DESIGN OF A SERVICE GARAGE AND OFFICE BUILDING

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

Frank Grove Foster, Jr.

1946

Design of A Service Garage

And Office Building

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

Frank Grove Foster, Jr.

Candidate for the Degree of

Bachelor of Science

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ACKNOWLEDGMENTS

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BIBLIOGRAPHY

Pulver - "Construction Estimates and Costs".

Sutherland & Reese - "Reinforced Concrete Design".

Hool & Johnson - "Concrete Engineers Handbook".

Eshback - "Handbook of Engineering Fundamentals".

A.I.S.C. - "Manual of Steel Construction".

Parker - "Simplified Design of Structural Steel".

The A.C.I. Building Code.

Lansing Building and Safety Code - (hereinafter referred to as L.B.C.)

Publications of the Portland Cement Association:

"Simplified Design of Concrete Floor Systems" (hereinafter referred to as S.D.C.F.S.)

"Reinforced Concrete Design Handbook" - (hereinafter referred to as R.C.D.H.)

Bulletin ST 51 - "Concrete Floors on Ground".

<u>CONTENTS</u>

PURPOSE AND SCOPE

OF THESIS

THE PURPOSE of this thesis is to present a practical design of a building suitable for use by the college for a service garage with separate office space provided in the second story. The inadequacy of the existing garage facilities make this a timely subject for thesis and the variety of design features entailed make it an excellent student problem as well as one which a practicing engineer might be called upon to solve.

The existing service garage space could be conveniently utilized for the parking of the trucks operated by the Buildings and Grounds Department whose offices are in the same building. This would also serve to relieve the parking problem in this congested area as well as protect the equipment.

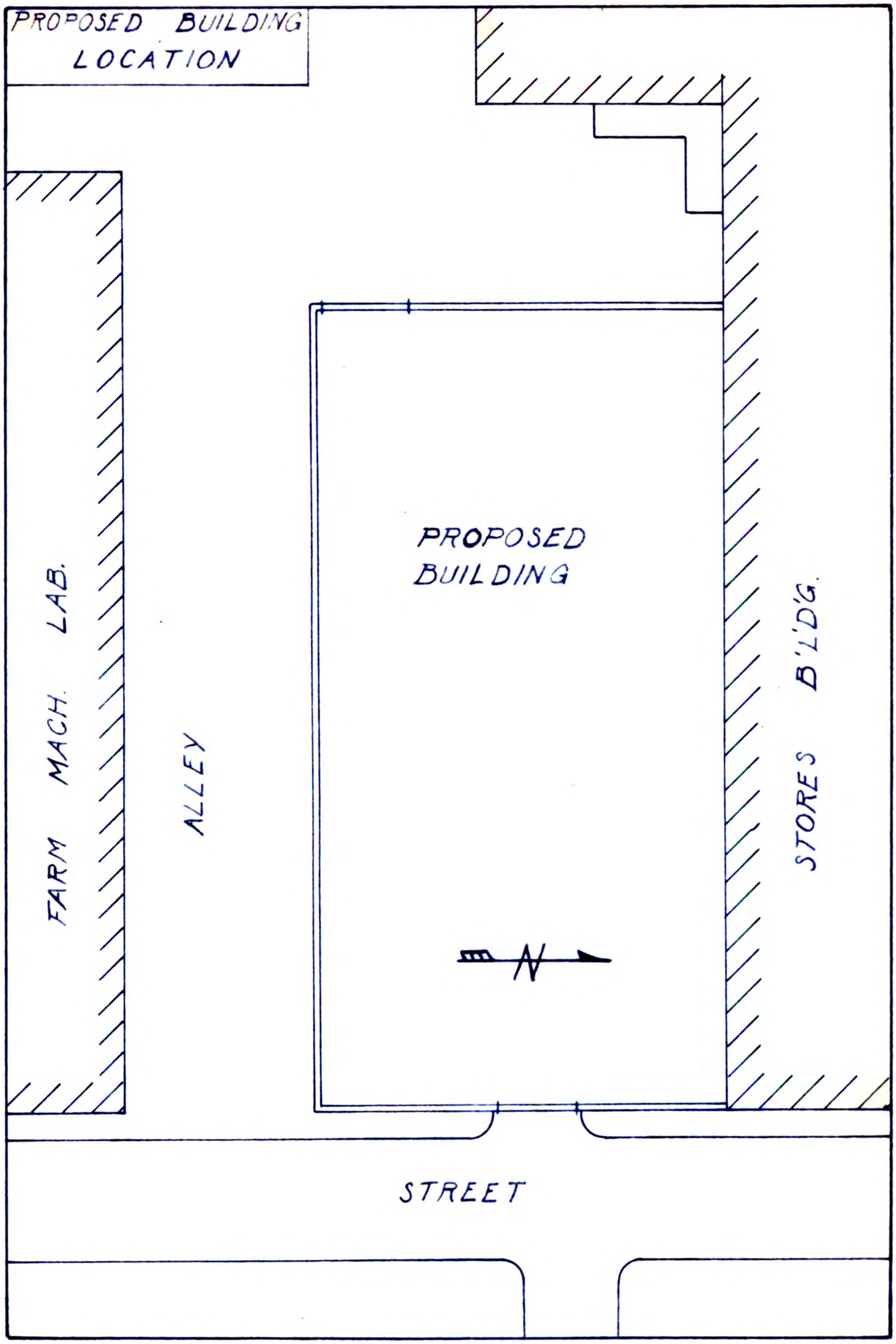
The college operates at East Lansing, including automobiles, trucks, buses and trailers, about 125 vehicles; this figure being liable to increases in the future. Occasionally the college garage is called upon to service as many as 25 of these in one day. The proposed building, with its more than 5,000 sq. ft. of floor space available, would be fully capable of providing the facilities required to cope with this situation.

The location of the proposed building, as shown by the sketch, provides two convenient entrances to the garage and does not in any way impair the functions of the existing buildings in the vicinity. The entrance to the east is directly in line with the exit from the main campus drive, while the entrance to the rear would be used for the necessary dead storage within the building.

The front (east end) of the proposed building is to be two stories high. The ground floor of this portion of the building contains a garage office and a parts storage room. The second story contains three offices with dimensions 19'x17', 19½'x13', and 15½'x13' for use by the Stores Department.

THE SCOPE of the thesis is naturally limited by the problem itself and by the time and space available for its development. All problems of engineering design which will be encountered in the evolution of this building have been adequately dealt with. No attempt has been made to invade the sphere of the architect, however, in computing dead loads certain basic architectural features have been assumed which should be adhered to by the architect. These assumptions, evident from the computations, tend to give the proposed building an architecture similar to that of the existing Stores building to which it is attached. Similarly no space is given to plumbing, heating or wiring.

Included in the thesis are all computations, sketches and drawings necessary to draw up complete architects and mechanics prints and form the bill of materials for the proposed building. Specifications should be made with reference to the unit stresses used in design as stated in the computations.


PROPOSED BUILDING LOCATION
FARM MACH. LAB.
ALLEY
PROPOSED BUILDING
N
STORES B'L'D'G.
STREET

BRIEF EXPLANATION OF COMPUTATIONS

The Lansing Building and Safety Code classifies all buildings according to occupancy and type of construction. The service garage portion of this building is classified as "sub-class B-1", with types 1, 2, 3, 4 or 5 construction allowable. I have chosen type 5 or skeleton construction for the service garage. The second floor portion of the building containing offices comes under "class C" buildings, with types 1, 2, 3, 4 or 6 construction allowable. I have chosen type 3 or protected construction for this portion of the building. Because of the class B and C occupancies in the same building a complete fire separation is necessary, otherwise both parts of the building would be subject to the strictest requirements of either classification. The fire separation is obtained through the design herein presented. All unit stresses and design methods are those prescribed by the L.B.C., A.C.I. Code and A.I.S.C.

TIMBER AND STRUCTURAL STEEL DESIGN

GARAGE ROOF DESIGN:

Using type 5 or skeleton construction for the service garage, unprotected steel and timber members may be employed. Using 5-ply felt, 5 coats of asphalt and 1" sheathing, (From Pulver - "Construction Estimates and Costs") we obtain a dead roof load of 6# per sq. ft. The L.B.C. specifies a live load of 40# per sq. ft. for flat (less than 1/6 pitch) roofs, giving a total of 46# per sq. ft.

Using unit stresses obtained from L.B.C. for common structural fir or pine, timber rafters spaced 18" o.c. and purlins spaced 5' o.c. were designed.

TO SUPPORT ROOF SYSTEM:

Instead of using four 60' trusses, it was decided in the interests of economy to use eight identical 30' trusses. This necessitates a row of columns at the center of the garage, however, the economy of the shorter span will outweigh any inconvenience involved.

The 30' truss is designed with 6 panels @ 5'. Panel loads are computed at 4,740# per panel. Using a horizontal lower chord and an upper chord with 1/6 pitch the truss was analysed for total depths of 4½', 5½' and 6½ ft. The lightest angle used in practice as truss members is the 2" x 2½" x 5/16" angle. With a total depth of truss of 6½ ft., it was found that two 2" x 2½" x 5/16" angles - 3/8" back to back - short legs outstanding could be used for all members throughout truss, therefore this design was used. The methods of analysis used were those of moments, shears, joints and summations of forces in H and V direction. The final design was checked by the methods of graphic statics. Shop details of truss were obtained graphically by drawing each panel to 1/2 actual size and scaling off dimensions. Rivet spacings used are those recommended by A.I.S.C.

TO SUPPORT TRUSS:

Using unit stresses in masonry prescribed by L.B.C. no pilasters are required in the south wall to support truss if made of brick 12" thick with portland cement mortar. For columns #9, #10, #11, #12 to support trusses at center of garage rolled sections were selected from A.I.S.C. tables computed for column formula -

$$f = \frac{18,000}{1 + 1/18000(l/r)^2}$$

as required by L.B.C. for l/r ratio between 60 and 120. At north side of building a cement block pier is used - ratio of height to least dimension being less than 12 and compressive stresses within the L.B.C. requirements. To provide lateral support for trusses and facilitate the addition of a moveable 2000# chainfall, a double angle section was designed to be run from column to column, welded at each end to truss bearing plate; the first and last member to be anchored in 12" wall.

LINTEL DESIGN:

Although architectural details are not included in this thesis it is assumed that window openings will be similar to those of the existing adjacent building, thus at the north and south ends of the building 11 ft. wide window openings will be used and at the south side 13 ft. wide window openings. Garage door openings are 14 ft. wide and office entrance is 3½ ft. wide. Lintels are designed to support a 12" brick wall with surface area shaped as an equilateral triangular with base equal to width of opening. Lintels are composed of "I" beams and angles welded to 12" plates.

JOISTS TO SUPPORT OFFICE ROOF:

The character of the roof slab over the offices necessitates a rolled section or small truss for its support. I am using Bethlehem rolled steel joists spaced 2½ ft. o.c. for this purpose.

BEARING PLATES:

Bearing plate areas are computed from allowable L.B.C. unit compressive stresses for material carrying load. Thicknesses are computed by A.I.S.C. method. Bearing plates were necessitated by bearing of truss on outside wall, bearing of truss on columns #9, #10, #11, #12 and bearing of truss on cement block piers, also at base of central columns and at base and top of column "A". No bearing plates were required for bearing of lintels on 12" wall, bearing of office roof joists on 12" wall, and bearing of Sect. 1 on 12" wall or Sect. 2 on 4" partition.

REINFORCED CONCRETE DESIGN

All unit stresses and requirements for reinforcement bending, placing, anchorage, stirrups spacing and etc. used in the design of reinforced concrete members are those prescribed by the A.C.I. Code as the sections of the

L.B.C. covering reinforced concrete were repealed in 1940 and the A.C.I. Code substituted.

OFFICE ROOF DESIGN:

The office roof was designed as a 2½" concrete slab with expanded metal reinforcement supported by rolled steel joists 2½ ft. o.c. The live load of 40# per sq. ft. as given in the L.B.C. was used.

OFFICE FLOOR DESIGN:

The L.B.C. recommended live load of 50# per sq. ft. for office space was used in the design of this floor system. The design was based on a location of partitions as shown on sheet 2 instead of increasing the unit design load for partitions as is sometimes done. The greater part of the slab is composed of a divided joist slab using tapered metal pans. Between beams #4 and #5 a one way flat slab is used.

REINFORCED CONCRETE BEAMS, GIRDERS AND JOISTS:

Total loads supported are computed with no reductions of live load and moments are figured by method of moment coefficients. Spans are measured between faces of supporting members. Actual design was accomplished by the methods of "Sutherland & Reese" using A.C.I. specifications.

STAIR DESIGN:

The two flight stairs with 7½" riser and 10" tread meets the class C building requirements of one separate exit from the office space. Stairs are designed for 100# per sq. ft. of horizontal span with a one way flat slab landing. Rolled sections #1 and #2, column "A" and beam #5 are designed to support stairs and landing.

REINFORCED CONCRETE COLUMNS:

Loads on columns supporting continuous girders were computed from the three moment theorem and then increased for ratio of "height of column" to "least dimension" greater than 10, by A.C.I. formula. Rectangular tied columns are used throughout. Reinforcement and lateral ties are computed by A.C.I. methods and formulas.

FOOTINGS:

Due to proximity of proposed building to existing Stores building, footings #1 and #4 were designed as rectangular beams with concentrated load and uniformly distributed soil reaction. Spread footings were designed by use of table 35 of R.C.D.H. Computations show that no reinforcement was required to resist bending or shearing stresses for wall footings and footing "A", therefore plain concrete with temperature steel only is used.

Dowels were used to transfer stress of longitudinal reinforcement to footings and in some cases pedestals were required.

FOUNDATION WALL:

Foundation walls were designed to bring the wall footings below the frost line. The weight supported by foundation wall produced unit stresses which required only the minimum A.C.I. reinforcement which was provided.

GARAGE FLOOR:

Garage floor was designed by the methods set forth in the Portland Cement Association bulletin, ST 51, "Concrete Floors On Ground". Assuming maximum wheel loads of 5000# a 6" slab is used with welded mesh reinforcement and dowels, expansion and contraction joints provided where necessary.

COMPUTATIONS

DESIGN OF STRUCTURAL STEEL
AND TIMBER MEMBERS

Allowable Unit Stresses From Lansing Building And Safety Code.

TIMBER

Common Structural Fir or Pine

Stress in extreme fibre in bending.........1400 P.S.I.

Horizontal shearing stress................. 125 P.S.I.

Compression parallel to grain..............1000 P.S.I.

Compression perpendicular to grain......... 250 P.S.I.

Extreme bearing stress.....................1700 P.S.I.

STRUCTURAL STEEL

Extreme fibre stress in bending............18,000 P.S.I.

Shearing stress............................12,000 P.S.I.

Compressive stress.........................15,000 P.S.I.

Rivet Bearing

Single shear...........................24,000 P.S.I.

Double shear...........................30,000 P.S.I.

RIVETS

Extreme fibre stress in bending............18,000 P.S.I.

Shearing stress............................13,500 P.S.I.

Rivet bearing

Single shear...........................24,000 P.S.I.

Double shear...........................30,000 P.S.I.

Standard notation is used throughout this thesis except that for any joist, beam, girder, rafter, purlin, lintel or othe horizontal member supporting a uniformly distributed verticle load:

W_1 = Total superimposed load in # per. lin. ft.

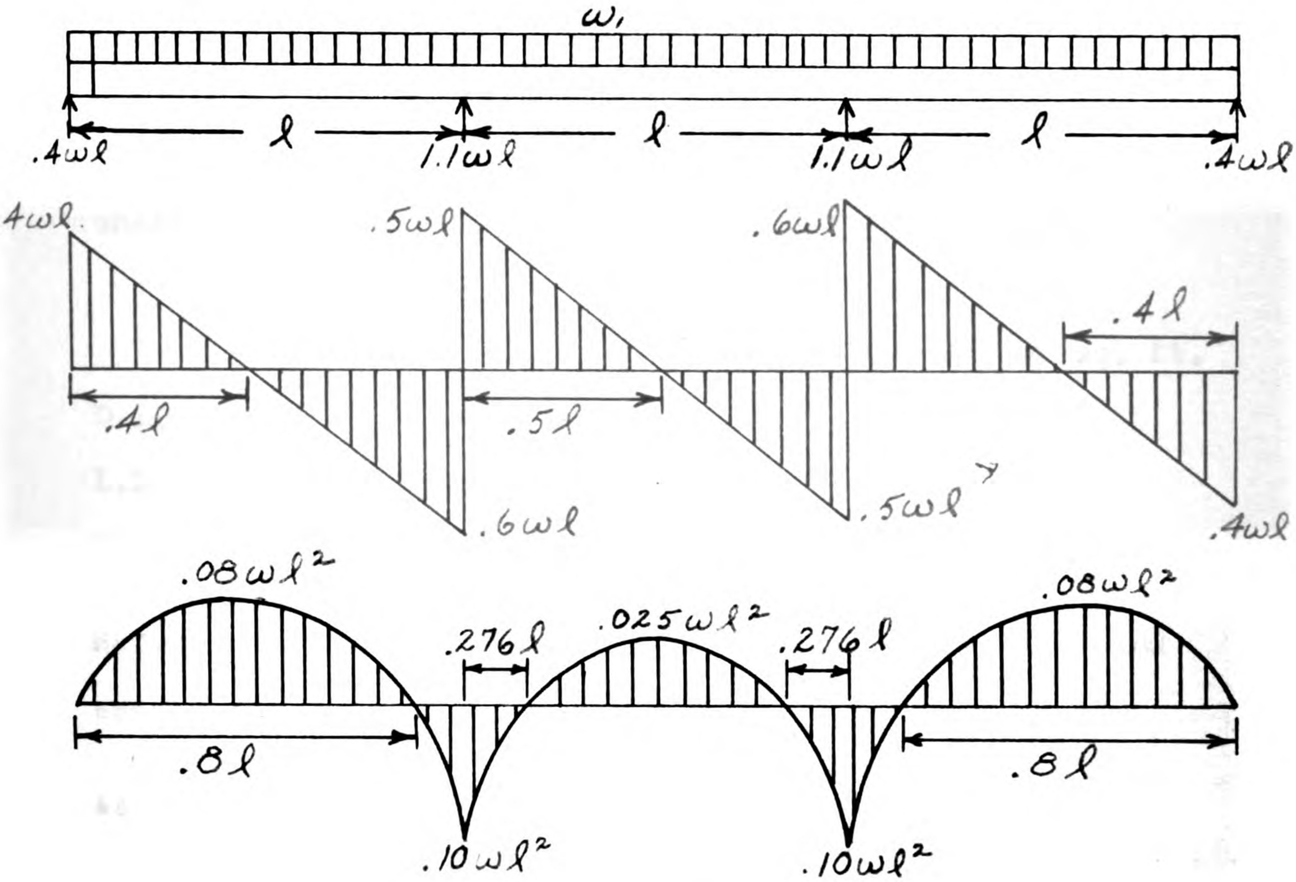
W_2 = Total weight of member in # per lin. ft.

$W = W_1$ plus W_2

SAFE LOADS FOR MASONRY

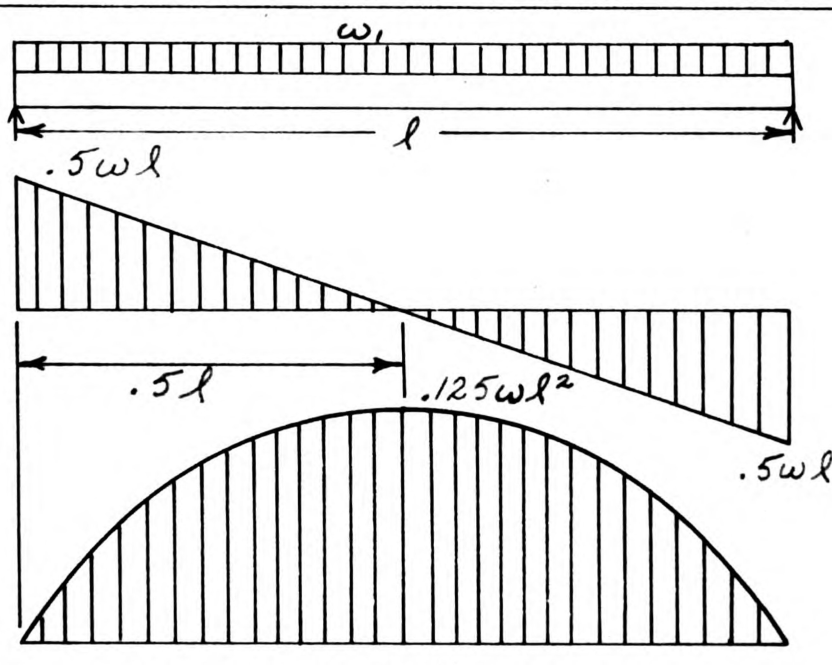
From - Lansing Building & Safety Code

Lime and Portland Cement Mortar	Compression In Bulk of Masonry	Bearing Pressure for concentrated loads.
Common Brickwork	150 P.S.I.	175 P.S.I.
Clam Tile Load on Gross Area	90 P.S.I.	115 P.S.I.
Concrete Blocks Load on Gross Area	90 P.S.I.	115 P.S.I.



ω,
l
l
l
.4wl
1.1wl
1.1wl
.4wl
.4wl
.5wl
.6wl
.4l
.4l
.5l
.6wl
.5wl
.4wl
.08wl²
.025wl²
.08wl²
.276l
.276l
.8l
.8l
.10wl²
.10wl²

FIG. #1



ω,
l
.5wl
.5l
.125wl²
.5wl

FIG. #2

RAFTER DESIGN

5 ply felt at 30# per sq.	=	150# per sq.
5 coats asphalt @ 25# per sq.	=	125# per sq.
sheating	=	300# per sq.
Total	=	575# per sq.
		Say 6# per sq. ft.

D.L. = 6# per sq. ft.

L.L. = 40# " " "

Total roof load = 46# " " "

Rafters 15 ft. long supported by purlins every 5 ft. and spaced 18" o.c.

Assume 2" x 6" nominal size

1-5/8" x 5-5/8" dressed size

$W_1 = 46 \times 1.5 = 69\#/ft.$

$W_2 = 2.5\#/ft.$

$W = 71.5\#/ft.$

Max. Mom. $= .10\ wl^2$ (See Fig. 1)

$= .1 \times 71.5 \times 5^2$

$= 179'\#$ or $2150''\#$

Allowable Mom. = fS

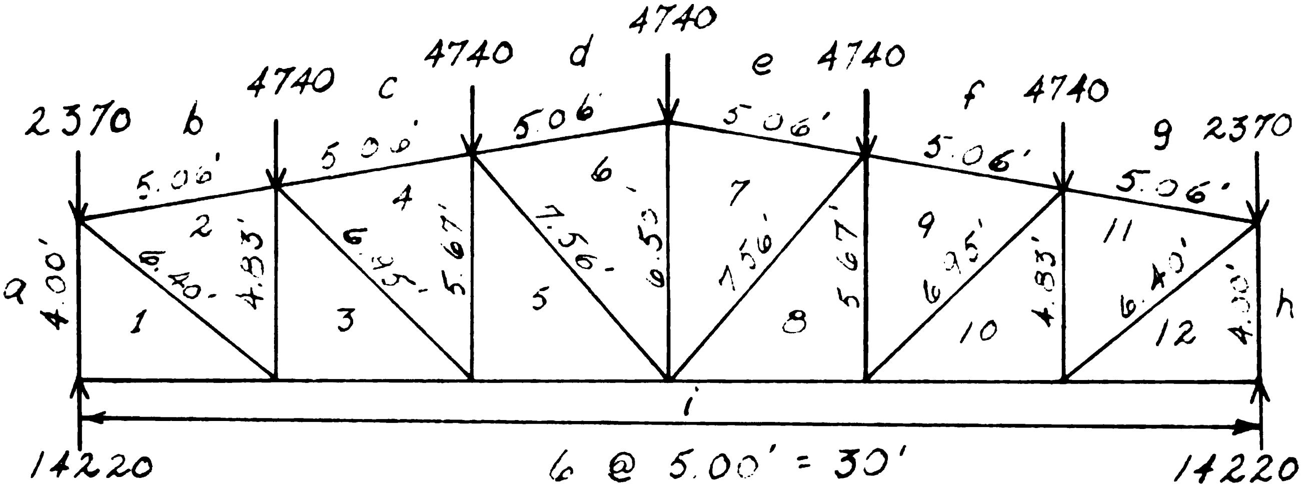
$$S = \frac{I}{c} = \frac{24.10}{2.81} = 8.57''^3$$

$fS = 1200 \times 8.57 = 10{,}300''\#$ (O.K. - overdesigned for rigidity).

PURLIN DESIGN

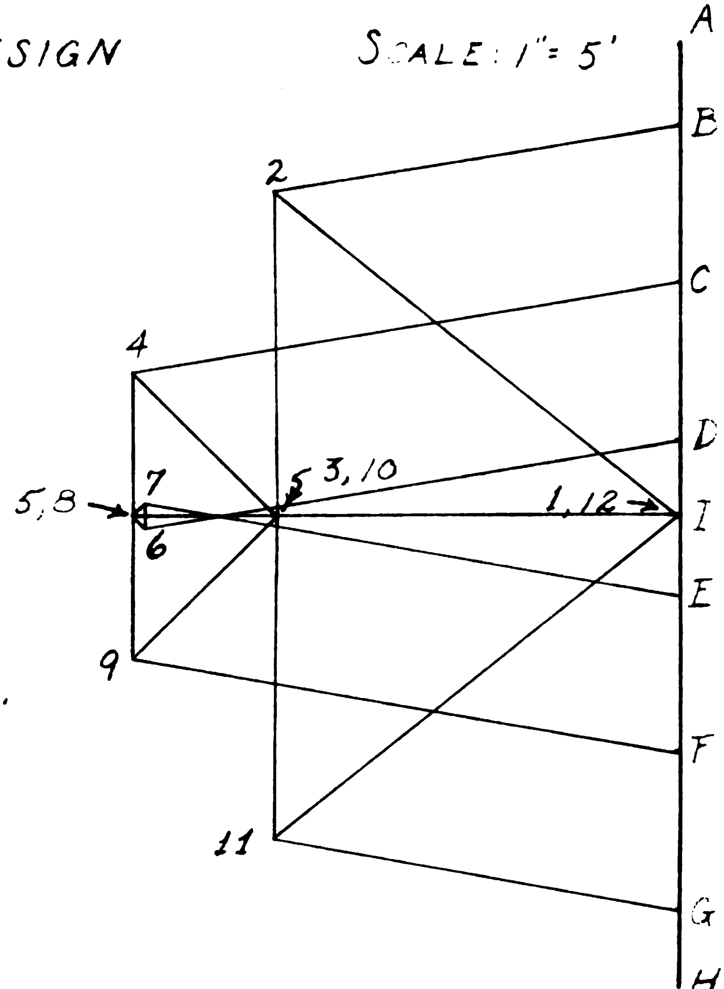
Purlins 18' long - simply supported - spaced 5 ft. o.c.

Use 4" x 14" timbers - dressed size - 3-5/8" x 13-1/2"



THEORETICAL TRUSS

FIG. # 3 DESIGN SCALE: 1" = 5'



GRAPHIC

FIG. #4 ANALYSIS SCALE: 1" = 6,000#

$W_1 = 71.5 \times 5 \times 12/18 = 238\#$ per ft.

$W_2 \qquad = 13\#$ per ft.

$W \qquad = 251.0\#$ per ft.

Max. Mom. $= \frac{wl^2}{8}$ (See Fig. 2) $= \frac{251 \times 18^2}{8} = 10{,}150'\#$

or $122{,}000''\#$

Allowable Mom. = Sf

$$S = \frac{I}{c} = \frac{743.23}{6.75} = 110.11$$

$$fS = 110.11 \times 1200 = 132{,}000''\# \text{ (O.K.)}$$

Check for horizontal shear:

$$v = \frac{3V}{2A} = \frac{251 \times 18 \times 3}{4 \times 48.93} = 69.6 \text{ P.S.I.} \quad \text{(OK)}$$

<u>LOADING OF TRUSS</u>

Wt. Roof per panel = $251 \times 18 = 4520\#$

Wt. Truss per panel (assume) = $220\#$

Total panel load = $4{,}740\#$

<u>ANALYSIS OF TRUSS</u>

See Fig. 3 for theoretical truss proportions.

Taking free body - Fig. 5

$$\sin \theta = \frac{4}{4.60}$$

$$d = 29 \times \frac{4}{4.60} = 18.13'$$

Summation $M_o = 0$

$$-11{,}850 \times 24 + X(18.13) = 0$$

$$X = 15{,}690$$

Stress In (1-2) = <u>15,690#T</u>

Taking free body - Fig. 6

$$\sin \theta = \frac{4.83}{6.95}$$

$$d = 34 \times \frac{4.83}{6.95} = 23.61$$

Summation $M_o = 0$

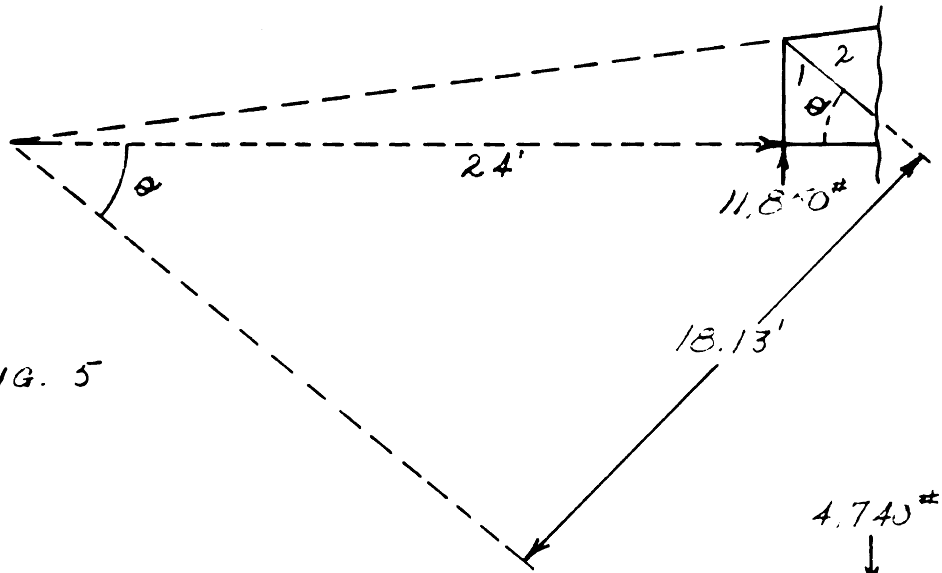


FIG. 5

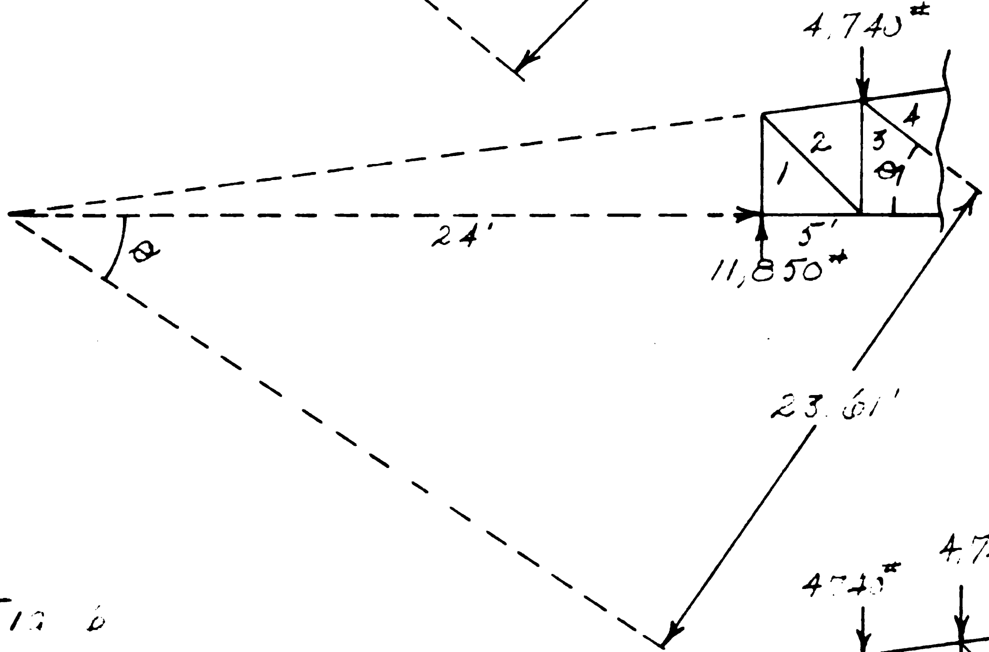


FIG. 6

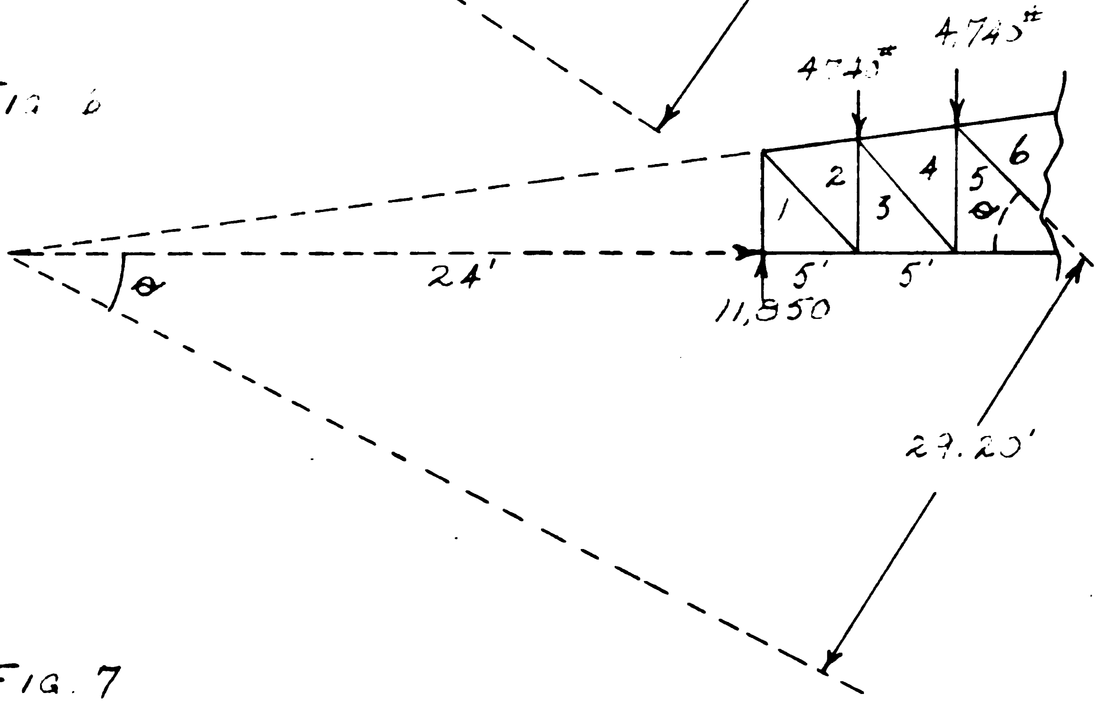


FIG. 7

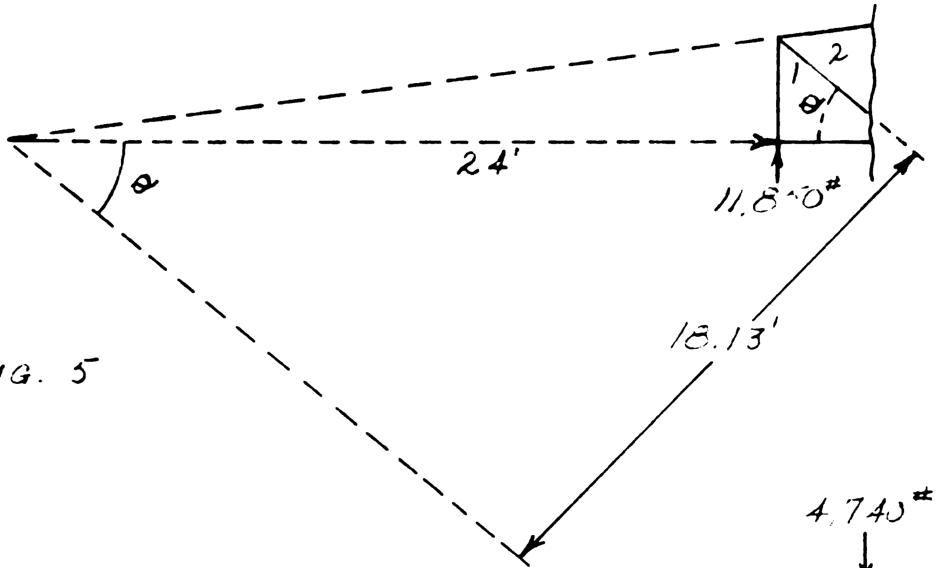


FIG. 5

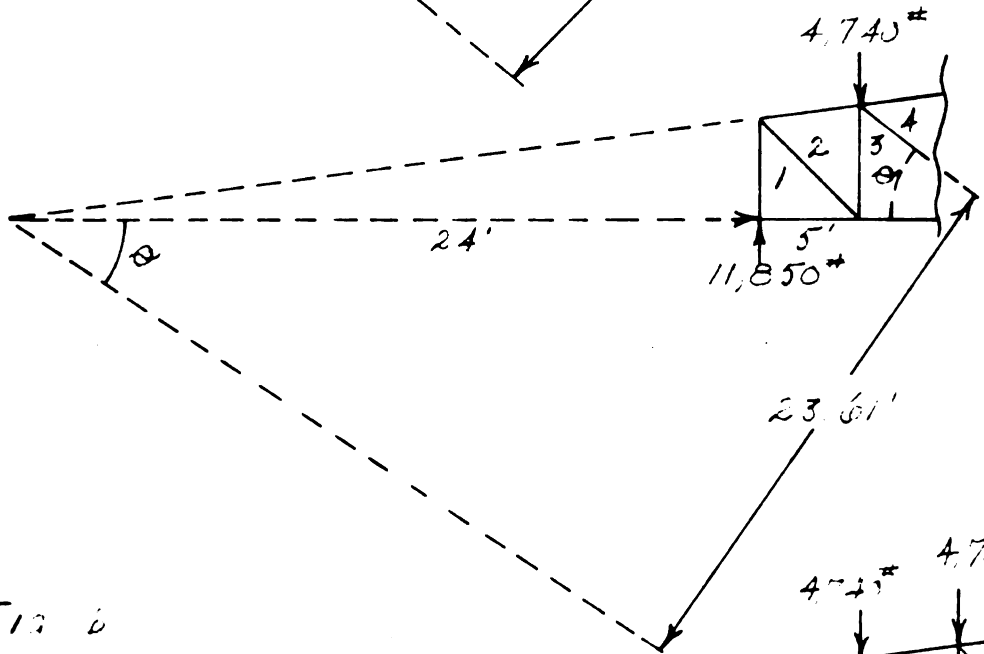


FIG. 6

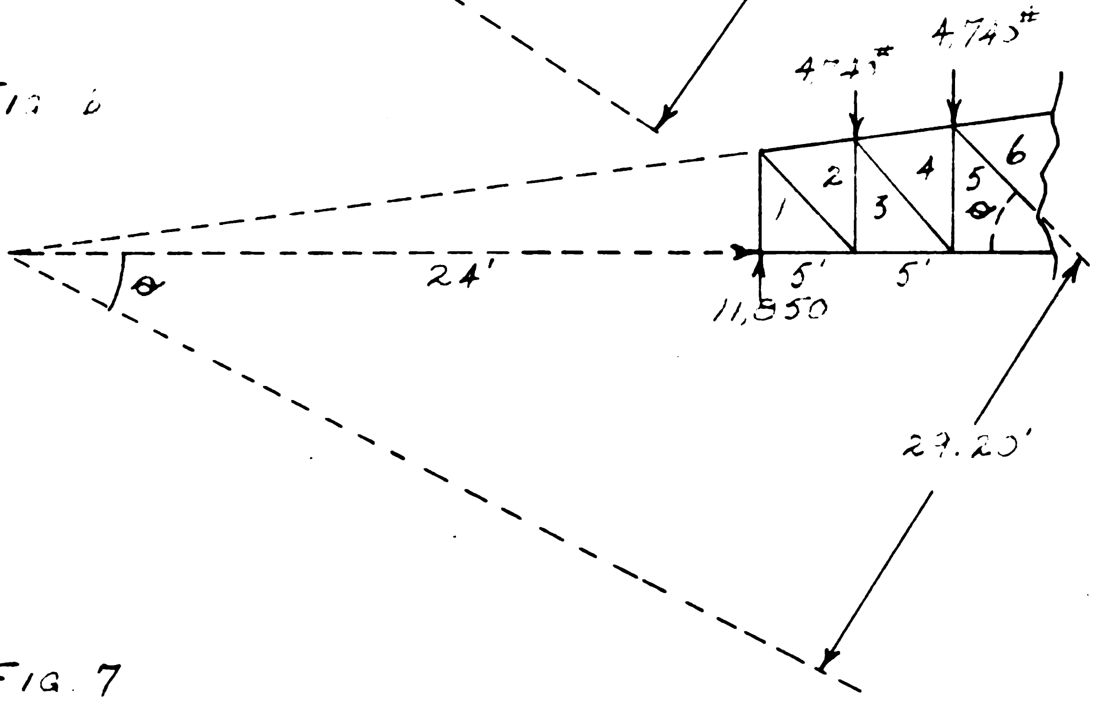


FIG. 7

$$-11{,}850\ (24) + 4740\ (29) + X(23.61) = 0$$

$$X = 6{,}220$$

Stress In (3-4) = <u>6,220#T</u>

Taking free body - Fig. 7

$$\sin\theta = \frac{5.67}{7.56}$$

$$d = 39 \times \frac{5.67}{7.56} = 29.20$$

Summation $M_o = 0$

$$-11{,}850(24) + 4740(29) + 4740(34) + X(29.2) = 0$$

$$X = -479$$

Stress In (5-6) = <u>479#C</u>

Taking Free Body - Fig. 8

Summation H = 0

$$(1\text{-}2)_H = 15690\ \left(\frac{5}{6.40}\right) = 12{,}250 = (b\text{-}2)_H$$

$$(b\text{-}2) = 12{,}250\ \left(\frac{5.06}{5}\right) = 12{,}400$$

Stress In (b-2) = <u>12,400#C</u>

Taking Free Body - Fig. 9

Summation V = 0

$$(2\text{-}3) + (b\text{-}2)_v - 11850 = 0$$

$$(b\text{-}2)_v = 12{,}400\ \left(\frac{.83}{5.06}\right) = 2{,}030$$

$$(2\text{-}3) = 9{,}820$$

Stress In (2-3) = <u>9,820#C</u>

Taking Free Body - Fig. 10

Summation H = 0

$$(b\text{-}2)_h - (c\text{-}4)_h + (3\text{-}4)_h = 0$$

$$12{,}250 + 6220\left(\frac{5}{6.95}\right) = (c\text{-}4)_h$$

$$(c\text{-}4)_h = 16720$$

$$(c\text{-}4) = 16{,}720\ \left(\frac{5.06}{5}\right) = 16{,}900$$

Stress In (c-4) = <u>16,900#C</u>

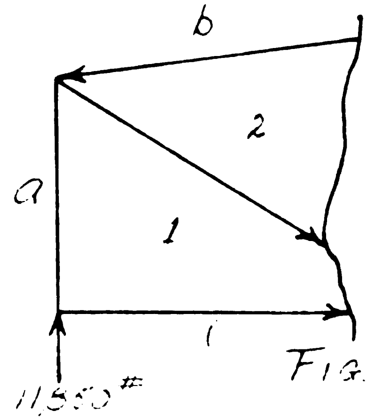


FIG. 8

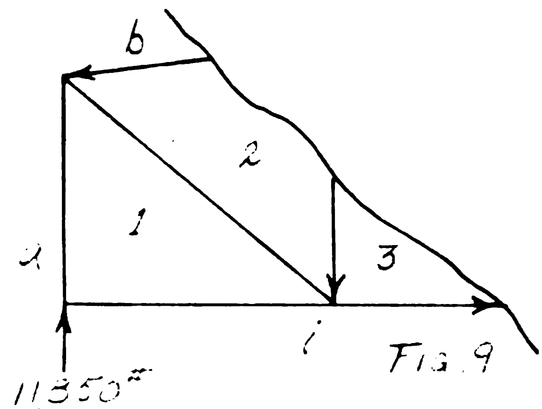


FIG. 9

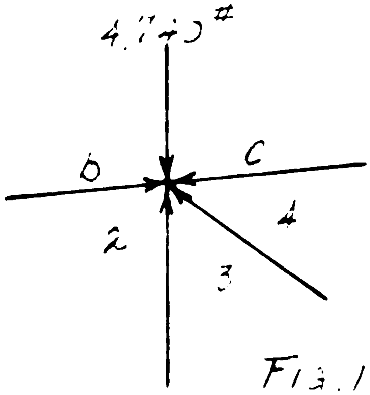


FIG. 10

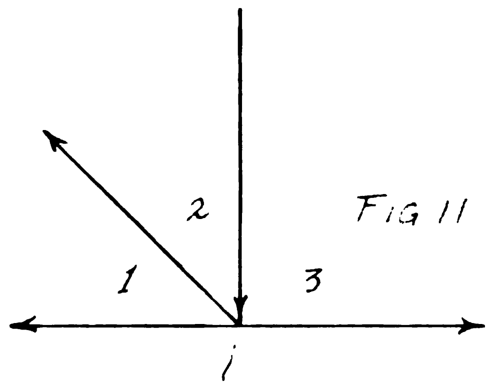


FIG. 11

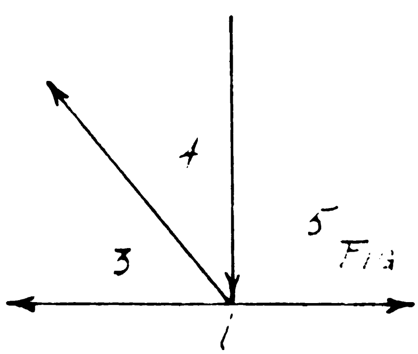


FIG. 12

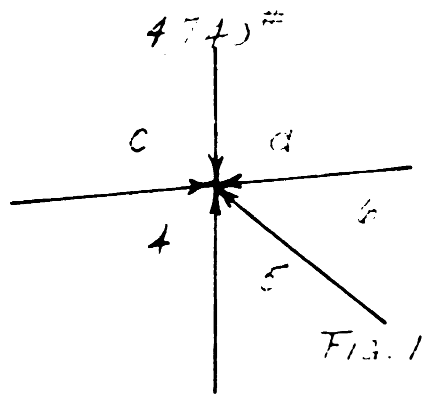


FIG. 13

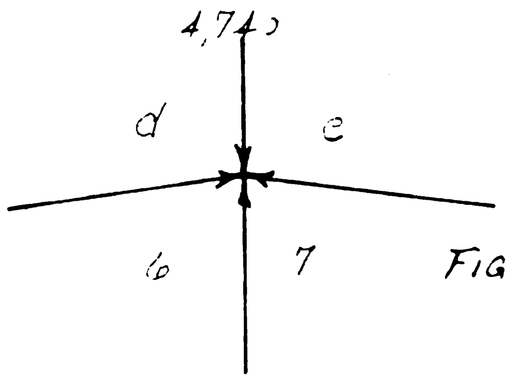


FIG. 14

Taking Free Body - Fig. 11

Summation H = 0

$$(3\text{-}1) = (1\text{-}2)_h = 12{,}250$$

Stress In (3-1) = <u>12,250#T</u>

Taking Free Body - Fig. 13

Summation H = 0

$$(c\text{-}4)_h - (5\text{-}6)_h = (d\text{-}6)_h$$

$$16{,}720 - 479\left(\frac{5}{7.56}\right) = (d\text{-}6)_h$$

$$(d\text{-}6)_h = 16{,}400$$

$$(d\text{-}6) = 16{,}400\left(\frac{5.066}{5}\right) = 16{,}620$$

Stress In (d-6) = <u>16,620#C</u>

Taking Free Body - Fig. 12

Summation V = 0

$$(4\text{-}5) = (3\text{-}4)_v$$

$$6220\left(\frac{4.83}{6.95}\right) = 4330$$

Stress In 4-5 = <u>4330#C</u>

Taking Free Body - Fig. 12

Summation H = 0

$$(5\text{-}1) = (3\text{-}1) + (3\text{-}4)_h$$

$$(5\text{-}1) = 12250 + 4470 = 16{,}720$$

Stress In (5-1) = <u>16,720#T</u>

Taking Free Body - Fig. 14

Summation V = 0

$$(6\text{-}7) = -2(d\text{-}6)_v + 4{,}740$$

$$(6\text{-}7) = -2\left(16620 \times \frac{.83}{5.06}\right) + 4740 = -710$$

Stress In (6-7) = <u>710#T</u>

Taking Free Body - Fig. 3

At joint a-1-1-a

Summation H = 0

Stress In (a-1) = 14,220#C

Summation V = 0

Stress In (1-i) = 0#

TRUSS ANALYSIS

STRESS SUMMARY

Member	Stress
a-1	14,220#C
b-2	12,400#C
c-4	16,900#C
d-6	16,620#C
i-1	0#
i-3	12,250#T
i-5	16,720#T
1-2	15,690#T
3-4	6,220#T
5-6	479#C
2-3	9,820#C
4-5	4,330#C
6-7	710#T

Selection of Members for Truss

Maximum tension = 16,720# (Member i-5)

Req'd. Area = $\frac{16720}{18000}$ = .93 Sq. In. effective

A.I.S.C. effective area in tension = Back - Rivet + 1/2 outstanding leg.

Using 2 - $2\frac{1}{2}$" x 2" x 5/16" angles back to back with short legs outstanding

Eff. Area = 1.31 - (5/16 x 7/8) - 1-11/16 x 1/2 x 5/16

= .73

.73 x 2 = 1.46 Sq. In. OK

Compression Members

For l/r Ratio greater than 60, L.B.C. requires use of column formula, $f = \frac{18000}{1 + 1/18000\ (l/r)^2}$

From A.I.S.C. Table,

2 angles - 2½" x 2" x 5/16" - 3/8" back to back will withstand compressive force of from 29,000# to 39,000# for unsupported lengths of from 4 to 7 ft. l/r ratio in all cases less than 120, complying with L.B.C.

Therefore 2 - 2½" x 2" x 5/16" angles - 3/8" back to back with short legs outstanding will withstand maximum tensile and compressive forces in truss, and being the minimum size angle which will take a 3/4" rivet.

This angle will be used throughout.

Rivets Required

Use 3/4" Shop rivets.

One rivet in shear (double)

$$\frac{2(Pi)d^2}{4} \times f = 2\ \frac{3.1416(.75)^2}{4} \times 13,500 = 11,920\#$$

One rivet in bearing (single)

3/8 x 3/4 x 30,000 = 8,440#

Bearing stress will govern.

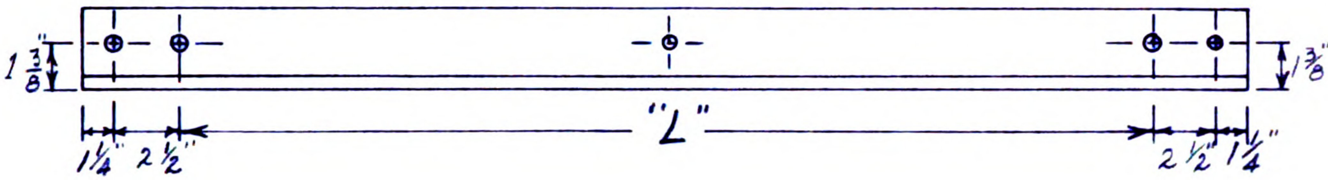
Member	Stress / Allowable bearing per rivet	Rivets Req'd.
a-1	$\frac{14,220}{8,440}$	2

Member	Stress / Allowable Bearing per Rivet	Rivets Req'd.
b-2	12,400 / 8,440	2
c-4	16,900 / 8,440	2
d-6	16,620 / 8,440	2
i-1	0 / 8,440	2
i-3	12,250 / 8,440	2
i-5	16,720 / 8,440	2
1-2	15,690 / 8,440	2
3-4	6,220 / 8,440	2
5-6	479 / 8,440	2
2-3	9,820 / 8,440	2
4-5	4,330 / 8,440	2
6-7	710 / 8,440	2

For shop details of truss members see Figs. 15, 16 and 17. Dimensions were obtained by laying out truss to 1/2 full scale and scaling off. Rivet spacing is in compliance with L.B.C. and A.I.S.C. Upper and lower chord members are to be fabricated as shown in Figs. 16 and 17.

Values of "L" for web members (Fig. 15) are as follows:

FIG. 15 - WEB MEMBERS - 2-2½×2×5/16 Ls



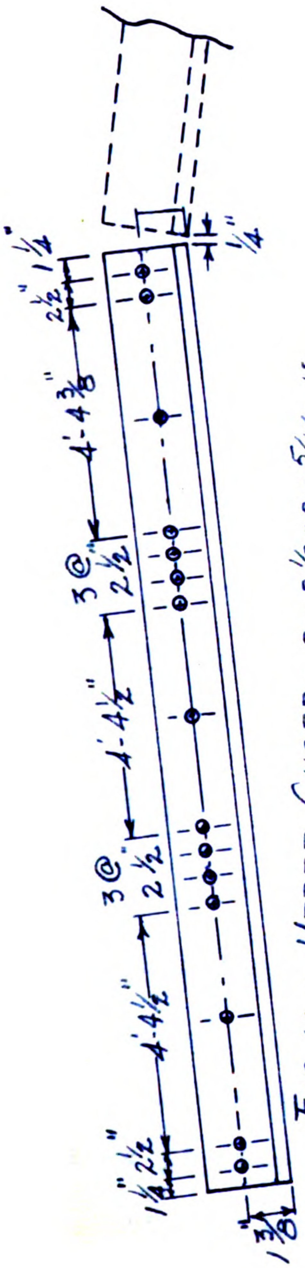


FIG. 16 - UPPER CHORD - 2-2½×2×5/16 Ls

NOTE:- DOUBLE Ls BACK TO BACK
SHORT LEGS OUTSTANDING

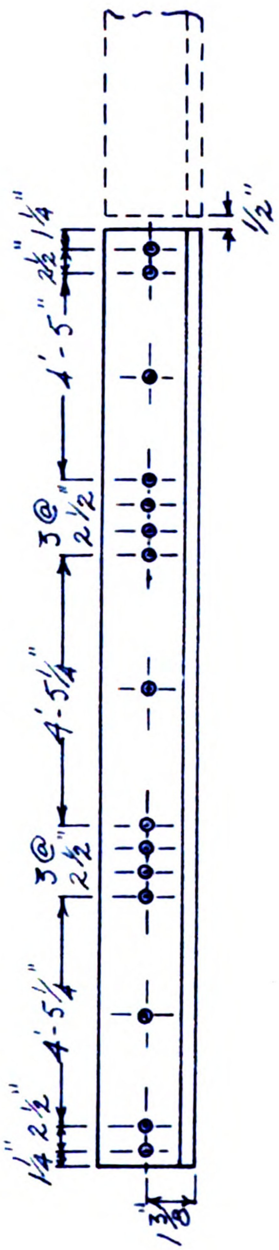


FIG. 17 - LOWER CHORD - 2-2½×2×5/16 Ls

Member	"L" in Inches
a-1	34-3/4
2-3	44-3/4
4-5	54-3/4
6-7	64-3/4
1-2	60-15/16
3-4	67-11/16
5-6	74-7/16

Check Wt. of Truss

Wt. $2\frac{1}{2}$" x 2" x 5/16" angles = 4.5# Per Ft.

5.06 x 6 =	30.36	6.40
5 x 6 =	30.00	6.95
5.25 x 7 =	36.75	7.56
	97.11	20.91
		2
		41.82
		97.11

Lineal Ft. of Double Angle = 139.93 Ft.

139.93 x 4.5 x 2 = 1260 # Wt. Angles
240# Wt. G.P. (Approximately)
1500 # Wt. Truss

$\frac{1500}{6}$ = 257# Per. Panel. OK

(Assumed 220# per panel)

Truss Bearing Plate in Wall

From L.B.C. - Allowable compressive stress under concentrated loads in common brickwork = 175 P.S.I.

Load = 14,220#

$\frac{14,220}{175}$ = 81.3 Sq. In. Req'd.

Use 8" x 12" plates.

8 x 12 = 96 sq. in.

$\frac{14,220}{96}$ = 148 P.S.I. - This is less than 150 P.S.I. in bulk of masonry - complying with L.B.C. and making pilasters unnecessary.

Bearing Plate Thickness (A.I.S.C.)

$$t^2 = \frac{pr^2}{6666} = \frac{148\ (16)}{6666} = .355$$

$$t = .6'' \text{ - Say } 5/8''$$

Use 8" x 12" x 5/8" Bearing Plate In Wall.

Truss Bearing Plates On Piers #13, #14, #15, #16.

From L.B.C. - Allowable compressive stress in cement block masonry = 115 P.S.I.

Required Area = $\frac{14,220}{115}$ = 124 sq. in.

Use 12" x 12" Plates.

$$p = \frac{14220}{144} = 98.7 \text{ P.S.I.}$$

$$t^2 = \frac{98.7(6)}{6666} = .53$$

$$t = .72''$$

Use 12" x 12" x 3/4" Bearing Plate on Piers.

Piers #13, #14, #15, #16 - 2' x 2' x 15' high - made of concrete blocks, ratio of height to least dimension must be less than 12 (L.B.C.)

$$\frac{15}{2} = \underline{7.5\ \ \text{OK}}$$

Columns #9, #10, #11, #12 - See Fig. 18.

Axial Load = 28,440# + 4,000 = 32,440#

Unsupported Height = 15 ft.

For $\frac{l}{r}$ ratio less than 120 as stipulated by L.B.C. we reduce

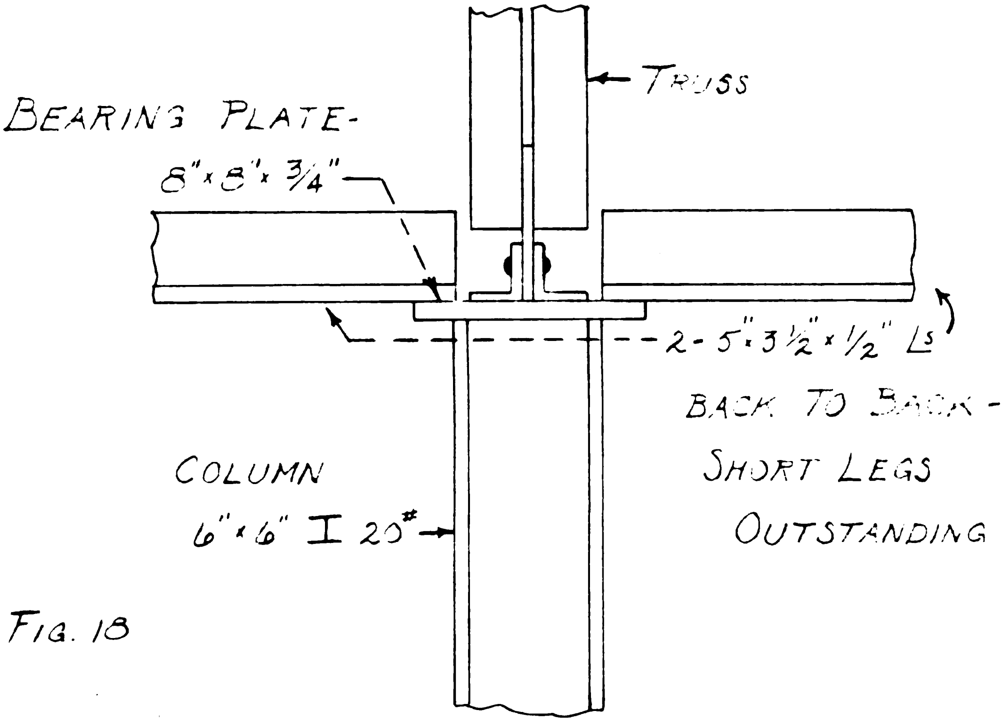


FIG. 18

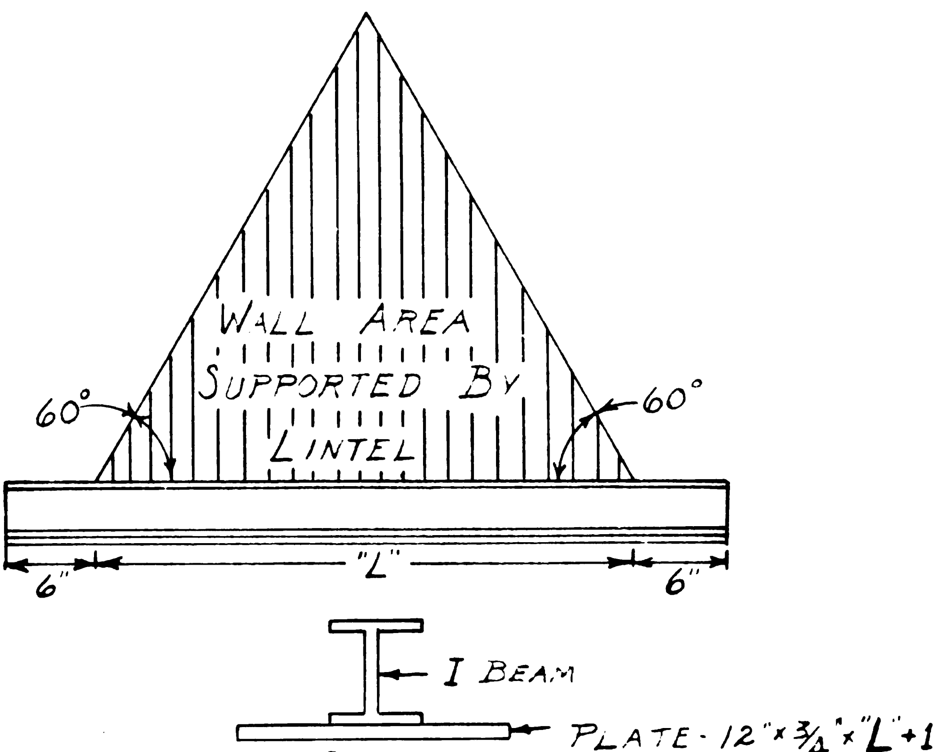


FIG. 19 LINTEL CROSS-SECTION

"f" to $f = \frac{18000}{1 + 1/18000\ (l/r)^2}$

From A.I.S.C. Tables A 6" x 6"H 20# column is required.

Footing Bearing Plates -

$f^1_c = 2000$

$\frac{32,440}{2000} = 16.22$ sq. in. req'd.

Use 8" x 8" x 3/4" plates welded to top and bottom of columns.

To support truss laterally and to facilitate the addition of movable chainfall, angle sections will be run from column to column and welded at each end to bearing plate. The first and last member to be anchored in the wall. See Fig. 18. These angles to support a maximum concentrated load of 2000#.

Max. Mom. = 2000 x 9 - 1000 x 9 = 9000'# or 108,000"#.

$S = \frac{M}{f} = \frac{108000}{18,000} = 6.0''^3$

Use 2 - 5" x $3\frac{1}{2}$" x $\frac{1}{2}$" angles back to back short legs outstanding.

Check shear -

$\frac{1000}{\frac{1}{2} \times 5 \times 2} = 200$ P.S.I. <u>OK</u>

<u>LINTEL DESIGN</u>

Wt. brickwork = 125# per cubic ft.

Wall = 13" thick "L" = 11 ft. (see fig. 19).

Area supported = 5.5 x 5.5 x $\sqrt{3}$ = 52.5 sq. ft.

52.5 x 13/12 x 125 = 7,100#

$M_o = \frac{WL}{6} = \frac{7100 \times 11}{6} =$ 13,000'# or 156,000"#

$$S = \frac{M}{f} = \frac{156000}{18,000} = 8.67''^3$$

Use 6" I 17.25# welded to 12" x ½" x 12' Plate.

Check shear

$$V = \frac{7,100}{2} = 3550 \quad A = 5.02$$

$$v = \frac{3550}{5.02} = 708 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Check bearing

$$\frac{3550}{6x12} = 59.2 \text{ P.S.I.} \quad \underline{\text{OK}}$$

(Allowable 175 P.S.I. - L.B.C.)

Lintel for "L" = 13' (Fig. 19)

Area supported = $6.5 \times 6.5 \times \sqrt{3} = 73.2$ sq. ft.

$73.2 \times 13/12 \times 125 = 9,900\#$

$$M = \frac{W''L''}{6} = \frac{9900 \times 13}{6} = 21,450'\# \text{ or } 258,000''\#$$

$$S = \frac{M}{f} = \frac{258000}{18000} = 14.3$$

Use 8" I 18.4# welded to 12" x ½" x 14' Plate.

Check Shear -

$$V = \frac{9900}{2} = 4,950 \qquad A = 5.34$$

$$v = \frac{4950}{5.34} = 934 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Check Bearing -

$$\frac{4950}{10x6} = 82.5 \text{ P.S.I.} \quad \underline{\text{OK}}$$

(175 P.S.I. Allowable - L.B.C.)

Lintel for "L" = 14' (Fig. 19)

Area supported = $7 \times 7 \times \sqrt{3} = 84.7$ sq. ft.

$84.7 \times 13/12 \times 125 = 11,450\#$

$$M = \frac{W''L''}{6} = \frac{11,450(14)}{6} = 26,800'\# \text{ or } 321,000''\#$$

$$S = \frac{M}{f} = \frac{321000}{18000} = 17.8''^3$$

Use 8" WF 21# welded to 12" x ½" x 15' Plate.

Check Shear -

$$V = \frac{11450}{2} = 5{,}725 \qquad A = 6.18$$

$$v = \frac{5725}{6.18} = 925 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Check Bearing -

$$\frac{5725}{10x6} = 95.4 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Joists to Support Office Roof (see fig. 20)

Spaced $2\frac{1}{2}'$ o.c. Span = 18'

$W_1 = 71.2 \times 2\frac{1}{2} = 178\#$ per ft.

$W_2 \qquad = \quad 8.5\#$ per ft. (assume)

$W \qquad = 186.5\#$ per ft.

$$M = \frac{wl^2}{8} = \frac{186.5(18)^2}{8} = 7{,}560'\# \text{ or } 90{,}700''\#$$

$$S = \frac{M}{f} = \frac{90{,}700}{18{,}000} = 5.04''^3$$

Use 6" x 4" - 8.5# per ft. Bethlehem Rolled Steel Joists.

Check Shear -

$$V = \frac{18 \times 186.5}{2} = 1678\# \qquad A = 2.50 \text{ Sq. In.}$$

$$v = \frac{1678}{2.50} = 670 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Check Bearing -

Load = 1678# Flange Width = 4"

$$\frac{1678}{4x6} = 70 \text{ P.S.I.} \quad \underline{\text{OK}}$$

(175 P.S.I. Allowable - L.B.C.)

REINFORCED CONCRETE DESIGN

A.C.I. ALLOWABLE UNIT STRESSES

Beams, Girders, Columns, Footings, Walls :-

$f'_c = 2500$ P.S.I. $n = 12$ $f_c = 1125$ P.S.I.

$f_s = 20{,}000$ P.S.I. $v_c = 50$ P.S.I. $u = 125$ P.S.I.

Stairs and office floor slab :-

$f'_c = 2000$ P.S.I. $n = 15$ $f_c = 900$ P.S.I.

$f_s = 20{,}000$ P.S.I. $v_c = 40$ P.S.I. $u = 100$

Garage Floor Slab :-

$f'_c = 5000$ P.S.I. $f_s = 25{,}000$ P.S.I.

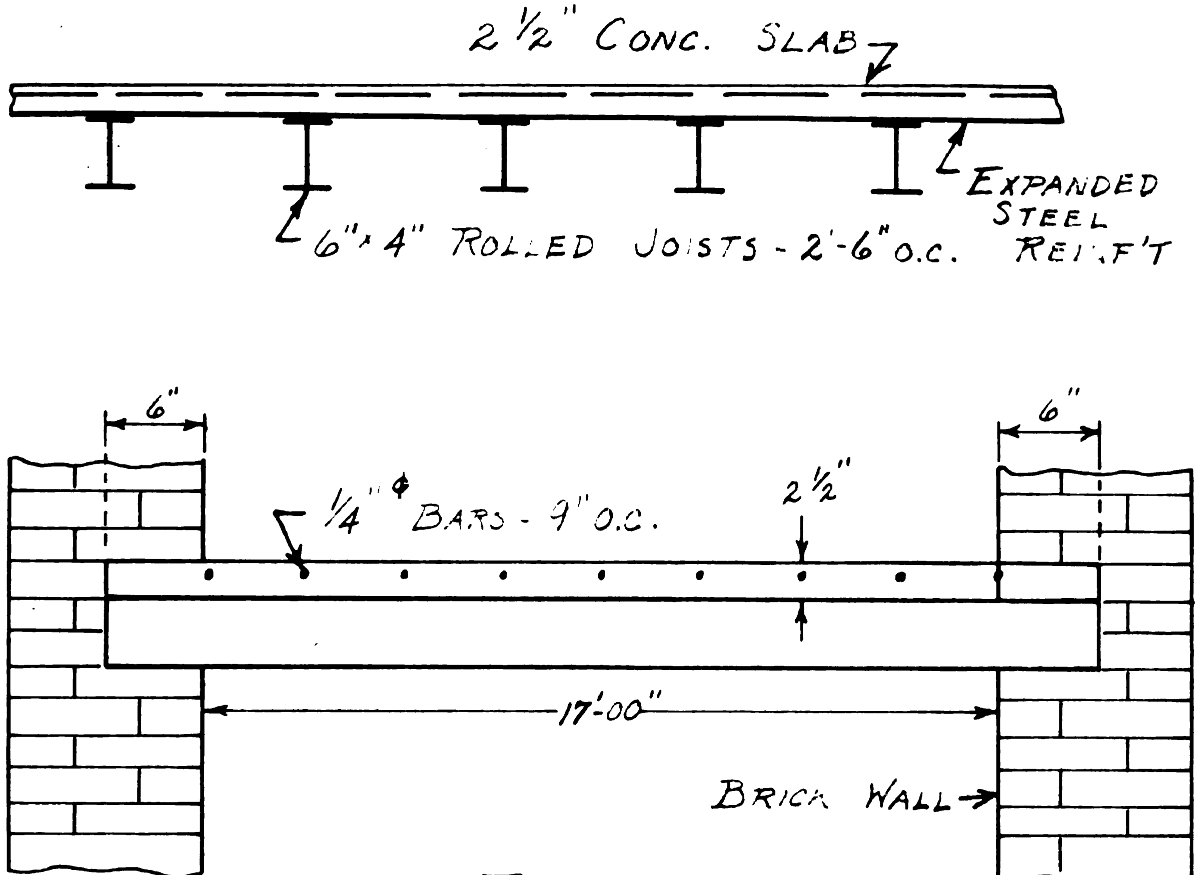

2 1/2" CONC. SLAB
EXPANDED STEEL REIN.F'T
6" x 4" ROLLED JOISTS - 2'-6" O.C.
6"
6"
1/4" ϕ BARS - 9" O.C.
2 1/2"
17'-00"
BRICK WALL

FIG. 20

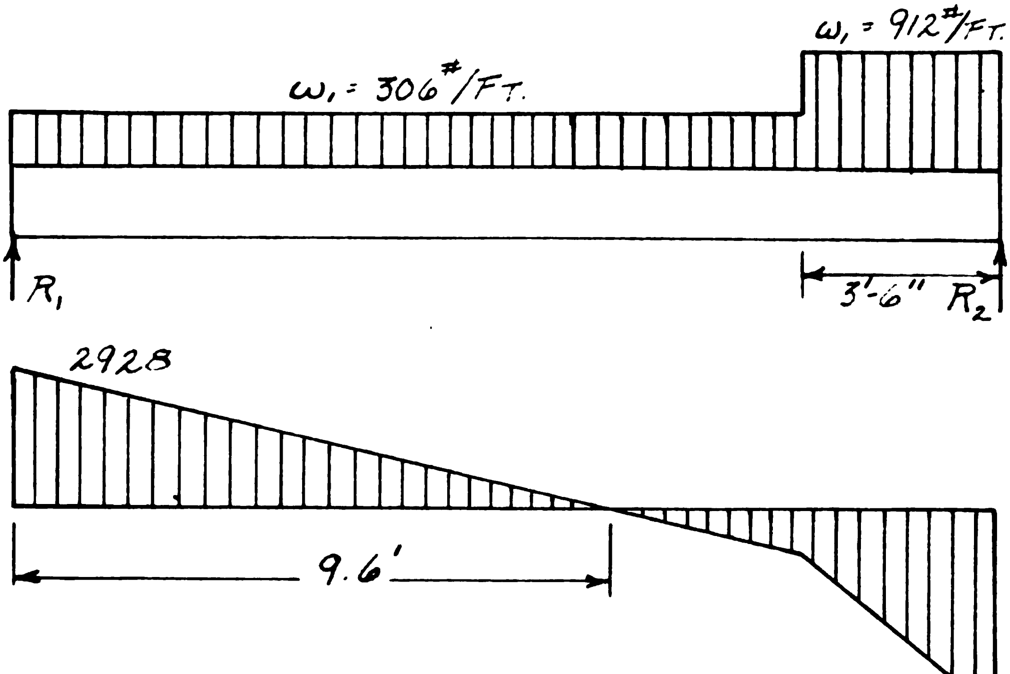

ω₁ = 912#/FT.
ω₁ = 306#/FT.
R₁
3'-6"
R₂
2928
9.6'
-5462

FIG. 21

<u>Design of Offices Roof Slab</u> (See Fig. 20)

Use $2\frac{1}{2}$" Slab.

w_1 = L.L. = 40# per sq. ft. (L.B.C.)

$w_2 = 1 \times \frac{2.5}{12} \times 150 = 31.2\#$ per ft.

w = 71.2# per ft.

$$M = \frac{wl^2}{8} = \frac{71.2\ (2.5)^2}{8} = 55.7'\# \quad \text{or } 667''\#$$

$$\frac{kd}{2.5} = \frac{1125}{2792} \qquad kd = 1.01''$$

2/3 kd = 1.01 x 2/3 = .67" 2.5-1.01 = 1.49

$$C = \frac{1125}{2} \times 1.01 \times 12 = 6{,}820\#$$

6820 x .67 = 4,570"#

A_s x 20,000 x 1.49 = 4,570"#

$$A_s = \frac{4570}{20{,}000 \times 1.49} = .154 \text{ sq. in per ft.}$$

Use Steel-Tex or comparable expanded steel reinforcement having an area of .154 sq. in. per ft. width.

Check Shear -

$$\frac{71.2 \times 2.5}{2} = 89\# \qquad \frac{89}{12 \times 2.5} = 2.9 \text{ P.S.I.}$$

<u>OK</u>

Temp. Steel for Roof Slab -(A.C.I.)

Max. Spacing = 5t or 18"

$$\frac{A_s}{2.5 \times 12} = .0022 \qquad A_s = .066 \text{ Sq. In. per ft.}$$

5t = 5 x 2.5 = 12"

Use $\frac{1}{4}$" round bars @ 9" o.c.

Wt. of Office Partitions

2" x 4" Studs - Plastered Both sides.

16# per sq. ft. x9x1 = 144# per ft.

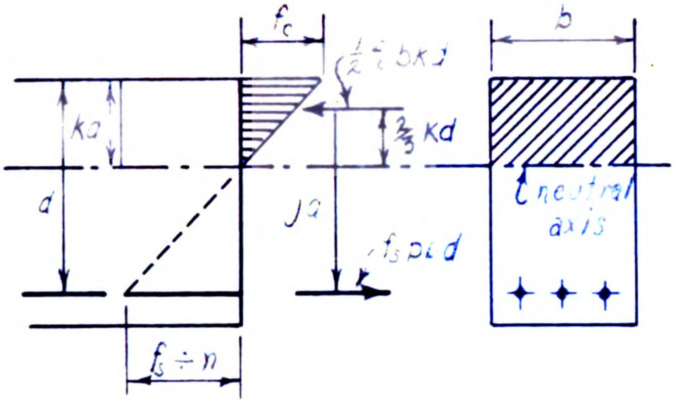


FIG. 22

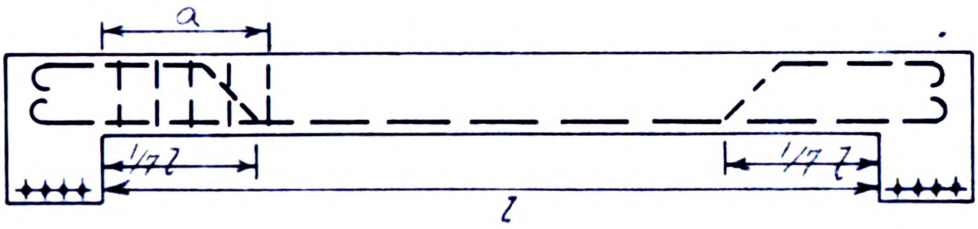
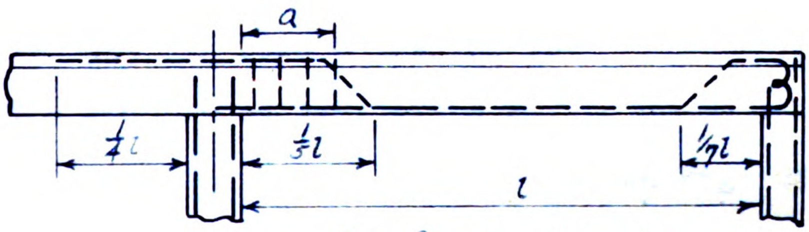
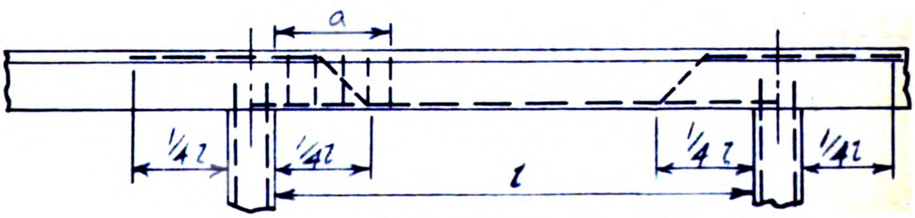


FIG 23 SIMPLE SPAN



END SPAN FIG. 24



INTERIOR SPAN FIG 25

Equiv. Uniformly Distributed Load on Floor Joists -

$M = 144 \times 2 \times \frac{13.5}{2} \times 3.5 = 800'\#$

$\frac{wl^2}{8} = 800$

$w = \frac{800(8)}{(17)^2} = 22.1\#$ per ft.

$\frac{22.1}{2} = 11\#$ per sq. ft. Equiv. partition load supported by joists.

DESIGN OFFICE FLOOR

Using 20" metal tapered pans and concrete joists.

Clear span = 17'

Live Load = 50# per sq. ft. (L.B.C.)

Finish Floor = 1/24 x 1 x 1 x 150 = 6# per sq. ft.

Equiv. Part. Load = 11# per sq. ft.

Total superimposed load = 67# per sq. ft.

From "Simplified Design of Concrete Floor Systems" - Table 10

6" Pans - 2½" Top - 4" Joists - 24" o.c.

Wt. Slab = 50# per sq. ft.

$A_s = \frac{67 + 50}{289} \times 1.95 = .79$ sq. in.

Use 2 - 3/4" round bars -2" o.c.

Bend as shown Fig. 26

Check bond -

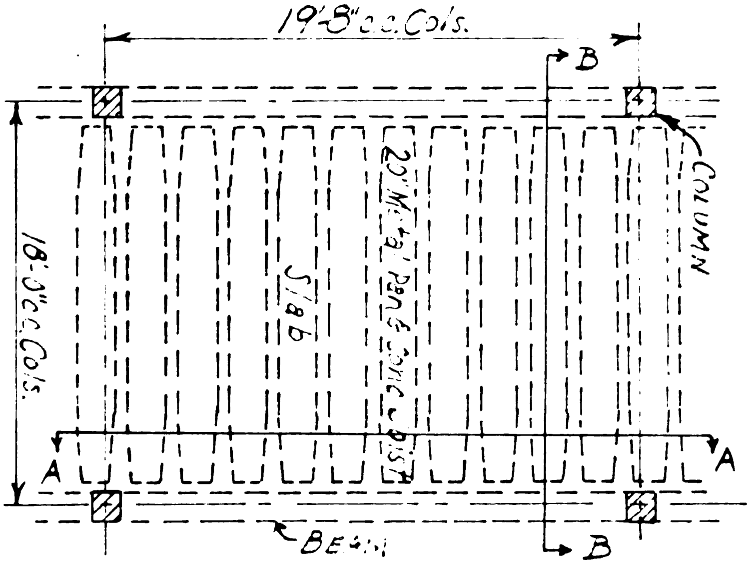
$u = \frac{V}{\Sigma o\, jd} = \frac{1131}{4.72 \times .875 \times 7.31} = 37.6$ P.S.I. OK

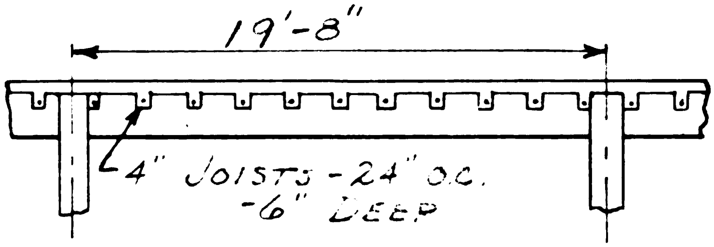
Using Standard Hooks -

$L = \frac{f_s D}{4u} = \frac{20,000(.75)}{4\ (150)} = 25"$

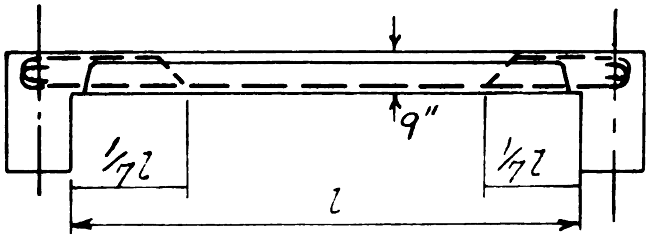
Tranverse Steel Req'd. = .049 Sq. In. per ft. at 5f o.c.

Use ¼" round bars - 12 o.c.





SECT. A-A



SECT. B-B

FIG. 26

$(\frac{.25}{2})^2 \times Pi = .049$ Sq. In. <u>OK</u>

Dead Load on Beam #1

Wt. Roof -

Concrete $= \frac{2.5}{12} \times 1 \times 1 \times 150 = 31.2$# Per Sq. Ft.

Joists $= 8.5 \times 1/2.5 = 3.4$# Per sq. ft.

Plaster and suspended ceiling = 10.0# Per sq. ft.

TOTAL = 44.6# Per sq. ft.

$\frac{17 \times 60 \times 44.6}{2 \times 60} =$ 379# Per ft. of beam from roof.

Wt. Wall -

$8/12 \times 125 \times 12 \times 60 = 60{,}000$#

Less Window Openings

$11 \times 7.5 \times 8/12 \times 125 \times 3 = 20{,}600$#

Remaining -

$60{,}000 - 20{,}600 = 39{,}400$#

$\frac{39{,}400}{60} =$ 657# per ft. due to brick work.

$5 \times 9 =$ 45# per ft. due to wall plaster.

$(6 + 11 + 50) \times 9 =$ 603# per ft. due to floor.

Total dead load on beam #1 = 379 + 657 + 45 + 603 =

<u>1684# per ft.</u>

Live Load on Beam #1

From Roof = $40 \times 17/2 =$ 340# per sq. ft.

From Floor = $50 \times 17/2 =$ 425# per sq. ft.

Total live load = 765# per sq. ft.

Design Beam # 1

$w_1 = 76.5 + 1684 = 2449$# per ft.

w_2 = 300# per ft. (Assume)

w = 2749# per ft.

Span = 18' - 8"

<u>From Fig. 1</u>

$$\text{Max. Mom.} = .1\ wl^2 = \frac{2749 \times (18.67)^2}{10} = 95{,}700\ '\#$$

or 1,148,000"#

<u>From Fig. 22</u> Assume b = 12"

$$C = \tfrac{1}{2} f_c bkd = \frac{1125}{2}(12)(.4030)d = 2725d$$

$$2725d \times jd = 1{,}114{,}800$$

$$d^2 = 486$$

$$d = 22.0"$$

22" + 2" cover = 24" Beam

Check Wt. Beam -

24/12 x 1 x 1 x 150 = 300# per ft. <u>OK</u>

$$A_s = \frac{M}{f_s jd} = \frac{1{,}114{,}800}{20{,}000 \times 22 \times .8657} = 2.90 \text{ sq. in.}$$

Use 4-1" round bars spaced $2\tfrac{1}{2}$" o.c. and bend as shown in Figs. 24 and 25.

Check Bond -

$$u = \frac{V}{\Sigma o\, jd} = \frac{2749 \times 18.67/2}{12.5 \times .8657 \times 22} = 106 \text{ P.S.I.} \quad \underline{OK}$$

Check Shear

$$u = \frac{V}{bjd} = \frac{25650}{12 \times .8657 \times 22} = 110 \text{ P.S.I.}$$

Stirrups -

v_c = 50 P.S.I. Allowable (A.C.I.)

v' = 110 - 50 = 60 P.S.I.

P.C.A. Method - From S.D.C.F.S., Fig. 17.

$v'b$ = 720

$$a = \frac{1}{2} \times \frac{v'}{v} = \frac{60 \times 18.7}{2 \times 110} = 5.08' = 5'\text{-}1"$$

Max. Allowable Spacing = 22/2 = 11"

Use 3/8" round stirrups S = 3 @ 6", 1 @ 9", 3 @ 11"

Using Standard Hooks -

$$L = \frac{f_s D}{4u} = \frac{20,000(1)}{4(150)} = 33.3"$$

Dead Load on Beam # 2

From Roof	= 379# per ft.
From Wall	= 657# per ft.
From Wall Plaster	= 45# per ft.
From Floor	= 603# per ft.
TOTAL	1684# per ft.

Live Load on Beam # 2

From Office Roof	= 340# per ft.
From Office Floor	= 425# per ft.
TOTAL	765# per ft.

Dead Load and Live Load from Garage Roof -

$$\frac{\frac{4520}{2} \times 12}{60} = 452\# \text{ per ft.}$$

w_1 = 452 + 765 + 1684 = 2901# per ft.

w_2 = 350# per ft. (Assume)

w = 3251# per ft.

Design Beam # 2

w = 3251# per ft. Span = 18.67'

M max. = $.10\ wl^2$ (see fig. 1)

$$= \frac{3251\ (18.67)^2}{10} = 113,100'\# \text{ or } 1,358,000"\#$$

From Fig. 22 Assume $b = 12"$

$$C = \frac{1}{2} f_c bkd = 1125/2\ (12)(.4030)\ d = 2725d$$

$$2725d \times jd = 1,358,000$$

$$2360d^2 = 1,358,000$$

$$d^2 = 575$$

$$d = 24.0"$$

$24" + 2"$ cover = 26" Beam

Check Wt. Beam -

$\frac{26}{12} \times 1 \times 1 \times 150 = 325\#$ per ft. <u>OK</u>

$$A_s = \frac{M}{f_s jd} = \frac{1,358,000}{20,000(.8657)(24)} = 3.25 \text{ sq. in.}$$

Use 3-$1\frac{1}{8}"$ round bars -3" o.c.

Bend as shown in Figs. #24 and # 25.

Check Bond -

$$u = \frac{V}{\Sigma o\, jd} = \frac{3251 \times 18.67}{13.5 \times .8657 \times 24} = 103 \text{ P.S.I.}$$ <u>OK</u>

Check Shear -

$$v = \frac{V}{bjd} = \frac{30,300}{12 \times .8657 \times 24} = 120 \text{ P.S.I.}$$

Stirrups -

$v_c = 50$ P.S.I. A.C.I.

$v' = 120 - 50 = 70$ P.S.I.

From S.D.C.F.S. Fig. 17 -

$v'b = 70 \times 12 = 840$

$a = \frac{1}{2} \frac{v'}{v} = \frac{18.67 \times 70}{2 \times 120} = 5.44' = 5'$-6"

Max. $S = 24/2 = 12"$

Use 3/8" round stirrups, S = 1 @ 3", 4 @ 6",

1 @ 9", 3 @ 12"

Using Standard Hooks -

$$L = \frac{f_s D}{4u} = \frac{20,000 \times 1\text{-}1/8}{4(150)} = 37.5''$$

DESIGN BEAM # 3 -

Wt. Partition	= 144# per ft.
Floor L.L. = 50 x 1.5 =	75# per ft.
Floor D.L. = 3/12 x 150 x 2 =	75# per ft.
	w_1 = 294# per ft.
	w_2 = 85# per ft. (Assume)
	w = 379# per ft.

Span = 17'

$$M = 1/8\, wl^2 = \frac{379(17)^2}{8} = 13,700'\# \quad \text{or} \quad 164,500''\#$$

Assume b = 6" See fig. 22

$$C = \tfrac{1}{2} f_c bkd = \frac{1125}{2}(6)(.4030)(d) = 1360d$$

$$1360d \times jd = 164,500$$

$$1190d^2 = 164,500$$

$$d^2 = 138$$

$$d = 11.75''$$

11.75" + 1.75" cover = 13½" Beam

Check Wt. Beam -

$$\frac{13.5}{12} \times \frac{6}{12} \times 1 \times 150 = 84.5\# \text{ per ft.} \quad \underline{\text{OK}}$$

$$A_s = \frac{M}{f_s jd} = \frac{164,500}{20,000 \times .8657 \times 12} = .784 \text{ sq. in.}$$

Use 2-3/4" round bars -2" o.c.

Bend as shown Fig. 23.

Check Bond -

$$u = \frac{V}{\Sigma o\, jd} = \frac{379 \times 8.5}{4.72 \times .8657 \times 12} = 65 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Check Shear -

$$v = \frac{V}{bjd} = \frac{3220}{6 \times .8657 \times 12} = 51.2 \text{ P.S.I.}$$

v_c = 50 P.S.I. (A.C.I.) Therefore no web reinforcement required.

Using Standard Hooks -

$$L = \frac{f_s D}{4u} = 25''$$

DESIGN ONE WAY RECTANGULAR SLAB -

13'-6" x 5'-0"

Live Load = 50# per sq. ft.

Wt. Slab = 38# per sq. ft.

From Table 7 - S.D.C.F.S.

3" Slab Allowable Load = 140# per sq. ft.

$$A_s = .19 \times \frac{50 + 38}{140 + 38} = .10 \text{ sq. in per ft.}$$

Use 3/8" round bars spaced 8.5" o.c.

Every other bar bent up at supports as shown in fig. 26.

Check P -

$$u = \frac{V}{Pjd} \qquad P = \frac{V}{ujd}$$

$$P = \frac{88(2.5)}{125(.8657)(3-.75)} = .9'' \quad \underline{\text{OK}}$$

Temp. Steel -

$$\frac{A_s}{3\text{x}12} = .002 \,, \qquad A_s = .072 \text{ sq. in. per ft.}$$

Max. Spacing = 5t = 15" (A.C.I.)

Use 3/8" round bars - 15" o.c.

DESIGN BEAM # 4

Span = 13'-0"

Floor Load (Dead & Live) = 88 x 2.5 = 220# per ft.

Partition Load = <u>144#</u> per ft.

w_1 = 364# per ft.

w_2 (Assume) = <u>70#</u> per ft.

w = 434# per ft.

$$M = 1/8\ wl^2 = \frac{434(13)^2}{8} = 9{,}170'\# \text{ or } 110{,}000''\#$$

From Fig. 22   Assume b = 6

$$C = \tfrac{1}{2} f_c bkd = \frac{1125}{2}(6)(.403)(d) = 1363d$$

$$1363d \times jd = 110{,}000''\#$$

$$1180d^2 = 110{,}000$$

$$d^2 = 93.3$$

$$d = 9.65 \quad \text{Say } 9.75$$

9.75" + 1.5" cover = 11.25" Beam

Check Wt. Beam-

$$\frac{6}{12} \times \frac{11.25}{12} \times 150 = 70\# \text{ per ft.} \quad \underline{\text{OK}}$$

$$A_s = \frac{M}{f_s jd} = \frac{110{,}000}{20000(.8657)(9.65)} = .66 \text{ sq. in.}$$

Use 2-3/4" round bars - spaced 2" o.c..

Bend as shown in fig. 23.

Check Bond -

$$u = \frac{V}{Pjd} = \frac{434 \times 6.5}{4.72(.8657)(9.75)} = 70.2 \text{ P.S.I.} \quad \underline{\text{OK}}$$

Check Shear -

$$v = \frac{V}{bjd} = \frac{2820}{6(.8657)(9.75)} = 55 \text{ P.S.I.}$$

v_c = 50 P.S.I. v' = 55 - 50 = 5 P.S.I.

From S.D.C.F.S. Fig. 17

(over)

From S.D.C.F.S. Fig. 17

$v'd = 5 \times 6 = 30$

$a = \frac{1}{2} \times \frac{v'}{v} = \frac{13 \times 5}{2 \times 55} = .59' = 7"$

Max. Spacing = 9.75/2 = 4.8"

Use 2-$\frac{1}{4}$" round stirrups @ 4"

Using Standard Hooks -

$L = \frac{f_s D}{4u} = \frac{20{,}000(.75)}{4(150)} = 25"$

<u>DESIGN STAIRS</u>

<u>Lower Flight</u> - 4'-6" Wide - Horizontal span = 11'-3"

From S.D.C.F.S. - Table 23

Thickness of slab = 7"

Steel Required = 5/8" round bars - 7" o.c.

For Bending and Placing details see Fig. 28

Wt. Lower Flight -

$\frac{7.5}{12} \times \frac{9}{12} \times 4.5 \times 150 \times .5 = 158\#$

$\frac{9}{12} \times \frac{7}{12} \times 4.5 \times 150 = \underline{296\#}$

$454\#$

$\frac{454 \times 12}{9} = 605\#$ per horizontal ft. of stairs.

<u>Upper flight</u> - 3'-6" Wide - Horizontal Span = 5'-3"

From S.D.C.F.S. - Table 23

Thickness of slab = 4"

Steel Required = 3/8" round bars - 5.5" o.c.

For Bending and Placing details see Fig. 28.

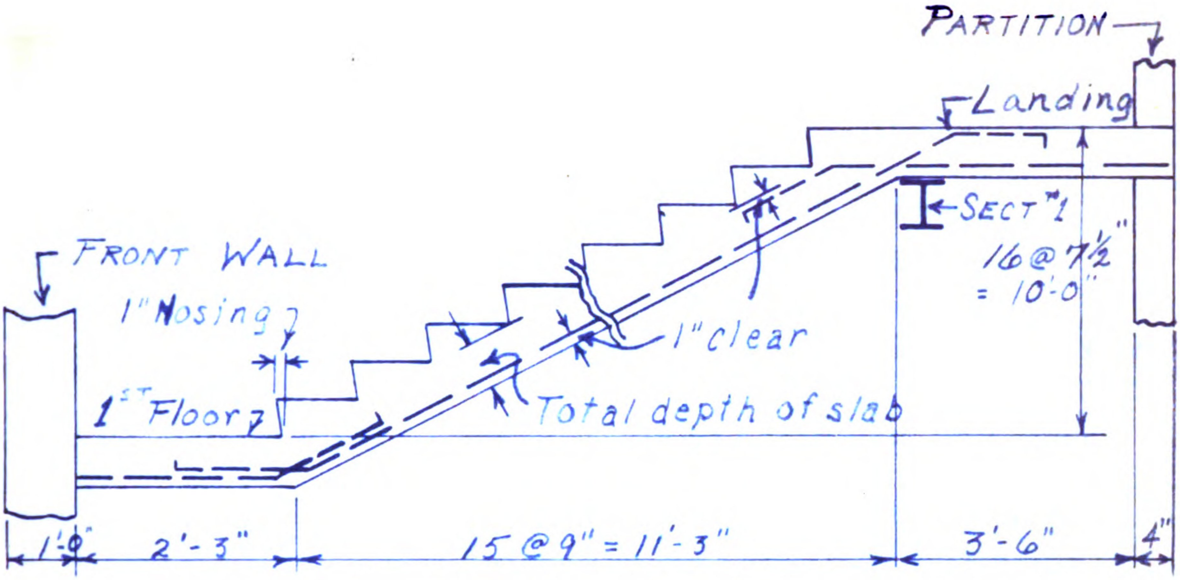


FIG. 27 LOWER FLIGHT

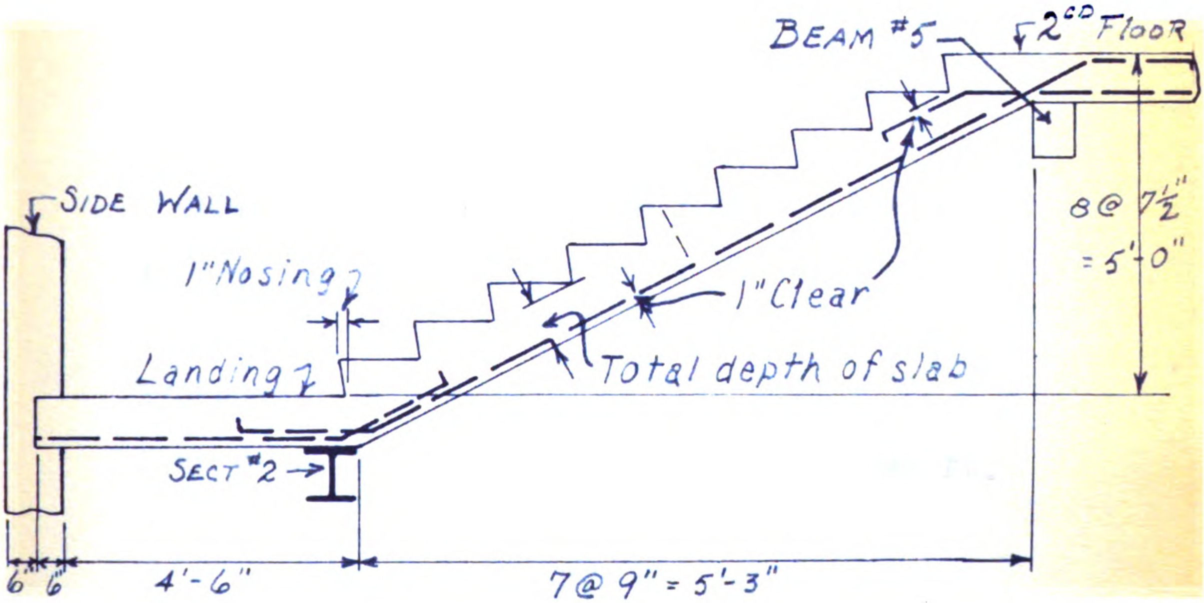


FIG. 28 UPPER FLIGHT

STU

Sect

Wt. Upper Flight -

$$\frac{7.5}{12} \times \frac{9}{12} \times 3.5 \times 150 \times .5 = 123\#$$

$$\frac{4}{12} \times \frac{9}{12} \times 3.5 \times 150 = \underline{131.3\#}$$

$$254.3\#$$

$$\frac{254.3 \times 12}{9} = 339\#$$ per horizontal ft. of span.

Note***Steel of upper flight to be continued across landing forming a one way slab - 3.5" thick, capable of withstanding load of 296# per sq. ft.
Table 7 - S.D.C.F.S.

STRUCTURAL STEEL TO SUPPORT STAIR

Section # 2 - See Fig. 28

Dead Load -

$\frac{1}{2}$ upper flight = 339 x 5.25 x .5 = 850#

$\frac{1}{2}$ landing = 44 x 3.5 x 4.5 x .5 = <u>327#</u>

1177#

Live Load -

$\frac{1}{2}$ upper flight = 100 x 5.25 x 3.5 x .5 = 918#

$\frac{1}{2}$ landing = 100 x 4.5 x 3.5 x .5 = <u>788#</u>

1706#

1706# + 1177# = 2883#

Span = 3.5 ft. $\quad w = \frac{2883}{3.5} = 824\#$ per ft.

$$V = \frac{2883}{2} = 1441$$

$$M = 1/8\ wl^2 = \frac{824(3.5)^2}{8} = 1265'\# \text{ or } 15,200''\#$$

$$S = \frac{M}{f} = \frac{15200}{18000} = .85''^3$$

$$A = \frac{V}{v} = \frac{1441}{12,000} = .12 \text{ sq. in.}$$

Use 5" I 10# - 3'-10" long - with Standard A.I.S.C. A-1 connection to column "A"

Section I - See Fig. 27

<u>Dead Load</u> -

$\frac{1}{2}$ lower flight = $11.25 \times 605 \times .5 = 3410\#$

<u>Live Load</u> -

$\frac{1}{2}$ lower flight = $100 \times 11.25 \times 4.5 \times .5 = 2530\#$

$3410\# + 2530\# = 5940\#$

$w = \frac{5940}{2} = 1{,}320\#$ per ft.

$M = 1/8\ wl^2 = \frac{1320(4.5)^2}{8} = 3360'\#$ or $40{,}250''\#$

$S = \frac{M}{f} = \frac{40250}{18000} = 2.24''^3$

$V = \frac{5940}{2} = 2970\#$

$A = \frac{V}{v} = \frac{2970}{12000} = .248$ sq. in.

Use 5" I 10# -5 ft. long with standard A.I.S.C. beam connection A-1 to column "A".

Bearing of Sect. I on 12" Wall. -

$V = 2970 + 10 \times 4.5 \times .5 = 2995\#$

Req'd. $A = \frac{V}{f} = \frac{2995}{175} = 17.1$ sq. in.

We have $6'' \times 3'' = 18$ sq. in. Therefore no bearing plate is required.

Bearing of Sect. II on 4" Clay Tile.

Partition -

$V = 1441 + 10 \times 3.5 \times .5 = 1459\#$

Req'd. $A = \frac{V}{f} = \frac{1459}{115} = 12.7$ sq. in.

We have $4'' \times 3'' = 12$ sq. in. Therefore no bearing plate is required.

DESIGN COLUMN "A"

Dead & Live Load from floor = 434 x 6.5 = 2820#

V-Sect. I = 2995#

V-Sect. II = 1459#

Total Axial Load= 7274#

Height = 15'

Reducing f by A.I.S.C. column formula and using A.I.S.C. table for l/r ratio less than 120, a 6" x 6" H 20# is required.

Bearing Plates -

$$A = \frac{P}{f_c} = \frac{7274}{2000} = 3.6 \text{ sq. in. Req'd.}$$

Use 8" x 8" x 3/4" plates welded at top and bottom of column.

DESIGN BEAM # 5 See Fig. 21

Span = 17'-00"

Floor Load (Dead & Live) = 88 x 3.5 = 306# per ft.

Stair Load (Dead & Live) = 88 x 1 x 824 = 912# per ft.

Find Point of Maximum Moment -

306 x 17 x .5 = 2600#

912 x 3.5 = 3190#

3190 x 1.75/17 = 328#

R_1 = 2600 + 328 = 2928#, R_2 = 2600 + 2862 = 5462#

2928 - 306X = 0

X = 9.6'

Max. Mom. = 2928 x 9.6 - 306 x 9.6 x 9.6 x .5

M = 14,000'# or 168,000"#

w_2 = 100# per ft. (Assume)

$17 \times 100 \times .5 = 850\#$

Additional Moment-

$M = 850 \times 9.6 - 100 \times 9.6 \times 9.6 \times .5 =$

$3550'\#$ or $42{,}600''\#$

Total $M = 168{,}000 + 42{,}600 = \underline{210{,}600''\#}$

From Fig. 22 Assume $b = 8''$

$C = \frac{1}{2} f_c bkd = \frac{1125}{2}(8)(.403)d = 1820d$

$1820d \times jd = 210{,}600''\#$

$1575d^2 = 210{,}600''\#$

$d^2 = 133.7$

$d = 11.5''$

$11.5'' + 1.5''$ cover $= 13''$ beam.

Check Wt. Beam -

$\frac{8}{12} \times \frac{13}{12} \times 150 = 108\#$ <u>OK</u>

$A_s = \frac{M}{f_s jd} = \frac{210{,}600}{20{,}000(.8657)(11.5)} = 1.05$ sq. in.

Use 2-7/8" round bars -3" o.c.

Bend as shown in Fig. 23.

Check Bond -

$u = \frac{V}{\Sigma o\, jd} = \frac{5462}{5.5(.8657)(11.5)} = 98.7$ P.S.I. <u>OK</u>

Check Shear -

$v = \frac{V}{bjd} = \frac{5462}{8(.8657)(11.5)} = 68$ P.S.I.

$v_c = 50$ A.C.I.

From S.D.C.F.S. Fig. 17

$v' = v - v_c = 68 - 50 = 38$ P.S.I.

$v'b = 38 \times 8 = 304$

$a = \frac{l}{2} \times \frac{v'}{v} = \frac{17 \times 38}{2 \times 68} = 4'\text{-}9''$

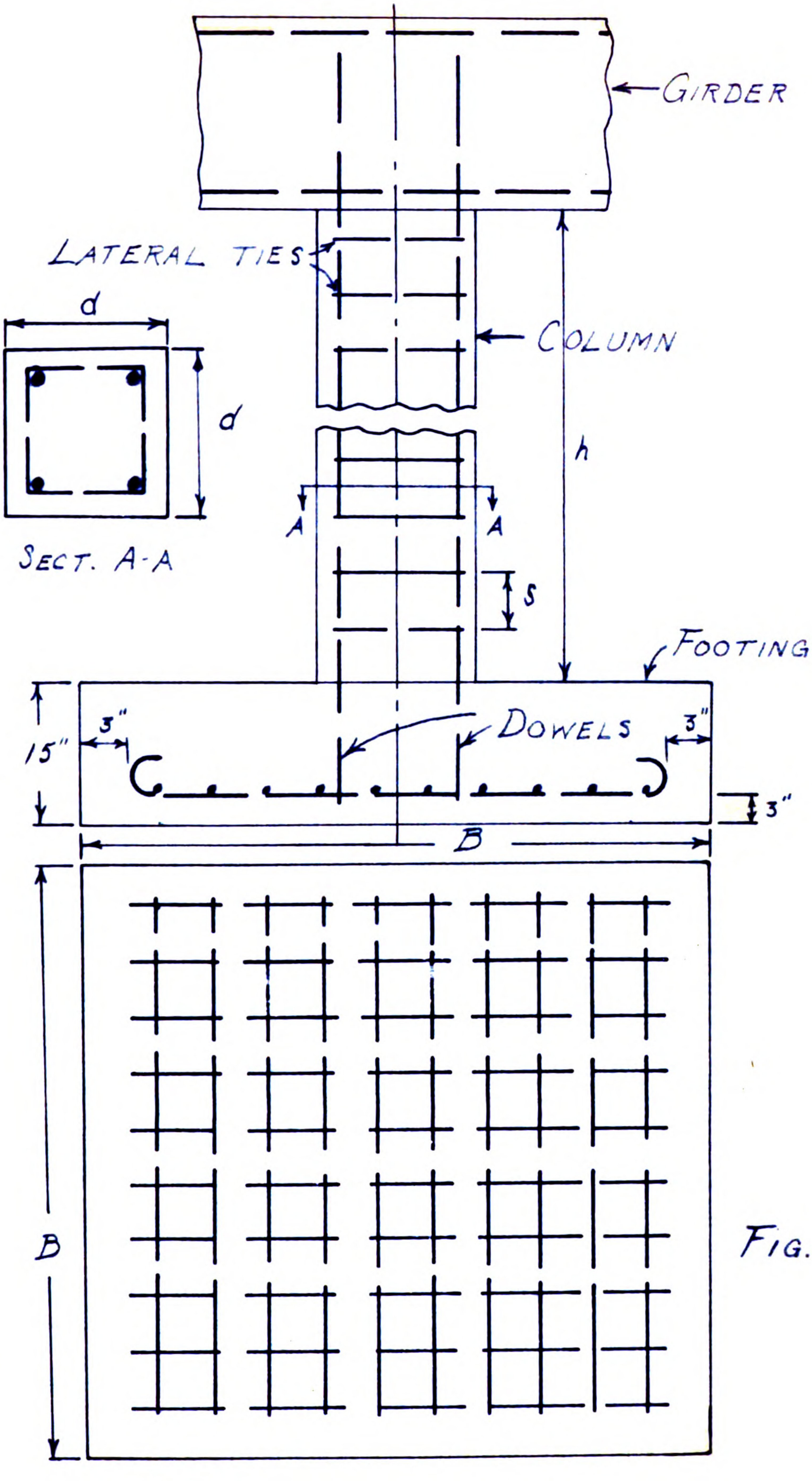

GIRDER
LATERAL TIES
d
d
COLUMN
h
A
A
SECT. A-A
S
FOOTING
15"
3"
DOWELS
3"
3"
B
B

FIG. 29

Max. Spacing = 13/2 = 6.5"

Use $\frac{1}{2}$" round stirrups   S = 10 @ 6"

Using Standard Hooks -

$$L = \frac{f_s D}{4u} = \frac{20,000(.875)}{4(188)} = 23"$$

DESIGN COLUMN # 6 & # 7

From Fig. 1 - w = 2750   l = 18.67

$$P = 1.1\ wl = 2750(18.67)(1.1) = 56,467\#$$

From Fig. 29 -

$$\frac{\text{Height}}{\text{Least Dimension}} = \frac{14.25}{1} = 14.25$$

Increase Load for $\frac{h}{d}$ ratio greater than 10 (A.C.I.)

$$P' = P(1.3 - .03\frac{h}{d})$$

$$P = \frac{56467}{1.3 - .03(14.25)} = 64,900\#$$

$$P = A_g(.225f'_c + f_s p_g).8$$

$$64,900 = 100(.225 \times 2500 + 20,000\ p_g).8$$

$p_g = .0125$ This is between .01 and .04 therefore <u>OK</u>

$$p_g = \frac{A_s}{A_g}\ ,\quad .0125 = \frac{A_s}{100}$$

$A_s = 1.25$ sq. in.

Use 4 - 5/8" round bars as shown in Fig. 24.

Lateral Ties

S = 16 x 5/8 = 10"   Use $\frac{1}{4}$" round ties as shown in Fig. 29.

DESIGN COLUMN # 5 & # 8

From Fig. 1   w = 2750   l = 18.67

$$P = .4\ wl = .4(2750)(18.67) = 20,534\#$$

This value is less than that of columns # 6 and 7, therefore the same design will be used as this is the min-

imum (A.C.I.) amount of steel allowable.

DESIGN COLUMN # 2 and #3

From Fig. 1 - $w = 3251$ $l = 18.67$

$$P = 1.1\ wl = 1.1(3251)(18.67) = 66{,}754\#$$

From Fig. 29 - $\frac{h}{d} = \frac{14.25}{1} = 14.25$

Increase P for $\frac{h}{d}$ greater than 10

$$P' = P(1.3 - .03\tfrac{h}{d})$$

$$P = \frac{66{,}754}{1.3 - .03(14.25)} = 76{,}600\#$$

$$P = A_g(.225\ f_c' + f_s p_g).8$$

$$76{,}600\# = 100(.225 \times 2500 + 20{,}000 \times p_g).8$$

$p_g = .0198$ This is between .01 and .04 - OK

$p_g = \frac{A_s}{A_g}$, $A_s = p_g A_g = 100(.0198) = 1.98$ sq. in.

Use 4 - 7/8" round bars as shown in Fig. 29.

Lateral Ties -

$S = 16 \times 7/8 = 14"$

$S = 48 \times \frac{1}{4} = \underline{12"}$ Use $\frac{1}{4}$" round ties as shown in Fig. 29.

DESIGN COLUMN # 1 & # 4

From Fig. 1 - $w = 3251$ $l = 18.67$

$$P = .4\ wl = .4(3251)(18.67) = 24{,}274\#$$

This value is less than that of columns #6 and #7, therefore the same design will be used as this is the minimum (A.C.I.) amount of steel allowable.

FOOTING DESIGN

Soil pressure = 2000# per sq. ft.

f'_c = 2500 P.S.I.

n = 12

f_c = 1125 P.S.I.

f_s = 20,000 P.S.I.

v = 75 P.S.I.

u = 141 P.S.I.

Column Footing #9, #10, #11, #12

Col. Load = 28,440 + 2000 + 13(20) = 30,700#

From Table 35 - R.C.D.H.

Use Footing 4'-9" square - 15" deep

9-3/8" round bars each way with standard hooks placed as shown in Fig. 29.

Column Footing # 2, # 3

Col. Load = 66,754 + 1 x 1 x 150 x 13 = 68,704#

From Table 35 - R.C.D.H.

Use Footing 6'-8" square - 15" deep.

17-3/8" round bars each way - standard hooks - placed as shown in Fig. 29.

Column Footing # 1

Col. Load = 24,274 + 1950 = 26,224#

From Table 35 - R.C.D.H.

Use Footing 4'-9" square - 15" deep

9-3/8" round bars each way - standard hooks - placed as shown in Fig. 29.

Column Footing # 6 and # 7

Col. Load = 56,467 + 1950 = 58,417#

From Table 35 - R.C.D.H.

Use 5'-9" square footing - 15" deep

11-3/8" round bars each way - standard hooks -

placed as shown in Fig. 29.

<u>Column Footing # 8</u>

Col. Load = 20,534 + 1950 = 22,484#

From Table 35 - R.C.D.H.

Use Footing 4'-9" square - 15" deep

9-3/8" round bars each way - standard hooks -

placed as shown in Fig. 29.

<u>Pier Foot # 13, # 14, #15, #16</u>

Pier Load = 14,220 + 2 x 2 x 13 x 120 = 20,460#

From Table 35 - R.C.D.H.

Use Footing 3'-4" square - 15" deep

6-3/8" round bars each way - standard hooks -

placed as shown in Fig. 29.

<u>Column Footing # 1, # 4</u>

Col. Load = 26,224#

Req'd. Area = $\frac{26,224}{2000}$ = 13.112 sq. ft.

Width Footing = 20" = 1.67'

Length " = $\frac{13.112}{1.67}$ = 7.8' = 7'-9"

$M = 13,112 \times \frac{1}{2} \times 8/2 = 26,224'\# = 315,000''\#$

Assume total depth = 15" d= 12" 3" clear.

From Fig. 22

$C = \frac{1}{2} f_c bkd = \frac{1}{2}(1125)(20)(.4030)(12) = 54,500\#$

$C \times jd = 54,500(.8657)(12) = 567,000''\#$ <u>OK</u>

$T \times jd = A_s(20,000)(.8657)(12) = 315,000''\#$

$$A_s = \frac{315{,}000}{20{,}000(.8657)(12)} = 1.515 \text{ sq. in.}$$

Use 5-5/8" round bars - 3" o.c. - Standard hooks - as shown Fig. 29 except only one way.

Check Shear -

$$v = \frac{13{,}112}{16 \times 15} = 54.7 \quad \underline{OK}$$

Check Bond -

$$u = \frac{V}{Pjd} = \frac{13112}{9.8(.8657)(12)} = 126 \text{ P.S.I.} \quad \underline{OK}$$

<u>Column Footing "A"</u>

Col. Load = 7,274#

$$\text{Req'd. Area} = \frac{7274}{2000} = 3.63 \text{ sq. ft.}$$

Use 20" x 20" Plain Concrete Footing 12" deep.

A.C.I. - Allowable for stresses for Plain Concrete - Footings -

$$T = .03\, f_c' = .03(2500) = 75 \text{ P.S.I.}$$

$$v = .02\, f_c' = .02(2500) = 50 \text{ P.S.I.}$$

Using A.C.I. method we take moment about section half-way between column and edge of base plate.

$$w = \frac{7274}{64} = 114 \text{ P.S.I.}$$

$$M = 2000\ (8/12)\ (20/12)(4/12) - 2(114)(8)(1/12)$$

$$= 589'\# \text{ or } 7068''\#$$

From Fig. 22

$$T = C = \frac{75}{2}(6)(20) = 4500\#$$

Allowable $M = 4500 \times 8 = 36{,}000''\#$ <u>OK</u>

$$V \text{ (at edge of base plate)} = \frac{2}{12}\left(\frac{20}{12}\right)(2000) = 555\#$$

$$\frac{555}{20 \times 12} = 2.3 \text{ P.S.I.} \quad \underline{OK}$$

Therefore, no steel required in this footing.

Use Dowels to transmit stress of longitudinal column reinforcement to footings.

Columns with 5/8" round bars - Use 3/4" round bowels.

Extension into footing-

20,000 x .31 = 6200# = 2.36 x L x 125

L = 21"

Use 1-42" -3/4" round dowel - 8" pedestals will be necessary for 15" footings.

Columns with 3.4" round bars - Use 7/8" round dowels.

Extension into footings -

20,000 x .60 = 12,000# = 3.14 x L x 125

L = 30"

Use 2 30"-7/8" round dowels - no pedestal req'd.

DESIGN FOUNDATION WALL

Maximum weight per foot of outside wall will occur under south wall. With 12" x 3.5 ft. foundation wall this weight will be -

From Roof- $\frac{14,220}{18}$ = 790# per ft.

From Brickwork - $\frac{125(27x18-11x13)}{18}$ = 2380# per ft.

From Windows - $\frac{11 \times 13 \times 5}{18}$ = 40# per ft.

From Foundation - 3.5 x 1 x 150 = 525# per ft.

TOTAL - <u>3735#</u> per ft.

Allowable (A.C.I.) working stress for minimum reinforcement = .25 f'_c = .25(2500) = 625 P.S.I.

625 x 144 = 90,000# per ft. Therefore, use minimum (A.C.I.) steel in foundation wall.

.0025 x 12 x 12 = .36 sq. in. per ft. each way required.

Use 5/8" round bars spaced 18" o.c. vertically 2" from outside face.

Use 5/8" round bars spaced 18" o.c. horizontally 2" from inside face.

DESIGN WALL FOOTINGS

Wt. Wall = 3735# per ft.

Wt. Footing = 265# per ft. (Assume)

Total = 4000" per ft.

Area req'd. per ft. = $\frac{4000}{2000}$ = 2 sq. ft.

Use 2' wide footing - 12" deep - plain concrete.

Allowable T = .03 f_c' = .03(2500) = 75 P.S.I.

Taking Moment at face of Wall -

$M = 2000 \times \frac{6}{12} \times \frac{12}{12} = 1000'\# = 12{,}000''\#$

$C = T = 75/2 \times 6 \times 12 = 2700\#$

$M = 2700 \times 8 = 21{,}600''\#$ Allowable OK

Check Shear -

.02 f_c' = 50 P.S.I. Allowable

Punching Shear = $2000 \times \frac{6}{12} \times \frac{2}{12} = 1000\#$

$\frac{1000}{12 \times 12} = 7$ P.S.I. OK

Therefore, no steel required for shear or bending. For Temperature steel - use 4-3/8" round bars - 5" o.c. running longitudinally to take care of cracks due to varying soil bearing and contraction.

DESIGN FLOOR OF GARAGE

Using Portland Cement Association Bulletin No. ST 51, "Concrete Floors on Ground".

Max. Wheel Load = 5000#

Area Loaded = 30 sq. in.

Impact = 25%   5000 x 1.25 = 6250#

From Fig. 2 (ST 51)

Using 5000# concrete - modulus of rupture = 675 P.S.I.

From Fig. 3 (ST 51)

A safety factor of 2 will be used due to frequent stress repititions. Therefore the design modulus will be -

$$\frac{675}{2} = 338 \text{ P.S.I.}$$

Fig. 1 (ST 51) is based on modulus = 300, therefore -

$$6250 \times \frac{300}{350} = 5360\#$$

Entering Fig. 1 (ST 51) with load = 5360# and loaded area of 30 sq. in. a 5.25" slab is required. Use 6" slab.

Steel to Prevent Large Cracks

$$A = \frac{WLf}{2f_s} \qquad W = \tfrac{1}{2} \times 1 \times 150 = 75\#$$

L = 18'

f = 2

f_s = 20,000

$$A = \frac{75(18)(2)}{2(25,000)} = .54 \text{ sq. in. per ft.}$$

Use any welded wire fabric having this cross sectional area. A fabric with longitudinal wires, gauge = 00, and apaced 2" o.c. is suggested.

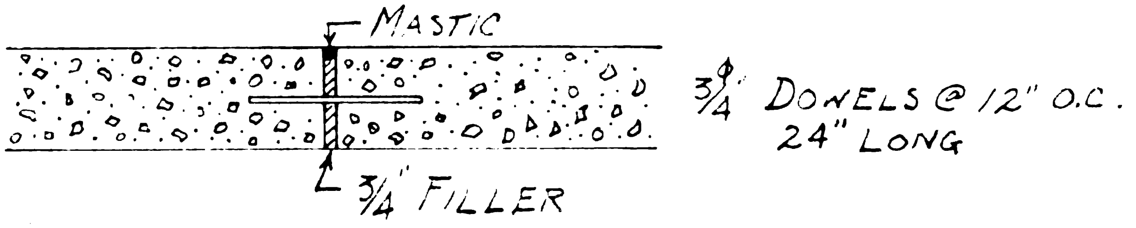


FIG. 30

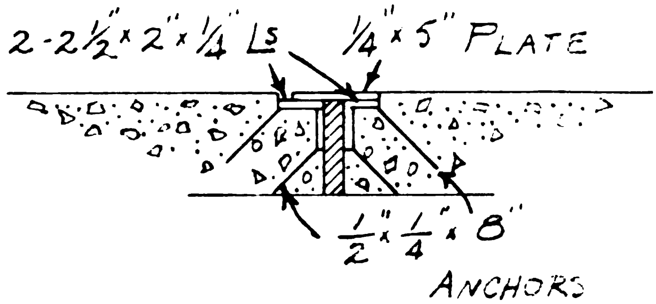


FIG. 31

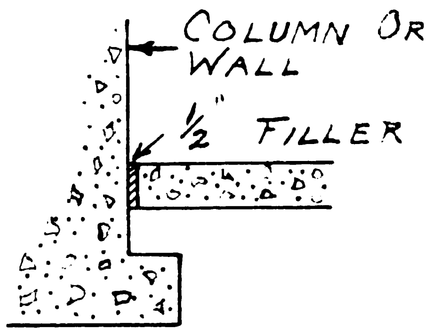


FIG. 32

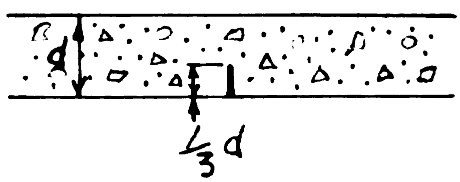


FIG. 33

3/4" expansion joints as shown in Fig. 30 should be placed on column lines as shown on sheet 1. Protected expansion joints as shown in Fig. 31 should be used at both exits. 1/2" expansion joints should be placed at all walls, columns and piers as shown in Fig. 32.

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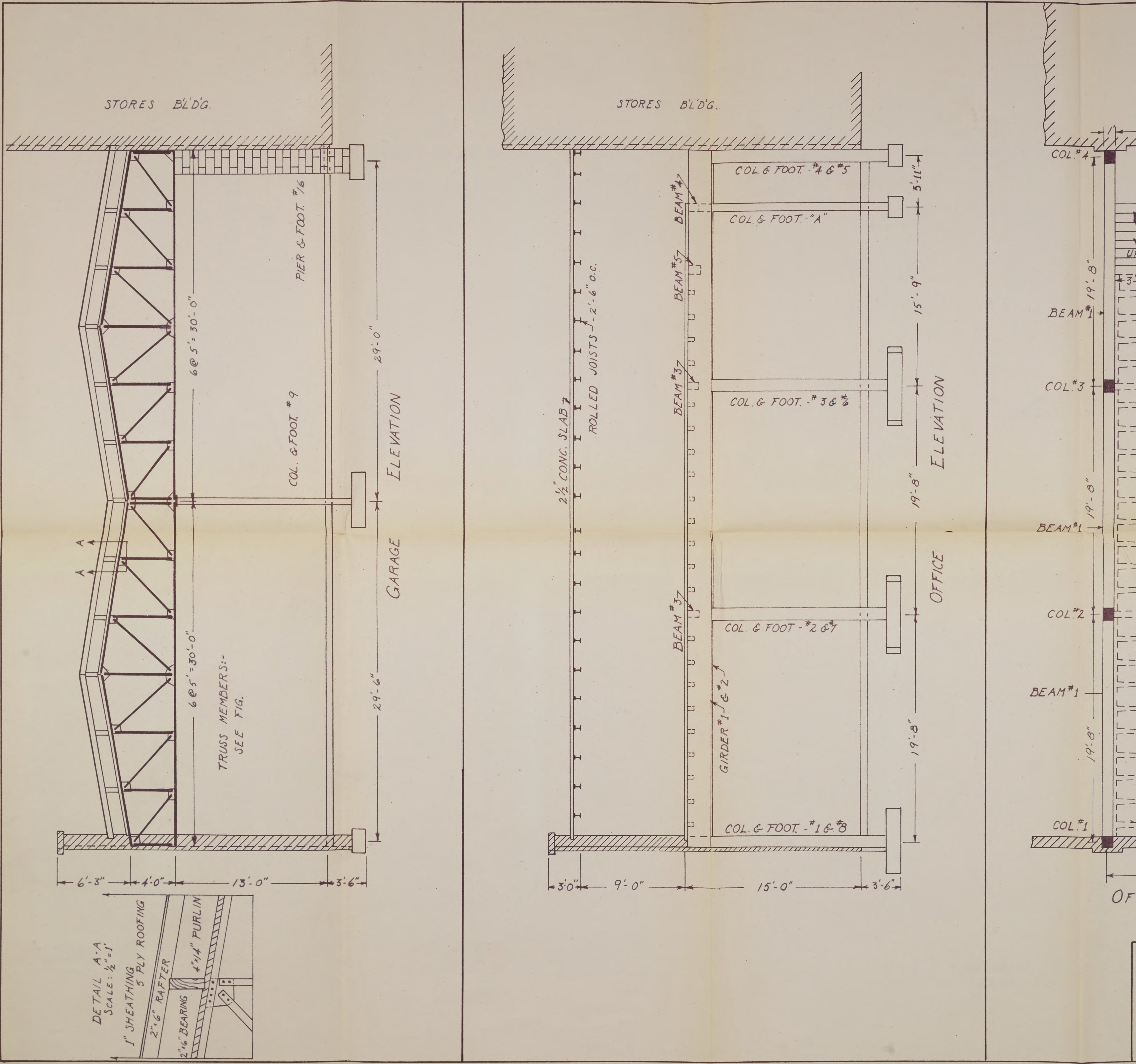
[redacted]

[redacted]



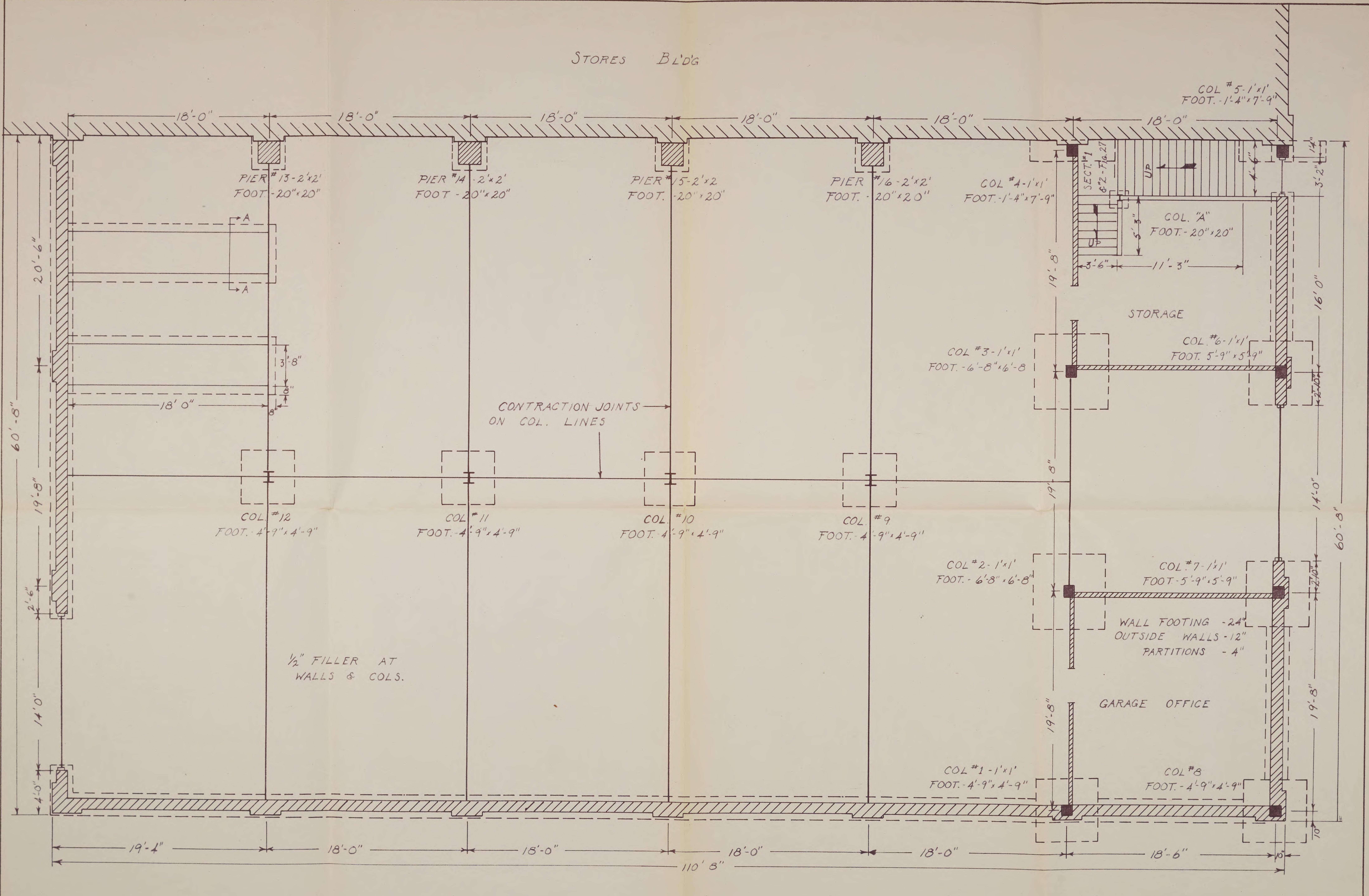
SERVICE GARAGE

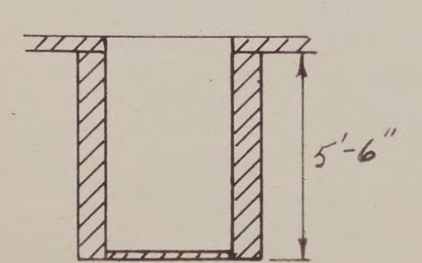
SHEET NO. 2

GARAGE & OFFICE ELEVATION

OFFICE FLOOR PLAN

DATE: 6/1/46 SCALE: 3/16"=1'





SECT. A-A

SERVICE GARAGE

SHEET NO. 1

FOOTING & GROUND FLOOR PLAN

DATE: 6/1/46 SCALE: 3/16" = 1'

Pocket has: 2 sheets