.351	0.893	18000	577	99.7		0.27	0.401	0.888	14500	715	103.3	
0.28	0.349	0.891	18200	572	101.5		0.28	0.398	0.885	14700	705	104.4	
0.29	0.348	0.889	18200	569	103.3		0.29	0.395	0.883	14900	696	105.4	
0.30	0.346	0.888	18400	565	105.2		0.30	0.393	0.881	15100	690	106.2	
0.31	0.345	0.887	18500	562	107.2		0.31	0.391	0.879	15200	685	107.0	
0.32	0.344	0.886	18600	559	109.1		0.32	0.389	0.877	15300	678	107.4	
0.33	0.344	0.886	18600	559	111.1		0.33	0.388	0.876	15400	676	108.0	
0.34	0.344	0.885	18600	559	113.0		0.34	0.386	0.874	15500	670	108.2	
							0.35	0.386	0.873	15500	670	108.6	
							0.36	0.385	0.873	15500	668	108.7	
							0.37	0.383	0.873	15700	662	108.7	
							0.38	0.380	0.873	15900	654	107.9	
Neutral axis in flange.							for greater values of $\frac{t}{d}$						
Values of p for neutral axis at lower edge of slab.							Values of p for neutral axis at lower edge of slab.						
Values below this line correspond to values of p in first column.							Values below this line correspond to values of p in first column.						
0.0063	0.35	0.350	0.883	18100	575	89.0	0.0083	0.39	0.390	0.870	15200	682	110.7
0.0068	0.36	0.360	0.879	17300	601	95.6	0.0089	0.40	0.400	0.867	14600	712	112.8
0.0072	0.37	0.370	0.878	16600	627	101.2	0.0095	0.41	0.410	0.864	14000	742	115.2
0.0078	0.38	0.380	0.873	15900	654	107.9	0.0100	0.42	0.420	0.861	13500	744	117.5

TABLE 7, Part 3. Use for T Beams.

$n=15$

$f_s = \text{or } < 16000 \text{ lb. per sq. in.}$

$f_c = \text{or } < 650 \text{ lb. per sq. in.}$

$P=0.010$

$P=0.012$

$\frac{t}{d}$	k	j	Maximum fiber stress in slab corresponding to $f_s = 16,000$ lb. per sq. in.	Maximum fiber stress in slab corresponding to $f_s = 16,000$ lb. per sq. in.	M_r $b d^2$	$\frac{t}{d}$	k	j	Maximum fiber stress in slab corresponding to $f_s = 16,000$ lb. per sq. in.	Maximum fiber stress in slab corresponding to $f_s = 16,000$ lb. per sq. in.	M_r $b d^2$
0.10	0.620	0.951	5780	1740	668	0.10	0.660	0.951	5020	2070	57.1
0.11	0.600	0.947	6500	1600	615	0.11	0.641	0.947	5450	1905	61.9
0.12	0.582	0.942	7000	1484	65.9	0.12	0.624	0.942	5880	1770	66.4
0.13	0.566	0.938	7470	1390	70.1	0.13	0.608	0.938	6290	1650	70.8
0.14	0.551	0.933	7940	1310	74.1	0.14	0.592	0.933	6720	1545	74.9
0.15	0.538	0.929	8360	1240	77.9	0.15	0.579	0.929	7100	1465	78.8
0.16	0.525	0.925	8820	1179	81.6	0.16	0.566	0.924	7480	1390	82.5
0.17	0.513	0.921	9260	1121	84.9	0.17	0.555	0.920	7810	1330	86.1
0.18	0.502	0.916	9670	1075	88.0	0.18	0.544	0.916	8180	1270	89.5
0.19	0.494	0.913	10000	1040	91.0	0.19	0.535	0.912	8470	1225	92.7
0.20	0.486	0.909	10300	1010	93.8	0.20	0.527	0.908	8750	1190	95.6
0.21	0.477	0.905	10700	973	96.4	0.21	0.518	0.904	9080	1145	98.5
0.22	0.470	0.901	11000	944	97.7	0.22	0.511	0.900	9310	1115	101.0
0.23	0.464	0.898	11300	922	101.0	0.23	0.504	0.896	9600	1080	103.4
0.24	0.459	0.894	11500	903	103.1	0.24	0.497	0.893	9870	1050	105.7
0.25	0.453	0.891	11800	888	104.8	0.25	0.490	0.889	10100	1025	107.6
0.26	0.448	0.888	12000	866	106.5	0.26	0.486	0.886	10300	1010	109.6
0.27	0.444	0.885	12200	851	108.1	0.27	0.481	0.883	10500	990	111.4
0.28	0.440	0.882	12400	839	109.5	0.28	0.476	0.879	10700	969	112.9
0.29	0.436	0.879	12600	825	110.5	0.29	0.472	0.876	10900	953	114.4
0.30	0.433	0.876	12800	815	111.7	0.30	0.469	0.874	11000	941	115.9
0.31	0.430	0.874	12900	805	112.7	0.31	0.465	0.871	11200	927	116.9
0.32	0.428	0.872	13000	799	113.5	0.32	0.462	0.868	11300	916	118.1
0.33	0.426	0.870	13100	792	114.3	0.33	0.460	0.866	11400	909	119.1
0.34	0.424	0.868	13200	784	114.9	0.34	0.457	0.863	11600	899	119.8
0.35	0.422	0.866	13300	779	115.2	0.35	0.455	0.861	11700	890	120.5
0.36	0.421	0.865	13400	776	115.8	0.36	0.453	0.860	11800	883	121.3
0.37	0.420	0.864	13400	773	116.4	0.37	0.452	0.858	11800	880	121.7
0.38	0.419	0.862	13500	769	116.3	0.38	0.450	0.856	11900	872	122.2
0.39	0.419	0.862	13500	769	117.0	0.39	0.449	0.855	11900	869	122.7
0.40	0.418	0.861	13600	766	117.0	0.40	0.448	0.854	12000	865	123.0
0.41	0.418	0.861	13600	766	117.0	0.41	0.447	0.853	12000	861	123.0
0.42	0.418	0.861	13600	766	117.0	0.42	0.447	0.852	12000	861	123.0
						0.43	0.447	0.852	12000	861	123.0
						0.44	0.446	0.851	12100	859	123.0
Neutral axis in flange for greater values of $\frac{t}{d}$											
Values of f for neutral axis at lower edge of slab	Values below this line correspond to values of f in first column.										
0.0108	0.43	0.430	0.857	12900	805	119.8					
0.0115	0.44	0.440	0.855	12400	839	121.1					

TABLE 8.

Use for T Beams.

$$f_c = 650 \text{ lb per sq. in.} \quad f_s = 16000 \text{ lb. per sq. in.}$$

$$n = 15$$

$$k = \frac{1}{1 + \frac{f_s}{nf_c}} = 0.379 \quad \frac{z}{t} = \frac{3k - 2\frac{t}{d}}{2k - \frac{t}{d}} \cdot \frac{1}{3} \quad jd = d - z$$

$\frac{t}{d}$	0.10	0.11	0.12	0.13	0.14	0.15	0.16	0.17
$\frac{z}{t}$	0.475	0.471	0.468	0.465	0.462	0.458	0.451	0.448

$\frac{t}{d}$	0.18	0.19	0.20	0.21	0.22	0.23	0.24	0.25
$\frac{z}{t}$	0.448	0.444	0.440	0.436	0.431	0.427	0.422	0.417

$\frac{t}{d}$	0.26	0.27	0.28	0.29	0.30	0.31	0.32	0.33
$\frac{z}{t}$	0.412	0.407	0.402	0.396	0.391	0.384	0.378	0.371

$\frac{t}{d}$	0.34	0.35	0.36	0.37	0.38			
$\frac{z}{t}$	0.364	0.356	0.349	0.341	0.332			

Neutral axis in flange when

$$\frac{t}{d} = k = 0.379$$

TABLE 9.

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$$n=15$$

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

$$M_c = bd^2 f_c L \text{ and } f_c = \frac{M}{bd^2 L} \text{ in which } L = \frac{k}{2}(1 - \frac{k}{3}) + \frac{np'}{k}(k - \frac{d'}{d})(1 - \frac{d'}{d})$$

$$M_s = bd^2 f_s K \text{ and } f_s = \frac{M}{bd^2 K} \text{ in which } K = p(1 - \frac{d'}{d}) - \frac{k^2}{2n(1-k)}(\frac{k}{3} - \frac{d'}{d})$$

$P' = 0.25P$						$P' = 0.5P$					
	P	P'	k	L	K		P	P'	k	L	K
$\frac{d'}{d} = 0.05$	0.005	0.00125	0.307	0.153	0.0045	$\frac{d'}{d} = 0.05$	0.005	0.0025	0.296	0.163	0.0046
	0.01	0.0025	0.394	0.202	0.0088		0.01	0.005	0.373	0.225	0.0090
	0.015	0.00375	0.451	0.239	0.0130		0.015	0.0075	0.421	0.275	0.0134
	0.02	0.005	0.490	0.269	0.0172		0.02	0.01	0.454	0.320	0.0178
	0.025	0.00625	0.521	0.295	0.0214		0.025	0.0125	0.479	0.361	0.0222
	0.03	0.0075	0.546	0.321	0.0256		0.03	0.015	0.499	0.400	0.0266
$\frac{d'}{d} = 0.10$	0.005	0.00125	0.309	0.150	0.0045	$\frac{d'}{d} = 0.10$	0.005	0.0025	0.300	0.158	0.0045
	0.01	0.0025	0.395	0.198	0.0087		0.01	0.005	0.381	0.216	0.0088
	0.015	0.00375	0.454	0.232	0.0125		0.015	0.0075	0.428	0.261	0.0131
	0.02	0.005	0.495	0.260	0.0170		0.02	0.01	0.462	0.302	0.0174
	0.025	0.00625	0.525	0.285	0.0205		0.025	0.0125	0.488	0.338	0.0215
	0.03	0.0075	0.551	0.308	0.0251		0.03	0.015	0.509	0.374	0.0258
$\frac{d'}{d} = 0.15$	0.005	0.00125	0.312	0.148	0.0044	$\frac{d'}{d} = 0.15$	0.005	0.0025	0.305	0.153	0.0044
	0.01	0.0025	0.402	0.194	0.0086		0.01	0.005	0.386	0.207	0.0087
	0.015	0.00375	0.458	0.226	0.0127		0.015	0.0075	0.436	0.249	0.0128
	0.02	0.005	0.499	0.253	0.0167		0.02	0.01	0.471	0.285	0.0169
	0.025	0.00625	0.530	0.275	0.0207		0.025	0.0125	0.498	0.319	0.0210
	0.03	0.0075	0.555	0.296	0.0247		0.03	0.015	0.518	0.350	0.0251
$\frac{d'}{d} = 0.20$	0.005	0.00125	0.314	0.146	0.0044	$\frac{d'}{d} = 0.20$	0.005	0.0025	0.309	0.149	0.0044
	0.01	0.0025	0.404	0.190	0.0086		0.01	0.005	0.392	0.199	0.0086
	0.015	0.00375	0.462	0.221	0.0126		0.015	0.0075	0.442	0.238	0.0126
	0.02	0.005	0.503	0.245	0.0165		0.02	0.01	0.479	0.271	0.0166
	0.025	0.00625	0.534	0.266	0.0204		0.025	0.0125	0.506	0.301	0.0206
	0.03	0.0075	0.560	0.285	0.0243		0.03	0.015	0.527	0.329	0.0245
$\frac{d'}{d} = 0.25$	0.005	0.00125	0.316	0.144	0.0045	$\frac{d'}{d} = 0.25$	0.005	0.0025	0.314	0.146	0.0044
	0.01	0.0025	0.408	0.187	0.0086		0.01	0.005	0.398	0.194	0.0085
	0.015	0.00375	0.465	0.216	0.0125		0.015	0.0075	0.450	0.229	0.0125
	0.02	0.005	0.507	0.239	0.0164		0.02	0.01	0.486	0.258	0.0164
	0.025	0.00625	0.539	0.259	0.0202		0.025	0.0125	0.514	0.287	0.0202
	0.03	0.0075	0.565	0.275	0.0240		0.03	0.015	0.537	0.311	0.0240
$P' = P$						$P' = 1.5P$					
$\frac{d'}{d} = 0.05$	0.005	0.005	0.274	0.183	0.0046	$\frac{d'}{d} = 0.05$	0.005	0.0075	0.256	0.203	0.0046
	0.01	0.01	0.336	0.271	0.0092		0.01	0.015	0.305	0.316	0.0093
	0.015	0.015	0.372	0.348	0.0138		0.015	0.0225	0.332	0.420	0.0140
	0.02	0.02	0.395	0.420	0.0184		0.02	0.03	0.349	0.521	0.0186
	0.025	0.025	0.412	0.491	0.0230		0.025	0.0375	0.361	0.619	0.0232
	0.03	0.03	0.425	0.561	0.0275		0.03	0.045	0.369	0.716	0.0280
$\frac{d'}{d} = 0.10$	0.005	0.005	0.284	0.172	0.0045	$\frac{d'}{d} = 0.10$	0.005	0.0075	0.268	0.186	0.0045
	0.01	0.01	0.349	0.250	0.0089		0.01	0.015	0.322	0.284	0.0090
	0.015	0.015	0.386	0.318	0.0133		0.015	0.0225	0.351	0.374	0.0134
	0.02	0.02	0.410	0.381	0.0177		0.02	0.03	0.369	0.458	0.0178
	0.025	0.025	0.428	0.442	0.0221		0.025	0.0375	0.382	0.541	0.0222
	0.03	0.03	0.442	0.503	0.0265		0.03	0.045	0.392	0.623	0.0267
$\frac{d'}{d} = 0.15$	0.005	0.005	0.292	0.163	0.0044	$\frac{d'}{d} = 0.15$	0.005	0.0075	0.280	0.171	0.0045
	0.01	0.01	0.360	0.233	0.0087		0.01	0.015	0.338	0.256	0.0087
	0.015	0.015	0.399	0.292	0.0129		0.015	0.0225	0.369	0.333	0.0129
	0.02	0.02	0.425	0.347	0.0171		0.02	0.03	0.389	0.404	0.0171
	0.025	0.025	0.444	0.400	0.0213		0.025	0.0375	0.403	0.475	0.0213
	0.03	0.03	0.458	0.451	0.0255		0.03	0.045	0.414	0.544	0.0256
$\frac{d'}{d} = 0.20$	0.005	0.005	0.299	0.154	0.0044	$\frac{d'}{d} = 0.20$	0.005	0.0075	0.292	0.160	0.0044
	0.01	0.01	0.371	0.218	0.0086		0.01	0.015	0.353	0.234	0.0086
	0.015	0.015	0.411	0.270	0.0126		0.015	0.0225	0.386	0.298	0.0126
	0.02	0.02	0.439	0.318	0.0166		0.02	0.03	0.409	0.360	0.0168
	0.025	0.025	0.460	0.364	0.0206		0.025	0.0375	0.424	0.420	0.0206
	0.03	0.03	0.475	0.408	0.0246		0.03	0.045	0.436	0.478	0.0246
$\frac{d'}{d} = 0.25$	0.005	0.005	0.308	0.149	0.0043	$\frac{d'}{d} = 0.25$	0.005	0.0075	0.304	0.152	0.0044
	0.01	0.01	0.382	0.206	0.0085		0.01	0.015	0.365	0.216	0.0084
	0.015	0.015	0.425	0.252	0.0124		0.015	0.0225	0.404	0.271	0.0123
	0.02	0.02	0.454	0.294	0.0162		0.02	0.03	0.428	0.325	0.0161
	0.025	0.025	0.475	0.333	0.0200		0.025	0.0375	0.444	0.373	0.0199
	0.03	0.03	0.491	0.371	0.0238		0.03	0.045	0.457	0.423	0.0237

TABLE 10.

Use for Columns.

$$\frac{P}{P'} = 1 + (n-1)p$$

P = total strength of reinforced column, for stress f_c .

P' = total strength of plain column, for stress f_c .

p	$n=10$	$n=12$	$n=15$	$n=20$
	Values of $\frac{P}{P'}$			
0.005	1.045	1.055	1.070	1.095
0.006	1.054	1.066	1.084	1.114
0.007	1.063	1.077	1.098	1.133
0.008	1.072	1.088	1.112	1.152
0.009	1.081	1.099	1.126	1.171
0.010	1.090	1.110	1.140	1.190
0.011	1.099	1.121	1.154	1.209
0.012	1.108	1.132	1.168	1.228
0.013	1.117	1.143	1.182	1.247
0.014	1.126	1.154	1.196	1.266
0.015	1.135	1.165	1.210	1.285
0.016	1.144	1.176	1.224	1.304
0.017	1.153	1.187	1.238	1.323
0.018	1.162	1.198	1.252	1.342
0.019	1.171	1.209	1.266	1.361
0.020	1.180	1.220	1.280	1.380
0.021	1.189	1.231	1.294	1.399
0.022	1.198	1.242	1.308	1.418
0.023	1.207	1.253	1.322	1.437
0.024	1.216	1.264	1.336	1.456
0.025	1.225	1.275	1.350	1.475
0.026	1.234	1.286	1.364	1.494
0.027	1.243	1.297	1.378	1.513
0.028	1.252	1.308	1.392	1.532
0.029	1.261	1.319	1.406	1.551
0.030	1.270	1.330	1.420	1.570
0.031	1.279	1.341	1.434	1.589
0.032	1.288	1.352	1.448	1.608
0.033	1.297	1.363	1.462	1.627
0.034	1.306	1.374	1.476	1.646
0.035	1.315	1.385	1.490	1.665
0.036	1.324	1.396	1.504	1.684
0.037	1.333	1.407	1.518	1.703
0.038	1.342	1.418	1.532	1.722
0.039	1.351	1.429	1.546	1.741
0.040	1.360	1.440	1.560	1.760

TABLE 11.

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Hooped Column Reinforcement.

Diameter of enclosed con- crete. inches.	Pitch inches.	Section area of hooping. Square inches.	Length of hooping in 1 ft. height. inches.
8	1	0.020	302
	$1\frac{1}{4}$ max.	0.025	242
9	1	0.022	339
	$1\frac{1}{2}$ max.	0.034	226
10	1	0.025	377
	$1\frac{5}{8}$ max.	0.041	232
11	$1\frac{1}{8}$	0.031	369
	$1\frac{3}{4}$ max.	0.048	237
12	$1\frac{1}{4}$	0.037	362
	2 max.	0.060	226
13	$1\frac{3}{8}$	0.045	356
	$2\frac{1}{8}$ max.	0.069	230
14	$1\frac{3}{8}$	0.048	384
	$2\frac{1}{4}$ max.	0.079	234
15	$1\frac{1}{2}$	0.056	377
	$2\frac{1}{2}$ max.	0.094	226
16	$1\frac{5}{8}$	0.065	371
	$2\frac{1}{2}$ max.	0.100	241
17	$1\frac{3}{4}$	0.074	366
	$2\frac{1}{2}$ max.	0.106	256
18	$1\frac{7}{8}$	0.084	362
	$2\frac{1}{2}$ max.	0.112	271
19	$1\frac{7}{8}$	0.089	382
	$2\frac{1}{2}$ max.	0.149	287
20	2	0.100	377
	$2\frac{1}{2}$ max.	0.125	302
21	$2\frac{1}{8}$	0.112	373
	$2\frac{1}{2}$ max.	0.131	317
22	$2\frac{1}{4}$	0.124	369
	$2\frac{1}{2}$ max.	0.137	332
23	$2\frac{1}{4}$	0.130	386
	$2\frac{1}{2}$ max.	0.144	347
24	$2\frac{3}{8}$	0.142	381
	$2\frac{1}{2}$ max.	0.150	362
25	$2\frac{1}{2}$ "	0.156	377
26	$2\frac{1}{2}$ "	0.162	392
27	$2\frac{1}{2}$ "	0.169	407
28	$2\frac{1}{2}$ "	0.175	422
29	$2\frac{1}{2}$ "	0.181	437
30	$2\frac{1}{2}$ "	0.187	452
31	$2\frac{1}{2}$ "	0.194	467
32	$2\frac{1}{2}$ "	0.200	483
33	$2\frac{1}{2}$ "	0.206	498
34	$2\frac{1}{2}$ "	0.212	513
35	$2\frac{1}{2}$ "	0.219	528
36	$2\frac{1}{2}$ "	0.225	543

TABLE 12.

Maximum diameter of round or square stirrups.

$$l = 2.4 \frac{u}{f_s} d$$

Values of $2.4 \frac{u}{f_s}$ for different values of tension and bond.

allowable unit bond stress u .	Allowable unit tension in stirrups f_s lb. per sq. in.				
lb. per sq. in.	12000	14000	15000	16000	20000
80	0.016	0.014	0.013	0.012	0.010
100	0.020	0.017	0.016	0.015	0.012
120	0.024	0.020	0.019	0.018	0.014
150	0.030	0.026	0.024	0.022	0.018

TABLE 13.

Minimum length of embedment of inclined rods.
(ϕ or θ)

$$l' = \frac{f_s}{4u} \text{ diameters.}$$

Values of $\frac{f_s}{4u}$ for different values of tension and bond.

Allowable unit bond stress u .	Allowable unit stress in inclined rods f_s lb. per sq. in.				
lb. per sq. in.	12000	14000	15000	16000	20000
80	37	44	47	50	62
100	30	35	38	40	50
120	25	29	31	33	41
150	20	23	25	27	33

Appendix No. 4.

Diagrams 1 to 5, inclusive.

Used for the design of beams and
slabs. Bending moments, spacing
of steel.

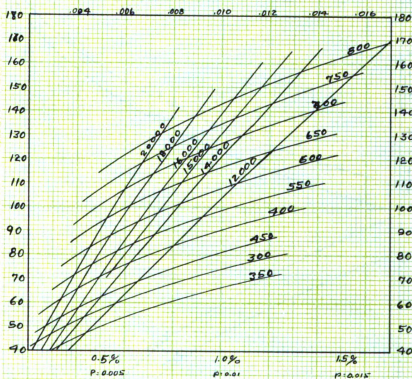
I N D E X for

Appendix No. 2.

Diagram 1.	For design of rectangular beams	233
"	2. Curves for j and k , rectangular beams,	237
"	3. Spacing of round rods in slabs,	238
"	4. Spacing of square rods in slabs,	238
"	5. Bending moments, uniformly distributed loads,	240
"	6. Twisting moments for designing slabs,	241
"	7. For designing T-beams,	242
"	8. " " " " "	243

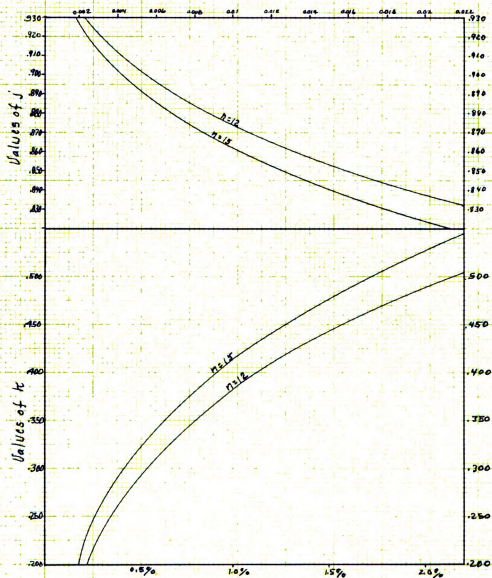
Diagram 2.
For Design of Rectangular Beams.

$$n = 15$$



Percentage Reinforcement

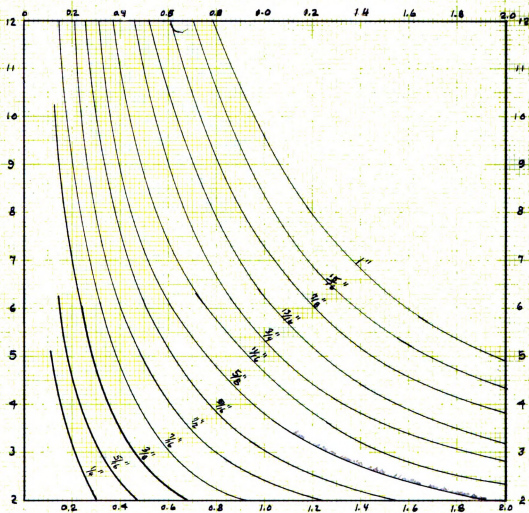
Diagram 2.
Curves for j and k for
Rectangular Beams.



Percentage of Reinforcement.

Diagram 3.

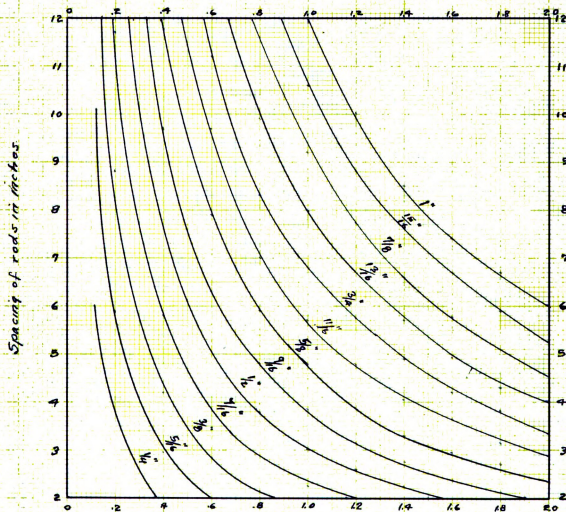
Spacing of round rods in slabs.



Sectional Area of Steel per
foot of slab.

Diagram 4

Spacing of square rods in slabs.



Sectional area of steel per foot of
slab in inches.

Diagram 5.

Bending moments for uniformly
distributed loads.

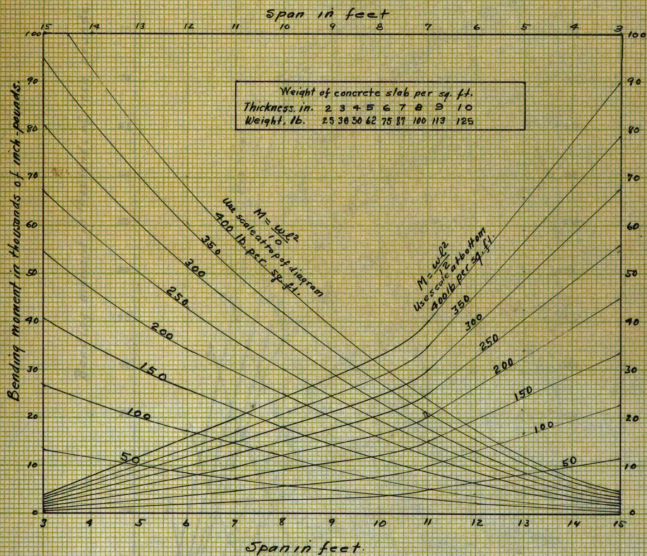
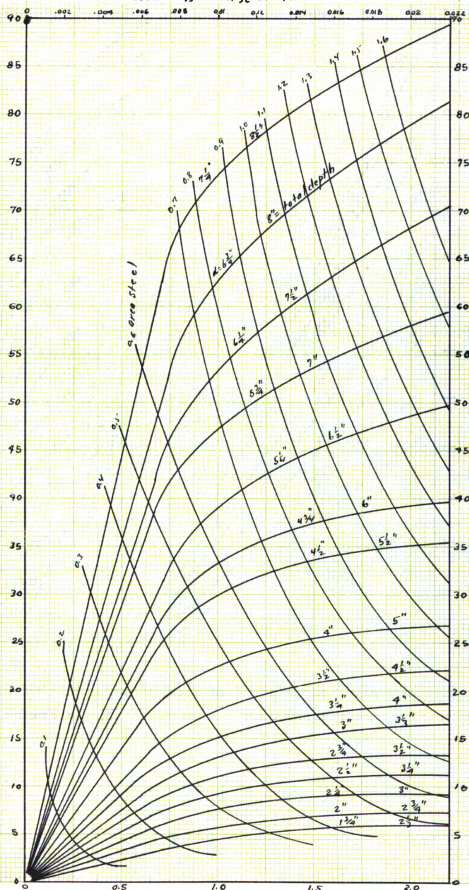


Diagram 6.
For designing slabs
based on $f_s = 16000$, $f_c = 600$, $n = 15$.

Bending moment in thousands of inch. pounds.



Percentage Reinforcement,

Diagram 7.
Use for T beams.
Based on $f_s = 16000$, $n = 15$.

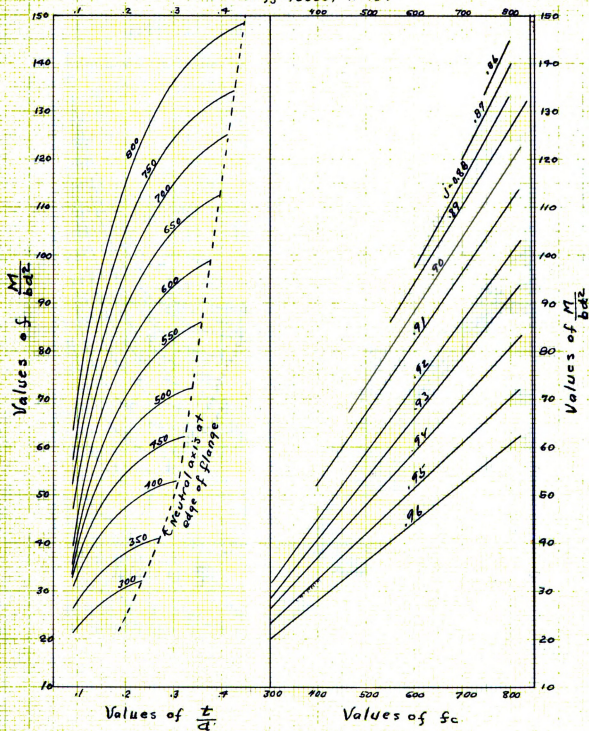
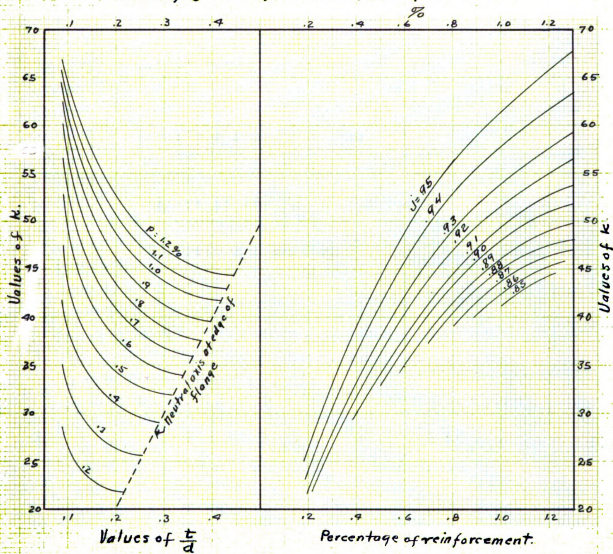


Diagram 8.
Use for T beams.

Values of j and k for various percentages of steel.



APPENDIX No. 8.

Report of the Joint Committee on
Concrete and Reinforced Concrete.

SECOND REPORT OF JOINT COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

Report of the Joint Committee as it appeared in the
Proceedings of the American Society of Civil Engineers,
February, 1913. Introduction omitted.

A. ADAPTABILITY OF CONCRETE AND REINFORCED CONCRETE.

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that they may be considered the recognized materials of construction. They have proved satisfactory materials, when properly used, for those purposes for which their qualities make them particularly suitable.

1. Uses.

Concrete is a material of very low tensile strength, and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly on its durability, its fire resistive qualities, its strength in compression, its relatively low cost, and its adaptability to placing, especially where space is cramped or limited. Its strength generally increases with age.

Concrete is well adapted for structures in which the principal stresses are compressive, such as foundations, dams, retaining and other walls, tunnels, piers, abutments, short columns, and, in many cases, arches. In the design of massive concrete, the tensile strength of the material in resisting principal stresses must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross bending may exist. In structures resisting lateral forces, it possesses advantages over plain concrete in that it may be designed so as to utilize more fully the strength rather than the weight of the material. Metal reinforcement may also be of value in distributing cracks due to shrinkage and temperature changes.

2. Precautions.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, faulty execution, and premature removal of forms.

The defects in a design may be many and various. The computations and assumptions on which they are based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses may be excessive, or the details of the design defective.

Articulated concrete structures designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete, and such structures are not recommended.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of poor aggregates, especially sand, which has not been tested, is a common source of defect. Inferior concrete is frequently due also to lack of experience on the part of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of metal for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, but the steel used for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution, careless workmanship, and too early removal of forms may generally be attributed to unintelligent or insufficient supervision.

3. Responsibility and Supervision.

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

While other engineering structures on the safety of which human lives depend are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Therefore, means for more adequate supervision and inspection should be provided.

The execution of the work should not be separated from the design, as intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications, therefore, should have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete for the purpose of fixing the responsibility and providing for adequate supervision during construction:

(a) Before work is commenced, complete plans shall be prepared, accompanied by specifications, stress computations, and descriptions showing the general arrangement and all details. The plans shall show the size, length, dimension for points of bending, the exact position of all reinforcement, including stirrups, ties, hooking, and splicing. The computations shall give the loads assumed separately, such as dead and live loads, wind, and impact, if any, and the resulting stresses.

(b) The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

(c) The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

(d) The drawings and specifications shall be signed by the engineer and the contractor.

(e) Plans and specifications for all public structures should be approved by a legally authorized State or City Official, and copies of such plans and specifications placed on file in his office.

(f) The approval of plans and specifications by other

authorities shall not relieve the engineer or the contractor of responsibility.

(g) Inspection during construction shall be made by competent inspectors employed by and under the supervision of the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes, and spacing of the reinforcement.
4. The proportioning, mixing and placing of the concrete.
5. The strength of the concrete, by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

(h) Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that one and three-quarters times the calculated working stresses in critical parts are reached, and such loads shall cause no injurious permanent deformations. Load tests shall not be made until after 60 days of hardening.

4. Destructive Agencies.

(a) Corrosion of Metal Reinforcement. Tests and experience indicate that steel sufficiently encased in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 inch in thickness. If the concrete is porous so as to be readily permeable by water, as when the concrete is laid with a dry consistency, the metal may corrode on account of the presence of moisture and air.

(b) Electrolysis. The most recent experimental data available on this subject seem to show that while reinforced concrete structures may, under certain conditions, be injured by the flow

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P.

of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former, or anode effect, decreases much more rapidly than the voltage, and almost if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even on very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small quantities, are very much more susceptible to the effects of electric currents than normal concrete, both the anode and the cathode effects progressing much more rapidly in the presence of chlorine.

There is great weight of evidence to show that normal reinforced concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

The results of experiments now in progress may yield more conclusive information on this subject.

(c) Sea Water. The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. Sea walls out of the range of frost action have been standing for many years without apparent injury. In many harbors where the water is brackish, through rivers discharging into them, serious disintegration has taken place. This has occurred chiefly between low and high tide levels, and is due, evidently, in part by frost. Chemical action also appears to be indicated by the softening of the mortar. To effect the best resistance to sea water, the concrete must be proportioned, mixed, and placed so as to prevent the penetration of sea water into the mass or through the joints. The cement should be of such chemical composition as will best resist the action of sea water; the aggregates should be

carefully selected, graded, and proportioned with the cement so as to secure the maximum possible density; the concrete should be thoroughly mixed; the joints between old and new work should be made water tight; and the concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(d) Acids. Concrete of first-class quality, thoroughly hardened, is affected appreciably by only strong acids which seriously injure other materials. A substance like manure is injurious to green concrete, but after the concrete has hardened thoroughly it resists the action of such acid satisfactorily.

(e) Oils. When concrete is properly made and the surface is carefully finished and hardened, it resists the action of such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which result in a disintegration of the concrete in contact with them.

(f) Alkalies. The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete stone, brick, iron, and other materials are injured under certain conditions. It would seem that at the level of the ground water, in an extremely dry atmosphere, such structures are disintegrated, through the rapid crystallization of the alkaline salts, resulting from the alternate wetting and drying of the surface. Such destructive action can be prevented by the use of a protective coating, and is minimized by securing a dense concrete.

B. Materials.

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The importance of the quality of the materials used cannot be overestimated, and not only the cement but also the aggregates should be subject to such definite requirements and tests as will insure concrete of the desired quality.

1. Cement.

There are available for construction purposes: Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

(a) PORTLAND CEMENT. This is the finely pulverized product resulting from the calcination to incipient fusion of an intimate

mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparative narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

(b) NATURAL CEMENT. This is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. Although the limestone must have a certain composition, this composition may vary within much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly, nor is it as uniform in composition, as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of natural cement and a leaner mixture of Portland cement that will develop the same strength.

(c) PUZZOLAN. This is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or natural cement, is not used extensively, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

(d) SPECIFICATIONS. The cement should meet the requirements of the Standard Methods of Testing and Specifications for Cement, or as may be hereafter amended, the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and others.

2. Aggregates.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density or a minimum percentage of voids.

(a) FINE AGGREGATES. This should consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes $\frac{1}{4}$ inch in diameter; it is per-

ferable that it be of silicious material, and should be clean, coarse, free from dust, soft particles, vegetable loan, or other deleterious matter; and not more than 5 per cent should pass a sieve having 100 meshes per linear inch. Fine aggregate should always be tested.

Fine aggregate should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand. If the aggregates be of poorer quality, the proportion of cement in the mortar should be increased to secure the desired strength.

If the strength developed by the aggregate in the 1:3 mortar is less than 70 per cent of the strength of the Ottawa sand mortar, the material should be rejected. To avoid the removal of any coating on the grains, which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined on a separate sample for correcting weight. From 10 to 40 per cent more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(c) COARSE AGGREGATES. This should consist of crushed stone or gravel which is retained on a screen having holes $\frac{1}{4}$ inch in diameter, and graded from the smallest to the largest particles; it should be clean, hard, durable, and free from all deleterious matter. Aggregates containing dust, and soft, flat, or elongated particles, should be excluded from important structures.

The maximum size of the coarse aggregates is governed by the character of the construction.

For reinforced concrete and for small masses of unreinforced concrete, the aggregate must be small enough to produce with the mortar a homogeneous concrete of viscous consistency which will pass readily between and easily surround the reinforcement and fill all parts of the forms.

For concrete in large masses, the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one, although it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced concrete structures. It may be allowable in masonry for very light loads or

for fire protection purposes. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3. Water.

The water used in mixing concrete should be free from oil, acid, alkalies, or organic matter.

4. Metal Reinforcement.

The Committee recommends, as a suitable material for reinforcement, steel fulfilling the requirements for structural steel reinforcement of the specifications adopted by the American Railway Engineering Association (Section H).

Where little bending or shaping is required, and also for reinforcement for shrinkage and temperature stresses, material fulfilling the requirements of the Specifications adopted by the American Railway Engineering Association for high-carbon steel (Section H) may be used, adopting the same unit stress as hereinafter recommended for structural grade material.

For the reinforcement of slabs, small beams, or minor details, or for reinforcement for shrinkage and temperature stresses, wire drawn from bars of the grade of rivet steel may be used, with the unit stresses hereinafter recommended.

The reinforcement should be free from excessive rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

C. PREPARING AND PLACING MORTAR AND CONCRETE.

1. Proportions.

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view of securing as nearly as possible a maximum density.

(a) UNIT OF MEASURE. The unit of measure should be the cubic foot. A bag of cement, containing 94 pounds net, should be considered the equivalent of 1 cubic foot.

The measurement of the fine and coarse aggregates should be by loose volume.

(b) RELATION OF FINE AND COARSE AGGREGATES. The fine and coarse aggregate should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by individual judgment, using correspondingly higher

proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained or the proportions changed to meet the varying sizes.

(c) RELATION OF CEMENT AND AGGREGATES. For reinforced concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are generally preferable, and in massive masonry, or rubble concrete, a mixture of 1:2 or even 1:1.2 may be used.

These proportions should be determined by the strength of the wearing qualities required in the construction at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is an excellent guide as to the proper proportions for any particular case.

For all important construction, advanced tests should be made of concrete, of the materials, proportions, and consistency to be used in the work. These tests should be made under laboratory conditions to obtain uniformity in mixing, proportioning, and storage, and in case the results do not conform to the requirements of the work, aggregates of a better quality should be chosen, or richer proportions used to obtain the desired results.

2. Mixing.

The ingredients of concrete should be thoroughly mixed, and the mixing should be continued until the cement is uniformly distributed and the mass is uniform in color and homogeneous. As the maximum density and greatest strength of a given mixture depends largely on thorough and complete mixing, it is essential that the work of mixing should receive special inspection.

Inasmuch as it is difficult to determine, by visual inspection, whether the concrete is uniformly mixed, especially where limestone or aggregates having the color of cement are used, it is essential that the mixing should occupy a definite period of time. The minimum time will depend on whether the mixing is done by machine or hand.

(a) MEASURING INGREDIENTS. Methods of measurement of the proportions of the various ingredients should be used which will secure separate and uniform measurements of cement, fine aggregate, coarse aggregate, and water, at all times.

(b) MACHINE MIXING. When the conditions will permit, a machine mixer of a type which will insure uniform proportioning

of the materials throughout the mass should be used, as a more uniform consistency can be thus obtained. The mixing should continue for a minimum time of at least one minute after all the ingredients are assembled in the mixer.

(c) HAND MIXING. When it is necessary to mix by hand, the mixing should be on a water tight platform, and especial precautions should be taken to turn all the ingredients together at least six times and until they are homogeneous in appearance and color.

(d) CONSISTENCY. The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement when used, and which, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

(e) RETEMPERING. Mortar or concrete should not be remixed with water after it has partly set.

3. Placing Concrete.

(a) METHODS. Concrete, after the completion of the mixing, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no circumstances should concrete be used that has partly set. A slow setting cement should be used when a long time is likely to occur between mixing and placing.

Concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper places by gravity, and the surplus water has been forced to the surface. Special care should be exercised to prevent the formation of laitance, which hardens very slowly and forms a poor surface on which to deposit fresh concrete. All laitance should be removed.

Before depositing concrete, the reinforcement should be carefully placed in accordance with the plans, and adequate means provided to hold it in its proper position until the concrete has been deposited and compacted; care should be taken to see that the forms are substantial and thoroughly wetted (except in freezing weather) or oiled, and that the space to be occupied by the concrete is free from debris. When the placing of concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance,

thoroughly wetted, and then slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least seven days.

(b) FREEZING WEATHER. Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials covered with ice crystals or containing frost, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

As the coarse aggregate forms the greater portion of the concrete, it is particularly important that this material be heated to well above the freezing point.

(c) RUBBLE CONCRETE. Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced by the use of clean stones thoroughly embedded in the concrete as near together as it is possible and still entirely surrounded with concrete.

(d) UNDER WATER. In placing concrete under water, it is essential to maintain still water at the place of deposit. The use of tremies, properly designed and operated, is a satisfactory method of placing concrete through water. The concrete should be mixed very wet (more than is ordinarily permissible) so that it will flow readily through the tremmie and into the place with practically a level surface.

The coarse aggregate should be smaller than ordinarily used, and never more than 1 inch in diameter. The use of gravel facilitates mixing and assists the flow of concrete through the tremie. The mouth of the tremie should be buried in the concrete so far that it is at all times entirely sealed and the surrounding water prevented from forcing itself into the tremie; the concrete will then discharge without coming in contact with the water. The tremie should be suspended so that it can be lowered quickly when it is necessary either to choke off or prevent too rapid flow; the lateral flow should preferably be not more than 15 feet.

The flow should be continuous, in order to produce a monolithic mass and prevent the formation of laitance in the interior.

In large structures it may be necessary to divide the mass of concrete into several small compartments or units, filling one at a time. With proper care, it is possible in this manner to obtain as good results under water as in the air.

D. Forms.

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours; and they shall be tight in order to prevent leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent and responsible person, especially where the atmospheric conditions are unfavorable.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than for plain or massive concrete, and that forms for floors, beams, and similar horizontal structures should remain in place much longer than for vertical walls.

When the concrete gives a distinctive ring under the blow of a hammer, it is generally an indication that it has hardened sufficiently to permit the removal of the forms with safety. If, however, the temperature is such that there is any possibility that the concrete is frozen, this test is not a safe reliance, as frozen concrete may appear to be very hard.

E. DETAILS OF CONSTRUCTION.

1. Joints.

(a) CONCRETE. For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joints in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams, and floor slabs perpendicular to the plane of their surfaces.

Girders should never be constructed over freshly formed columns without permitting a period of at least 2 hours to elapse, thus providing for settlement or shrinkage in the columns.

Shrinkage and contraction joints may be necessary in concrete subject to great fluctuations in temperature. The frequency of these joints will depend, first, on the range of temperature to which the concrete will be subjected, and second, on the quantity and position of the reinforcement. These joints should be determined, and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, contraction joints should be provided at intervals of from 25 to 50 feet and with reinforcement from 50 to 80 feet (the smaller the height and thickness, the closer the spacing) throughout the length of the structure. To provide against the structure being thrown out of line by unequal settlement, each section of the wall should be tongued and grooved into the adjoining section, a groove should be formed in the surface of the concrete at vertical joints in walls and abutments.

Shrinkage and contraction joints should be lubricated by either an application of petroleum residuum oil or a similar material so as to permit a free movement at the joint when the concrete expands or contracts.

The insertion of a sheet of a sheet of copper or zinc, or even tarred paper, will be found advantageous in securing expansion and contraction at the joint.

(c) REINFORCEMENT. Wherever it is necessary to splice tension reinforcement, the length of lap should be determined on the basis of the safe bond stress, the stress in the bar, and the shearing resistance of the concrete at the point of splice; or a connection should be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, bars more than $\frac{3}{4}$ inches in diameter, not subject to tension, should be properly squared and cutted in a suitable sleeve; smaller bars may be treated as indicated for tension reinforcement, or the stress may be cared for by embedment in large masses of concrete. At foundations, bearing plates should be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress of the steel to the concrete by means of the bearing and bond resistance; in no case shall the ends of the bars be permitted merely to rest on concrete.

2. Shrinkage and Temperature Changes.

Shrinkage of concrete due to hardening and contraction from temperature changes, causes cracks the size of which depends on the

extent of the mass. The resulting stress are important in monolithic construction, and should be considered carefully by the designer; they cannot be counteracted successfully, but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long continuous lengths of concrete, it is better to provide shrinkage joints at points in the structure where they will do little or no harm. Reinforcement is of assistance, and permits longer distances between shrinkage joints than when no reinforcement is used.

Smaller masses or thin bodies of concrete should not be joined to larger or thicker masses without providing for shrinkage at such points. Fillets similar to those used in metals castings, but of larger dimensions, for gradually reducing from the thicker to the thinner body, are of advantage.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence, in placing the concrete, construction joints should be made on horizontal and vertical lines; and if possible, at points where joints would naturally occur in dimension-stone masonry.

3. Fire-proofing.

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests made thus far, indicate that concrete, on account of its low rate of heat conductivity and the fact that it is incombustible, may be used safely for fire-proofing purposes.

The dehydration of concrete probably begins at about 500°F., and is completed at about 800°F., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is affected by fire remains in position and affords protection to the concrete beneath it.

The thickness of the protective coating required depends on the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for

ordinary conditions, it is recommended that the metal in girders and columns be protected by a minimum of 2 inches of concrete; that the metal in beams be protected by a minimum of $1\frac{1}{2}$ inches of concrete; and that the metal in floor slabs be protected by a minimum of 1 inch of concrete.

It is recommended that, in monolithic concrete columns, the concrete to a depth of $1\frac{1}{2}$ inches be considered as protective covering, and not included in the effective section.

It is recommended that the corners of columns, girders, and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

4. Water-proofing.

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under moderate pressure.

A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, provided the concrete itself is impervious, cracks may be so reduced by horizontal and vertical reinforcement properly proportioned and located, that they are too minute to permit leakage, or are soon closed by infiltration of silt.

Coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

For retaining and similar walls in direct contact with the earth, the application of one or two coatings of hot coal-tar pitch to the thoroughly dried surface of concrete is an efficient method of preventing the penetration of moisture from the earth.

5. Surface finish.

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with its use is the character of the

finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work should be conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, making some special treatment desirable. A treatment of the surface, either by scrubbing it while green or by tooling it after it is hard, which removes the film of mortar and brings the aggregates of the concrete into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. The plastering of surfaces, as ordinarily applied, should be avoided, for even, if carefully done, it is likely to peel off under the action of frost or temperature changes.

F. DESIGN.

1. Massive Concrete.

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses, except in slight amounts to resist indirect stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but, in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete, therefore, will often be suitable for massive concrete structures.

It is desirable, generally, to provide joints at intervals, to localize the effect of contraction.

Massive concrete is suitable for dams, retaining walls, and piers and short columns in which the ratio of length to least width is relatively small. Under ordinary conditions, this ratio should not exceed six. It is also suitable for arches or moderate span, where the conditions as to foundations are favorable.

2. Reinforced Concrete.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the ocean, where both compression and tension exist; it is also advantageous

in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design, therefore, will relate mainly to the analysis of beams and columns.

3. General Assumptions.

(a) LOADS. The loads or forces to be resisted consist of:

1. The Dead Load.—This includes the weight of the structure and fixed loads and forces.
2. The Live Load.—This consists of the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses thus determined. In the case of high buildings, the live loads on columns may be reduced in accordance with the usual practice.

(b) LENGTHS OF BEAMS AND COLUMNS.—The span length for beams and slabs shall be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

(c) INTERNAL STRESSES.—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations will be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains a plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams therefore, is rectilinear.
4. In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.
5. Perfect cohesion is assumed between the concrete and reinforcement. Under compressive stresses, the two materials, therefore, are stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15, except as modified in Section 3.
7. Initial stress in the reinforcement, due to contraction or expansion in the concrete is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

The deflection of beams is affected by the tensile strength developed throughout the length of the beam. For calculations of deflections, a value of 3 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

4. T-Beams.

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

- (a) It shall not exceed one-fourth of the span length of the beam;
- (b) Its overhanging width on either side of the web shall not exceed four times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

Beams in which the tee form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form, the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in the design.

5. Floor Slabs.

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds one and five tenths times its width, the entire load should be carried by the transverse reinforcement. Square slabs may well be reinforced in both directions.

The continuous flat slab with multiple-way reinforcement is a type of construction used quite extensively, and has recognized

advantages for special conditions, as in the case of warehouses with large, open, floor space. At present, a considerable difference of opinion exists among engineers as to the formulas and constants which should be used, but experience and tests are accumulating data which it is hoped will in the near future permit the formulation of the principles of design for this form of construction.

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and its distribution will depend on the relative stiffness of the slab and the supporting beam. The distribution under ordinary conditions of construction may be expected to be that in which the load on the beam varies in accordance with the ordinates of a parabola having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained, and the moments in the beam calculated accordingly.

6. Continuous Beams and Slabs.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at the points of negative moment, and the stresses in concrete recommended in Section F, should not be exceeded, (These stresses have been increased in some parts in a later supplement to this report). In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

- (a) That for floor slabs, the bending moments at center and at support be taken at $wl^2/12$ for both dead and live loads, where w represents the load per linear foot and l the length of span.
- (b) That for beams, the bending moment at center and at support for interior spans be taken at $wl^2/12$ and for end spans it be taken at $wl^2/10$ for center and adjoining support, for both dead and live loads.
- (c) In the case of beams and slabs continuous for two spans only, the bending moment at the center support should be taken at $wl^2/8$ and near the middle of the span at $wl^2/10$.
- (d) At the ends of continuous beams, the amount of negative moment which will develop will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. There will usually be some restraint, and there is likely to be considerable. Provision should be made for the negative bending moment, but, as its amount will depend on the form of construction, the coefficient cannot be specified here, but must be left to the judgment of the designer.

For spans of unusual lengths, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by a or b , the negative moment at the support should not be taken as less than the values given.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of the stress, in accordance with the provisions of Section F, Part 3, c-6. In the case of cantilever and continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support and beyond the point of inflection to develop the requisite bond strength.

7. Bond Strength, and Spacing of Reinforcement.

Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the reinforcement is bent up near the end, the bond stress at places in both the straight bars and the bent bars will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams, full tensile stress exists in the reinforcing bars at the point of support, and the bars must be anchored in the support sufficiently to develop this stress.

In the case of anchorage of bars, an additional length of bar must be provided beyond that found on the assumption of uniform bond stress, for the reason that, before the bond resistance at the end of the bar can be developed, the bar may have begun to slip at another point, and "running resistance" is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength; but it should be recognized that, even with a deformed bar, initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond stress may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard such anchorage may properly be used in special cases. Anchorage furnished by short ends at a right angle is less effective than hooks con-

sisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than three diameters, from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than one inch. The use of more than two layers is to be discouraged, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down.

8. Diagonal Tension and Shear.

When a reinforced concrete beam is subjected to flexural action, diagonal tensile stresses are set up. If, in a beam not having web reinforcement, these stresses exceed the tensile strength of the concrete, failure of the beam will ensue. When web reinforcement, made up of stirrups, or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail; but even in this case, at the beginning of loading, the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For homogeneous beams, the analytical treatment of diagonal tension is not very complex - the diagonal tensile stress is a function of the horizontal and vertical shearing stresses and of the horizontal tensile stress at the point considered, and as the intensity of these three stresses varies from the neutral axis to the remotest fiber, the intensity of the diagonal tension will be different at different points in the section, and will change with different proportionate dimensions of length to depth of beam. For the composite structure of reinforced concrete beams, an analysis of the web stresses, and particularly of the diagonal tensile stresses is very complex; and when the variations due to a change from no horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circumstances, in designing, recourse is had to the use of the calculated vertical shearing stress as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shear-

ing stress is not the numerical equivalent of the diagonal tensile stress, and even that there is not a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams, it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces, and bent up bars, and it is here recommended for calculations in designing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may, under the best conditions, increase the vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used. Where vertical stirrups are used without being secured to the longitudinal reinforcement, the force transmitted between longitudinal bar and stirrup must not be greater than can be taken through the concrete, and care must be taken to provide for the larger bond stress developed in the longitudinal bars with this construction than exists in the absence of stirrups. Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided. Where the longitudinal bars are bent up, the points of bending of the several bars should be distributed along a portion of the length of the beam in such a way as to give efficient web reinforcement over the portion of the length of the beam in which it is needed. The higher resistance to diagonal tension failures given by unit

frames having the stirrups and bent up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked and cracked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at the support, even though the shear at the latter point be much greater.

The longitudinal spacing of stirrups or diagonal members, or the distribution of the points of bending of adjacent bent-up bars, should not exceed three-fourth the depth of the beam.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that the proper connection must always be made between stirrups or other web reinforcement and the longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as in the case near the supports in a continuous beam, web reinforcement, to be effective, must be looped over, or wrapped around, or be connected with, the longitudinal tension reinforcing bars at the top of the beam, in the same way it is necessary at the bottom of the beam at sections where the bending moment is positive and the tension reinforcing bars are at the bottom of the beam.

Inasmuch as the smaller the longitudinal deformations in the horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by arranging and proportioning the horizontal reinforcement so that the unit stresses at points of large shear shall be relatively low.

Where pure shearing stress occurs, or shearing stress combined with but a small amount of tensile stress in the concrete, as when a concentrated load rests on a slab, or other forms of punching shear are produced, or in the case of compression pieces, the element of tension will not need consideration, and the permissible limit of the shearing stress will be higher than the allowable limit when

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this stress is used as a means of comparing diagonal tensile stress. The working values recommended are given in Section G, Working Stresses.

9. Columns.

By columns are meant compression members of which the ratio of unsupported length to least width exceeds about six, and when these are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Section E, Part 3, or, in the case of hooped columns, or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by longitudinal bars, or bands, hoops, or spirals, together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of closely spaced hooping is greatly to increase the toughness of the column and its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of toughening are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

Composite columns of structural steel and concrete in which the steel forms a column by itself, should be designed with caution. To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel will generally take the greater part of the load. When this type of column is used, the concrete should not be relied on to tie the steel units together or to transmit stresses from one unit to another. The units should be adequately tied together by tie-plates, or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection, and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and

the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

- (a) Columns with longitudinal reinforcement only, to the extent of not less than 1 per cent and not more than 4 per cent: the unit stress recommended for axial compression in Section G, Part 3.
- (b) Columns with reinforcement of bands, hoops, or spirals, as hereinafter specified, stresses 20 per cent higher than given for a, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.
- (c) Columns reinforced with not less than 1 per cent and not more than 4 per cent of longitudinal bars, and with bands, hoops, or spirals, as hereinafter specified: stresses 45 per cent higher than given for a, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

The foregoing recommendations are based on the following conditions.

In all cases, longitudinal reinforcement is assumed to carry its proportion of stress, in accordance with Part 3. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall not be less than 1 per cent of the volume of the column enclosed. The clear spacing of such hooping shall be not greater than one-sixth of the diameter of the enclosed column, and preferably not greater than one-tenth, and in no case more than $2\frac{1}{2}$ inch. Hooping is to be circular, and the ends of the bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much on the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increase beyond five. The working stresses recommended are for hooped columns with a length of not more than eight diameters of the hooped core.

Bending stresses due to eccentric loads and lateral forces must be provided for by increasing the section until the maximum stress does not exceed the values above specified; and, where

tension is possible in the longitudinal bars, adequate connection between the ends of the bars must be provided to take this tension.

10. Reinforcing for Shrinkage and Temperature Stresses.

When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that cracks may occur in the mass, unless precautions are taken to distribute the stresses so as to prevent the cracks altogether, or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be effective, therefore, reinforcement (in amount generally not less than 1 per cent) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. The allowable size and spacing of cracks depends on various conditions, such as the necessity for water tightness, the importance of appearance of the surface, and the atmospheric changes.

G. WORKING STRESSES.

The working stresses as recommended by the Joint Committee and made a part of this report will be found in the introduction to this thesis, Pages 6 to 8.

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