A DESIGN OF A REINFORCED CONCRETE BUILDING FOR A SOY BEAN PROCESSING PLANT

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE

W. R. Radcliff

1947

A Design of a Reinforced Concrete Building for a Soy Bean Processing Plant

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

By

Robert Radcliff

Candidate for the Degree of

Bachelor of Science

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THESIS

c. 1

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CONTENTS

PART I

PURPOSE AND SCOPE OF THESIS

The purpose of this thesis is to present a practical and economical design of a building for use by the owner. The property is located in the Central Ohio farm area. The objective of the owner is to have a building large enough and strong enough to house a Soy Bean Processing Plant. Most of the construction work will be done by the owner's organization. The building will be constructed with materials available in that area, and a cost of materials estimate will be included. The project offers several practical design problems for the author. The methods used are those set forth in the respective design courses as taught at Michigan State College. Much more labor was applied to this problem than would have been necessary if it had been handled by an experienced design engineer. Despite this fact, it is a worthwhile project because it employs design fundamentals necessary to a good designer as a basis for the tricks and short cuts of the trade.

The author will design the building in accordance with specifications supplied by the owner. The design and materials estimate will constitute the object of the thesis.

The owner plans to design and fabricate all plant equipment. While this is being done the building will be used for farm machinery repair and as a farm locker agency. By means of these operations the owner hopes to establish himself in the area.

The scope of the thesis will include the following:

A. The design of a fireproof, durable building to perform the following functions:

1. Provide suitable space for operating a Soy Bean Processing Plant.
2. Provide suitable space for repairing farm

machinery.

3. Provide suitable space for operating a farm locker agency.

4. House the following equipment:

    a. Metal working tools.

    b. Welding equipment.

    c. Overhead crane.

    d. Work benches.

    e. Woodworking machines.

    f. Lubrication facilities.

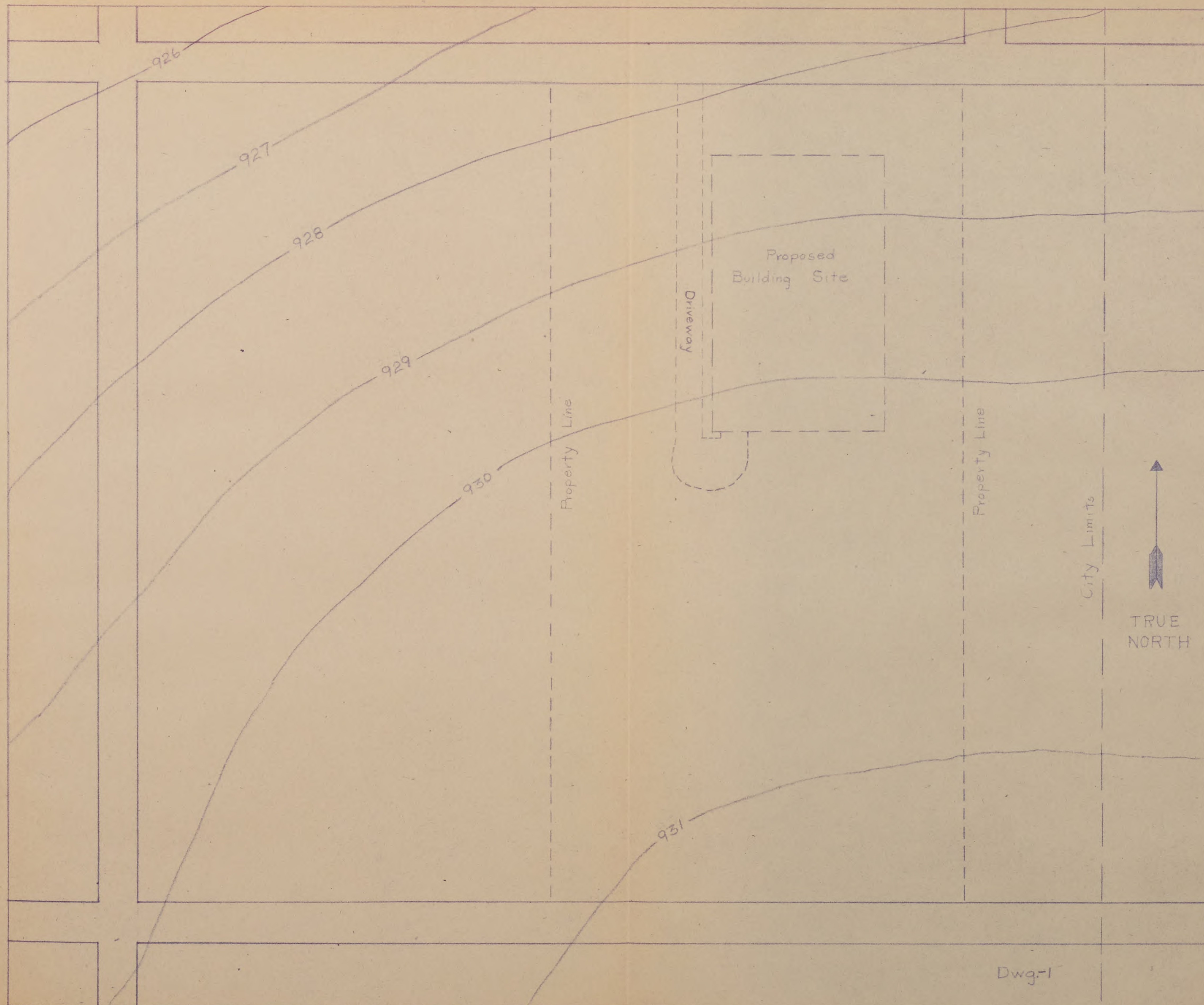
B. To locate and design:

1. A concrete driveway from the rear of the building, running parallel to the side of the building and connecting to the existing road.

2. A three-ton overhead crane system to be located on the ground floor.

C. Estimate the cost of materials for construction.

The scope of the thesis is limited to the structural design of units heretofore described. No attempt has been made to perform the functions of an architect in that architectural features and details of the allied trades such as plumbing, heating, or wiring have not been mentioned or treated.


926
927
928
929
930
931
Property Line
Driveway
Proposed
Building Site
Property Line
City Limits
TRUE
NORTH
Dwg.-1

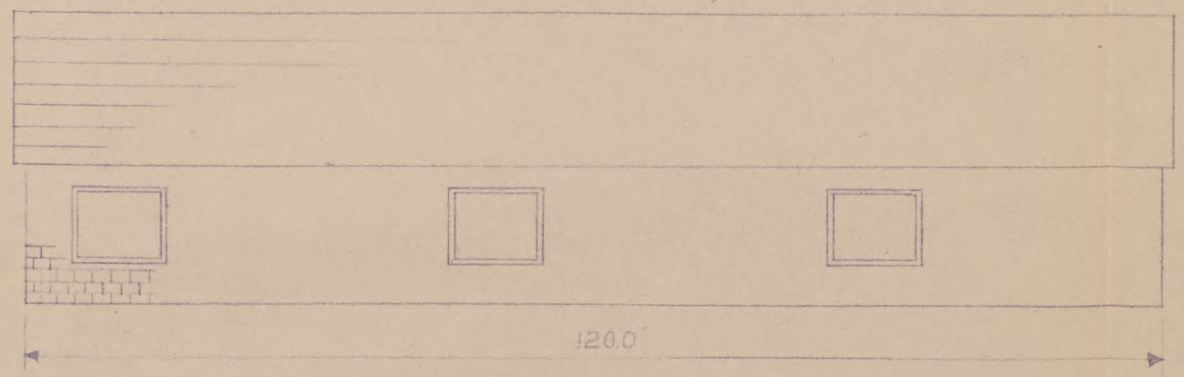
BUILDING DETAILS

Drawing 1-2-3-4.

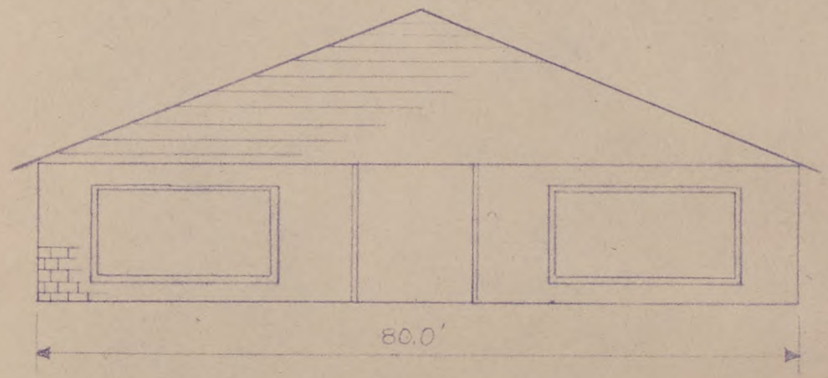
The building will be 120.0' long and 80.0' wide and will be located on the property as shown in Drawing 1. Because the author was unable to visit the building site, the data for the site map was obtained from the owner and from U.S. Geological Survey topographic maps of the area. The ground floor walls will be concrete blocks. The roof system will be constructed of wood with wood sheeting and asphalt roofing. The remainder of the building will be constructed of reinforced concrete. The driveway will be poured concrete with no tensile reinforcement. The drive will be widened at the curve to facilitate entrance into the building. Due to the heavy floor loads expected, the floor system will be designed for a live load of 400 p.s.f.; column spacing taken as 20.0'c.c. with T-Beams interesting the girders at the 1/3 points.

PROPOSED BUILDING

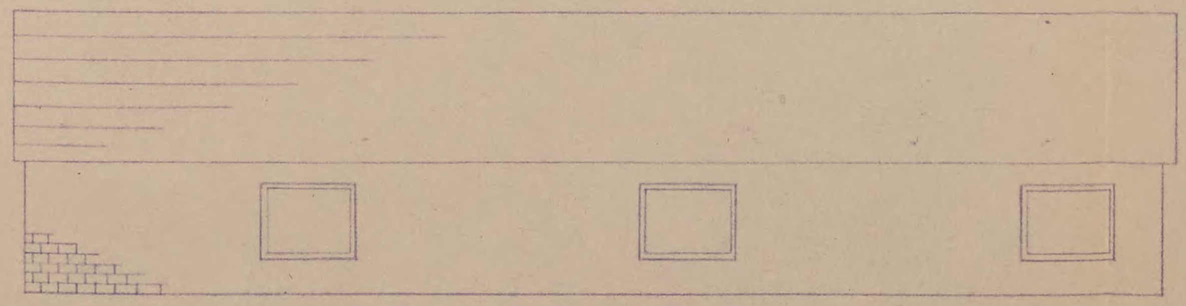
EAST SIDE



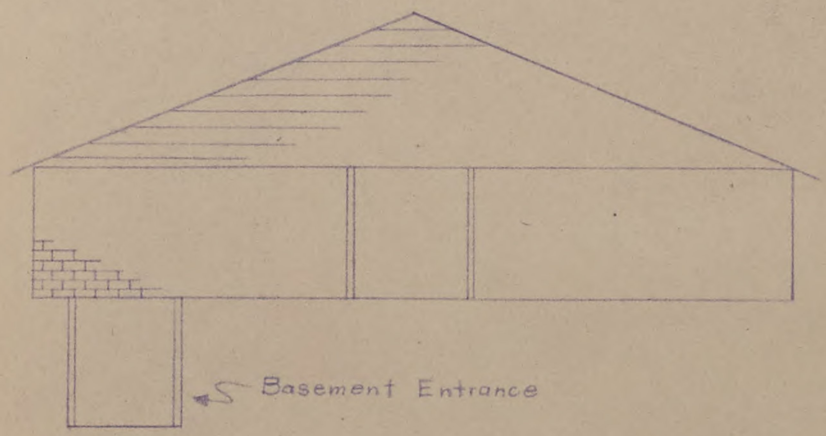
NORTH SIDE



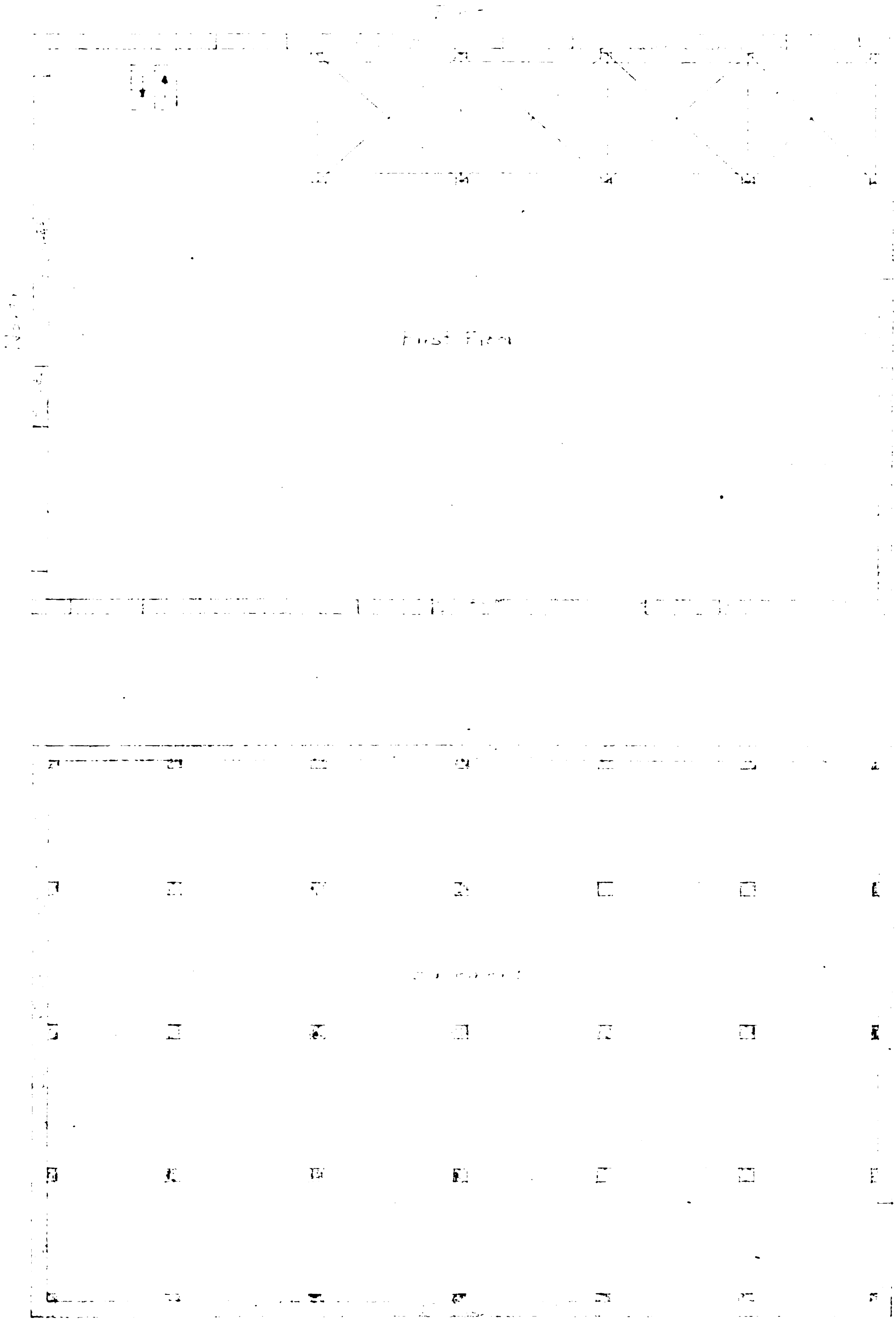
WEST SIDE



SOUTH SIDE



Dwg.-2



Dwg-3

ALLOWABLE UNIT STRESSES USED

Taken from A.C.I Building Code 1946.

Concrete

Using a concrete with fc' = 3000 p.s.i.

n = 10

Compression fc = 1350 p.s.i.

Shear v

Beams

W/o Web Reinforcement

W/o SA - 60 p.s.i.

W/ SA - 90 p.s.i.

W/ Web Reinforcement

W/o SA - 180 p.s.i.

W/ SA - 360 p.s.i.

Footings - 75 p.s.i.

Bond u

Beams

Plain bars - 120 p.s.i.

Deformed bars - 150 p.s.i.

Two-way footings

Plain bars (hooked) - 135 p.s.i.

Deformed bars (hooked) - 168 p.s.i.

Bearing fc

On full area - 750 p.s.i.

On one-third area - 1125 p.s.i.

Steel

Billet, hard grade fs = 20,000 p.s.i.

Lumber - Pine Southern Shortleaf (Dense Structural)

Extreme fiber in bending - 1900 p.s.i.

Horizontal shear - 120 p.s.i.

Compression perpendicular to grain	- 455 p.s.i.
Compression parallel to grain	-1450 p.s.i.
Modulus of elasticity	-1,600,000 p.s.i.

Soil Pressure - "American Civil Engineers' Handbook."

Dry Silt-Loam	-3 Tons/ sq. ft.

Bearing on Masonry Wall

Portland Cement Association	-800 p.s.i.

PART II

ROOF DESIGN

Sketch 1

This truss was selected from Timber Engineering Company Truss Manual.

Fink Truss

80' - 00" Span

16' - 00" Rise

16' - 00" Spacing C.C.

1' - 00" Eaves

Dead Load = 2470 @ $\frac{3000}{1000}$ FBM = 7410# Truss Weight

$\frac{1}{2}$ Roof Area = 44.0 x 16 = 704 s.f./truss

Wind Load = 20 p.s.f. of vertical surface

P normal = $\frac{(2(0.372))}{(1+(.372)^2)}$ 20 = 13.1

Snow Load = 20 p.s.f. vertical

Maximum Expected Load

Dead + Wind + $\frac{1}{2}$ Snow

Dead + $\frac{1}{2}$ Wind + Snow

Wooden sheathing 1.0" thick	- 3 p.s.f.
Asphalt covering	- 2 p.s.f.
Roof covering	5 p.s.f.
Truss = $\frac{7410}{(2)704}$ =	5.8
	10.8 Total D.L.

Dead + Wind + $\frac{1}{2}$ Snow

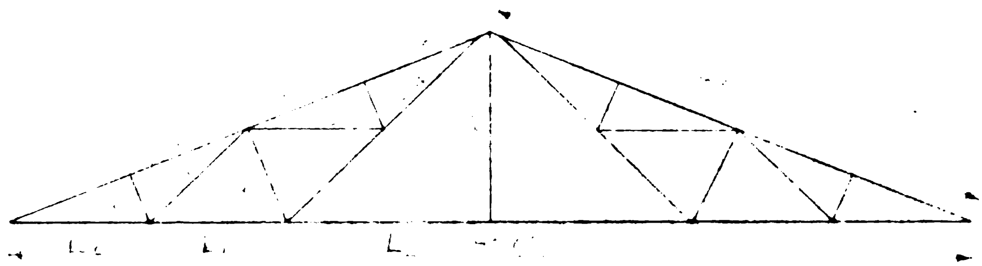
10.8 + 13.1 + 10 = 33.9 p.s.f.

Dead + $\frac{1}{2}$ Wind + Snow

10.8 + 6.6 + 20 = 37.4 p.s.f.

Therefore 40 p.s.f. is a safe value to use.

ROOF TRUSS #1



STRESS DIAGRAM

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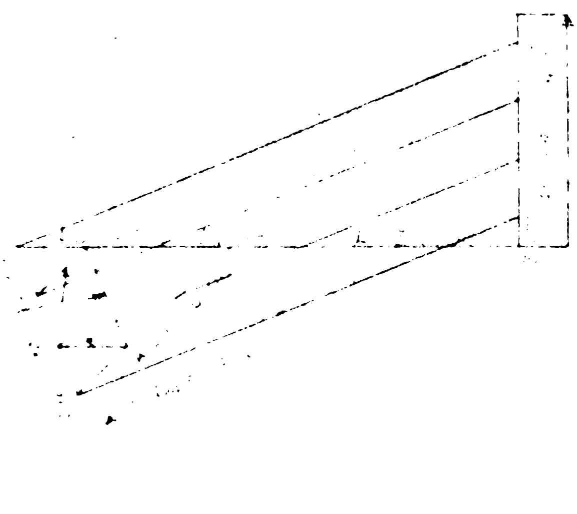

DESIGN OF MEMBERS

Sketch 2

Top Chord

Length = 10' $9\frac{1}{4}$" = 129.25"

Try 4.0" member

$$\frac{L}{d} = \frac{129.25}{3.625} = 35.7$$

$$\text{Working stress} = C\left(1 - 1/3\left(\frac{L}{K_2 d}\right)^4\right)$$

$$K = 0.64\left(\frac{2.5E}{C}\right)^{\frac{1}{2}} = 0.64\left(\frac{2.5 \times 1{,}600{,}000}{1450}\right)^{\frac{1}{2}} = 33.7$$

$$\text{Working Stress} = 1450\left(1 - 1/3\left(\frac{129.25}{(35.7)(3.625)}\right)^4\right) = 845 \text{ p.s.i.}$$

$$\text{Area required} = \frac{60300}{845} = 71.5 \text{ sq. in.}$$

Two 4 x 12" timbers with a cross sectional area of 2(41.69) = 83.38 sq. in. will satisfy requirements.

Bottom Chord - Lo - L_1 - L_2

$$\text{Area required} = \frac{56000}{1900} = 29.6$$

Two 4 x 5" timbers with a cross sectional area of 2(16.77) = 33.54 sq. in. will satisfy requirements, but further investigation shows that the joint between L_1 and L_2 requires a larger width. Therefore two 4 x 8" timbers are used.

Web Members - Compression

Select a 4.0" timber for web members in compression.

V_2 length = 8' - 7 3/8" = 103.375"

$$\frac{L}{d} = \frac{103.375}{3.625} = 28.5$$

$$K = 0.64\left(\frac{1{,}600{,}000}{1450}\right)^{\frac{1}{2}} = 21.3$$

$$\frac{P}{A} = \frac{0.274E}{\left(\frac{L}{d}\right)^2} = \frac{0.274 \times 1{,}600{,}000}{(28.5)^2} = 543 \text{ p.s.i.}$$

$$\text{Area required} = \frac{11{,}900}{543} = 22.0 \text{ sq. in.}$$

A 4 x 8" member with a cross sectional area of 2719 sq. in. will satisfy the requirements, but further investigation shows that joint C requires a 12" width. Therefore a 4 x 12" member is used.

V_1 and V_3 length = 4.0' 3 11/16" = 51.688"

Use 4.0" member

$\frac{L}{d} = \frac{51.688}{3.625} = 14.25$

K greater than $\frac{L}{d}$ greater than 11.0, therefore the intermediate column formula is used.

$\frac{P}{A} = 1450 \left(1 - 1/3 \left(\frac{14.25}{21.3}\right)^4\right) = 966$ p.s.i.

Area required = $\frac{5950}{966}$ = 6.16 sq. in.

A 4 x $2\frac{1}{2}$" member with a cross sectional area of 7.70 will satisfy the requirement. Four inch split rings are to be used at the joints and a minimum of $5\frac{1}{2}$" width is required. Therefore a 4 x 6" member is used.

Web Members - Tension

Maximum tension load on D_4

Area required = $\frac{24000}{1900}$ = 12.6 sq. in.

Two 2 x 6" members with a cross sectional area of 2(9.14) = 18.28 will satisfy this requirement but in order to eliminate fillers and to provide enough thickness for split rings, two 3 x 8" members are used for D_3 and D_4.

D_1 and D_2

Area required = $\frac{8000}{1900}$ = 4.22 sq. in.

At least 6.0" members are required for 4.0" split rings, therefore two 2 x 6" members are used for D_1 and D_2.

DESIGN OF JOINTS

Sketch 3

Joint "A"

The load on D_1 and D_2 acts at an angle of 20.2° with the chord U_1 - U_2. The allowable load on one ring = 4400#.

$\frac{8000}{4400} = 1.8$

Therefore two rings are required, one in each member.

The standard edge distance = 3 7/16".

$\frac{8000}{2 \times 4400}$ = 91% of full load developed.

Therefore the edge distance can be reduced to 2 3/4".

The edge distance furnished by a 6.0" members = 2 13/16".

The load on V_2 acts an angle of 90° to U_1 - U_2.

The allowable load on one ring = 3750#.

$\frac{11900}{4700} = 2.54$

Therefore three rings are required.

There is not enough room for three rings, so two rings and four 3/4 x 15½" machine bolts are used.

The allowable load on bolts = 700 x 4 = 2800#.

2800 + 2(4700) = 12200#.

Therefore four bolts and two rings are sufficient.

Minimum bolt spacing = 4d = 3.0"

Spacing between rows = 5d = 3 3/4".

Joint "B"

D_4 - U_3

Angle of load to grain = 22°

Allowable load on one ring = 6200#

$\frac{24000}{6200}$ = 3.87 rings are required - 6.0 rings used.

o/o of capacity developed = $\frac{24000}{6 \times 6200}$ = 64.5 o/o

Standard spacing parallel to grain in D_4 = 9.0"

Reduced spacing = 5.0"

Spacing used = 7.0"

Edge distance required = 2 3/4" - 3 3/4" used

End distance required = 3 3/4" - $5\frac{1}{2}$" used

U_3 to splice plates

Angle of load to grain = 22^{o}

Allowable load on one ring = 6200#

$\frac{32000}{6200}$ = 5.2 rings required - 8.0 rings used

o/o of capacity developed = $\frac{32000}{8 \times 6200}$ = 64.5 o/o

Standard spacing parallel to grain in D_4 = 9.0"

Reduced spacing = $\frac{64.5 \times 2 - 100}{2}$ = 36 o/o of capacity for one group.

50 o/o reduction allows 4 7/8" minimum

Use 7.0" spacing

Edge distance required = 2 3/4" - 2 3/4" used.

End distance required = 3 3/4" - $5\frac{1}{2}$" used.

Joint "C"

V_2 to $L_1 - L_2$

Angle of load to grain = 67.5^{o}

Allowable load on one ring = 4900#

$\frac{11900}{4900}$ = 2.43 rings required - 4 rings used

o/o of capacity developed = $\frac{11900}{4(4900)}$ = 61 o/o

Reduced edge distance = 2 3/4"

Spacing required parallel to grain = $5\frac{1}{2}$"

End distance required = $3\frac{1}{2}$"

D_3 to $L_1 - L_2$

Angle of load to grain = 45^{o}

Allowable load on one ring = 5350#

$\frac{16000}{5350}$ = 2.98 rings required - 4 rings used

o/o of capacity developed = $\frac{16000}{4(5350)}$ = 75 o/o

Reduced edge distance = 2 3/4"

Spacing required parallel to grain = $5\frac{1}{2}$"

End distance required = $3\frac{1}{2}$"

Length of Members

U_o, U_1, U_2, U_3 = 10' - $9\frac{1}{4}$"

L_o, L_1 = 11' - 7 3/16"

L_2 = 16' - 9 5/8"

V_1 = 4' - 3 11/16"

V_2 = 8' - 7 3/8"

V_3 = 4' - 1 7/8"

D_1 = 11' - 7 3/16"

D_2 = 10' - 7 7/16"

D_3 = 11' - 7 7/8"

D_4 = 10' - 10 3/16"

MATERIALS REQUIRED FOR TRUSSES

Building 120.0' long

Nine trusses @ 16.0' C.C. required

Lumber

36 pieces 2" x 6" x 12'

36 pieces 2" x 6" x 14'

36 pieces 2" x 6" x 16'

72 pieces 3" x 8" x 14'

18 pieces 3" x 8" x 20'

9 pieces 4" x 8" x 12'

18 pieces 4" x 6" x 16'

54 pieces 4" x 8" x 14'

45 pieces 4" x 8" x 18'

18 pieces 3" x 12" x 16'

9 pieces 3" x 12" x 10'

18 pieces 4" x 12" x 14'

81 pieces 4" x 12" x 16'

27 pieces 4" x 12" x 18'

18 pieces 10" x 12" x 18'

18 pieces 4" x 12" x 6'

Total = 22,230 FBM

Hardware

2736 Teco split rings 4"

144 Teco shear plates 3 1/8"

72 Machine bolts $\frac{1}{2}$" x 12"

72 Machine bolts 3/4" x $6\frac{1}{2}$"

216 Machine bolts 3/4" x $12\frac{1}{2}$"

126 Machine bolts 3/4" x $15\frac{1}{2}$"

396 Machine bolts 3/4" x $17\frac{1}{2}$"

144 Machine bolts 3/4" x 18"

36 Machine bolts 3/4" x 23"

36 Machine bolts 3/4" x 24"

144 Machine bolts 3/4" x 10"

9 Steel plates $\frac{1}{2}$" x 3" x 10 7/8"

9 Steel plates $\frac{1}{2}$" x 3" x 12"

36 Steel plates $\frac{1}{2}$" x 20" x 16"

36 Steel plates 3/8" x 10" x 12"

36 Angles 4" x 7" x 3/8", 12" long

9 Threaded rods 17' - 3" long

144 Washers 2"

2124 Plate washers 3" x 3" x 3/16"

Check bearing on wall

Total roof load = 80 x 120 x 40 = 384,000#

Wall load = 192,000#

18 pieces 4" x 12" x 14'

81 pieces 4" x 12" x 16'

27 pieces 4" x 12" x 18'

18 pieces 10" x 12" x 18'

18 pieces 4" x 12" x 6'

Total = 22,230 FBM

Hardware

2736 Teco split rings 4"

144 Teco shear plates 3 1/8"

72 Machine bolts ½" x 12"

72 Machine bolts 3/4" x 6½"

216 Machine bolts 3/4" x 12½"

126 Machine bolts 3/4" x 15½"

396 Machine bolts 3/4" x 17½"

144 Machine bolts 3/4" x 18"

36 Machine bolts 3/4" x 23"

36 Machine bolts 3/4" x 24"

144 Machine bolts 3/4" x 10"

9 Steel plates ½" x 3" x 10 7/8"

9 Steel plates ½" x 3" x 12"

36 Steel plates ½" x 20" x 16"

36 Steel plates 3/8" x 10" x 12"

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126 Machine bolts 3/4" x 15½"

396 Machine bolts 3/4" x 17½"

144 Machine bolts 3/4" x 18"

36 Machine bolts 3/4" x 23"

36 Machine bolts 3/4" x 24"

144 Machine bolts 3/4" x 10"

9 Steel plates ½" x 3" x 10 7/8"

9 Steel plates ½" x 3" x 12"

36 Steel plates ½" x 20" x 16"

36 Steel plates 3/8" x 10" x 12"

36 Angles 4" x 7" x 3/8", 12" long

9 Threaded rods 17' - 3" long

144 Washers 2"

2124 Plate washers 3" x 3" x 3/16"

Check bearing on wall

Total roof load = 80 x 120 x 40 = 384,000#

Wall load = 192,000#

Load per lineal foot $= \frac{192,000}{120} = 1600$ p.f.

Gross area of 8" x 8" x 16" concrete block = 15.75 x 7.75 =

122 sq. in.

$122 \times \frac{12}{16} = 91.5$ sq. in. per lineal foot

$\frac{1600}{91.5} = 17.5$ p.s.i.

Therefore, the bearing on the top course is within the allowable.

Use 3/4" bearing plate over top course on E and W walls to

distribute truss load.

UPPER WALL DESIGN

Weight of 8" x 8" x 16" heavy bearing blocks + $\frac{1}{4}$" joints = 50#

14.0' wall height = 168"

$\frac{168}{8} = 21$ courses

120.0' wall length = 1440"

$\frac{1440}{16} = 90$ blocks

Wall area = 1680 sq. ft. requires 1890 blocks

1.125 blocks required per square foot of wall

North Wall

80.0' long	1 - 12' x 14' door
	2 - 10' x 20' windows

South Wall

80.0' long	1 - 12' x 14' door

East Wall

120.0' long	3 - 8' x 10' windows

West Wall

120.0' long	3 - 8' x 10' windows

	N.W.	S.W.	E.W.	W.W.
Area sq. ft.	552	952	1440	1440
No. Blocks	620	1080	1620	1620

Total Wt.#	63,000	63,000	94,500	94,500
(Assuming solid wall)				
Unit Wt.#/Lin.'	787	787	787	787

Check for bearing on bottom course - East and West Walls.

Live load - 1600

Dead load - 787

Total load - 2387#/Lin. ft.

$\frac{2387\#/\text{Lin. ft.}}{91.5 \text{ sq. in./ft.}} = 26.2$ p.s.i. O K

WINDOW LINTEL DESIGN

East and West Walls

Sketch 15

3 courses on top = 3 x 50 = 150#

$150 \times \frac{12}{16} = 113\#/\text{lin. ft.}$

Live load 1600#/lin. ft.

Dead load 113#/lin. ft.

Total load 1713#/lin. ft.

Maximum B.M. $= \frac{w\,l^2}{12} = \frac{1713 \times 100}{12} = 14{,}300^{1\#}$

Section modulus $= \frac{M}{f} = \frac{14{,}300 \times 12}{20{,}000} = 8.6$ in.^{3}

Use 2 angles 6 x 4 x 7/8"

Section modulus $= 2 \times 7.2 = 14.4$ in.^{3}

North Wall

Sketch 16

3 courses on top

$150 \times \frac{12}{16} = 113\#/\text{ft.}$

Max B.M. $= \frac{wl^2}{12} = \frac{113 \times (20)^2}{12} = 3770^{1\#}$

Section modulus $= \frac{3770 \times 12}{20{,}000} = 2.27$ in.^{3}

Use 2 angles 4 x 4 x $\frac{1}{2}$"

Section modulus = 2(2.0) = 4.0 $in.^3$

Doors

Use 4 x 4 x $\frac{1}{2}$" angles for door lintels.

DESIGN UPPER FLOOR SLAB

Sketches 4 and 5

Live load assumed 400 p.s.f. This figure used in order to satisfy future requirements to which building may be put.

Assume an 8.0" slab

$Wt. = 1.0 \times \frac{8}{12} \times 1 \times 150 = 100$ p.s.f.

L. L. = 400 p.s.f.

D. L. = 100 p.s.f.

T. L. = 500 p.s.f. or for a 1.0' section 500#/lin. ft.

Considering this a simply supported beam with a uniformly distributed load, $B.M. = \frac{w\ l^2}{8}$

$Max\ B.M. = \frac{w\ l^2}{8} = \frac{500 \times 64}{8} = 4000^{1\#}$

$V = \frac{w\ l}{2} = \frac{500 \times 8}{2} = 2000\#$

From table #2 RCDH

$d = 4\frac{1}{4}"$

Change slab thickness to 6.0"

TL = 475 p.s.f.

$BM = 3800^{1\#}$ $\quad d = 4\frac{1}{2}"$ $\quad V = 1900\#$

From table #1 RCDH

$a = 1.44 \quad As = \frac{M}{ad} = \frac{3800}{(1.44)(4.5)} = 0.587$ sq. in./ft. required

Use $\frac{1}{2}$" circular 4.0" spacing

Check shear

$v = \frac{V}{bjd} = \frac{1900}{12 \times 7/8 \times 4.5} = 41$ p.s.i. $\quad$ O.K.

Check Bond

$u = \frac{V}{\Sigma ojd} = \frac{1900}{4.7 \times 7/8 \times 4.5} = 103$ p.s.i. $\quad$ O.K.

Placement of negative moment steel.

Sketch 6

Reinforcing steel of the same size and placing as that used for tension will be placed over supports; the length to be $\frac{1}{4}$ clear span each side of support.

80.0" - 10.0" = .70.0"

$\frac{70.0}{4} = 17.5$ Use 18.0" each side.

T - Beam Design

Sketch 7

Span 20.0' cc

Assume clear span 19.0'

LL =	475 x 6.67 =	3170#
DL = Weight of stem assumed =		230#
TL =		3400#/'

$V = \frac{W\,l}{2} = \frac{3400 \times 19}{2} = 32{,}300\#$

v allowable with web reinforcement = 180 p.s.i.

$v = \frac{V}{bjd}$ $\quad\quad$ $bd = \frac{32{,}300}{180 \times 7/8} = 205$ sq. in.

Let b = 10.0"

d = 23.0"

d + 3 = 26.0"

26 - t = 20.0"

Check weight of stem

$\frac{9.5 \times 20}{144} \times 150 = 198\#$

Assume j = 0.875

$BM = \frac{w\,l^2}{10} = \frac{3400 \times (19)^2 \times 12}{10} = 1{,}470{,}000''\#$

BM = Tjd

$T = \frac{1{,}470{,}000}{0.875 \times 23} = 72{,}900\#$

$As = \frac{72,900}{20,000} = 3.65$ sq. in./ft. required

Use 4 - 1" squpre bars

Area = 4.0 sq in. two rows

Check bond

$$u = \frac{V}{\Sigma ojd} \quad \frac{32,300}{16\ (.875)\ 23} = 101 \text{ p.s.i.}$$

Check fiber stresses

Sketch 8

$\frac{1}{4}$ of span $= \frac{20 \times 12}{4} = 60.0"$ <u>Least</u>

$(8 \times 6)\ 2 + 10 = 106.0"$

$(80.0" - 10") \times \frac{1}{2} = 35"$

$35 \times 2 + 10 = 80"$

Moment NA $= (60 \times 6)\ (x - 3) = 40\ (23 - x)$

$X = 6.25"$

$\frac{.25}{6.25}\ fc = 0.04fc = Z$

$fc - Z = 0.96fc$

$23.0 - \frac{6.25}{3} = 20.92"$

Compression	C		Arm		Moment
$C_1 = 0.04fc \times 6 \times 60 =$	$14.4fc$	x	20	=	$288fc$
$C_2 = \frac{1}{2} \times 0.96fc \times 6 \times 60 =$	$\underline{172.8fc}$	x	20.92	=	$\underline{3620fc}$
$C_1 + C_2 = C$	$= 187.2fc$				$3908fc$

$3908fc = 1,470,000$

$fc = 377$ p.s.i.

$C = T = 377 \times 187.2 = 70,500\#$

$T = Asfs$

$fs = \frac{70,500}{4.0} = 17,600$ p.s.i.

Check T beams at support

Sketch 9

$(N - 1)\ As = 36$ sq. in.

$NAs = 40$ sq. in.

$10X = 10\ x$ sq. in.

$36\ (X - 3) + 10\ x\ (\frac{X}{2}) = 40\ (23 - X)$

$36X - 108 + 5X^2 = 920 - 40X$

$5X^2 + 76X = 1028$

$X = 8.6''$

$jd = 23.0 - \frac{8.6}{3} = 20.2$

$Cc = \frac{fc}{2}\ (10)\ (8.6) = 43.0fc$

$Cs = \frac{6.6}{8.6}\ fc\ (36) = 37.6fc$

$43.0fc \times 20.2 = 876fc$

$37.6fc \times 20.0 = 752fc$

$80.6fc\ (a) = 1628fc$

$a = 20.2$

$T = C = \frac{1,470,000}{20.2} = 72,700\#$

$fc = \frac{72,700}{80.6} = 901$ p.s.i. <u>O.K.</u>

$fs = \frac{72,700}{4.0} = 18,200$ p.s.i. <u>O.K.</u>

T Beam Web Reinforcement

Sketch 10

The maximum shear occurs at the ends when the entire span is loaded.

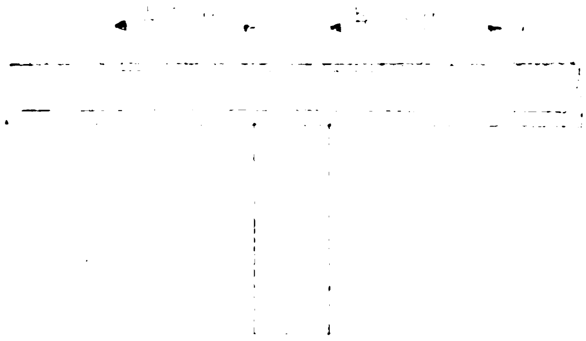
$v_{max} = \frac{32,300}{10 \times 0.92 \times 23} = 153$ p.s.i.

Center shear taken as 25 o/o of maximum.

$v_c = 153 \times .25 = 38.3$ p.s.i.

Shear taken by stirrups $= \frac{92.5 \times 93}{2} \times 10 = 43,000\#$

Use $\frac{1}{2}''$ circular stirrups.



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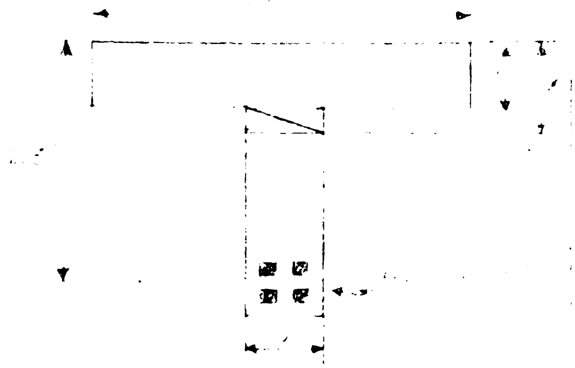

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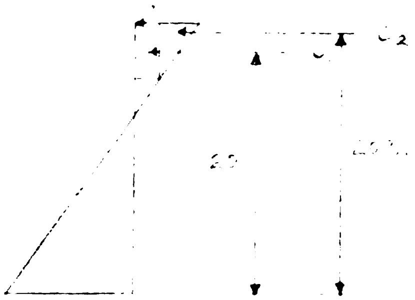

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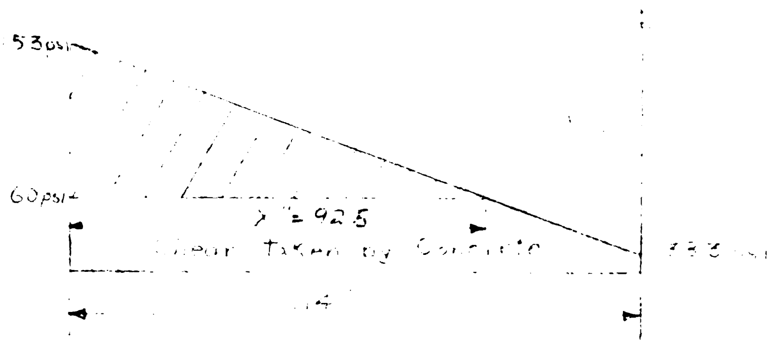


10

6-10

$A_s = 0.1963 \times 2 = .3926$ sq. in.

$16{,}000 \times .3926 = 6280\#$ taken by each stirrup

$\frac{43{,}000}{6280} = 6.85$ Use 7.

$\frac{d}{2} = \frac{23}{2} = 11.5''$

$\frac{93.0}{11.5} = 8.1$

Therefore 9 stirrups will be used from support to center.

2 @ 5"

7 @ 10"

GIRDER DESIGN

Sketch 11

Span 20.0'

Clear span = 18.5' assumed simply supported beam.

Assume stem weight = 500#/ft.

$B.M. = 64{,}600 \times 80 \times 2/3 + 1/10 \times 500 \times (18.5)^2 + 12$

$= 3{,}450{,}000 + 205{,}000$

$= 3{,}655{,}000''\#$

$V_{max.} = 64{,}600 + \frac{500 \times 18.5}{2}$

$= 69{,}225\#$

Assume $j = 0.875$ at support

$bd = \frac{V}{180j} = \frac{69{,}225}{180 \times .875} = 440$ sq. in.

Use $b = 15.0''$

$d = 32.0''$

$d + 3 = 35.0''$

$35 - 6 = 29.0''$

Check stem weight

$\frac{29 \times 15}{144} \times 150 = 455\#/'$

Assume $j = 0.92$ between supports

$B.M. = Tjd$

$$T = \frac{3{,}655{,}000}{0.92 \times 32} = 124{,}200\#$$

$$A_s = \frac{T}{f_s} = \frac{124{,}200}{20{,}000} = 6.21 \text{ sp. in.}$$

Use 1.0" circular bars

Use 8.0 bars - two rows - $3\frac{1}{2}$" spacing.

$A_s = 6.32"$

$o = 25.1"$

Bond

$$u = \frac{V}{\Sigma\, ojd} = \frac{69{,}225}{25.1 \times 0.92 \times 32} = 93.7 \text{ p.s.i.} \qquad \underline{\text{O.K.}}$$

Review of Girder Design

Sketch 12

$$\tfrac{1}{4} \text{ span} = \frac{20 \times 12}{4} = 60.0"$$

$8 \times \text{thickness} \times 2 + 15 = 111.0"$

Clear span $= 18.5 \times 12 = 222.0"$

Moment NA $= (60 \times 6)\ (X - 3) = 63.2\ (32 - X)$

$X = 7.36"$

$$\frac{Z}{.36} = \frac{f_c}{7.36}$$

$Z = 0.0488 f_c$

$f_c - Z = .95 f_c$

$C_1 = (60 \times 6)\ (.05 f_c) = 18.0 f_c \times 29 = 522 f_c$

$C_2 = (60 \times 6)\ \left(\frac{0.95 f_c}{2}\right) = 171 f_c \times 30 = 5130 f_c$

$C = C_1 + C_2 \qquad = 18.9 f_c \qquad = 5652 f_c$

$$f_c = \frac{3{,}655{,}000}{5652} = 646 \text{ p.s.i.} \qquad \underline{\text{O.K.}}$$

$T = C = 189.0 \times 646 = 122{,}200\#$

$$f_s = \frac{122{,}200}{6.32} = 19{,}500 \text{ p.s.i.}$$

Check girder over support

Sketch 13

Moment NA $= \left(\frac{X}{2}\right)(15X) + (56.9)\ (X - 2) = 63.2\ (32 - X)$

$X = 10.7''$

$Cc = \frac{fc}{2}(15)(10.7) = 80.3fc \times 28.2 = 2270fc$

$C_s = \frac{8.7}{10.7} fc\ (56.9) = \frac{46.4fc}{126.7fc} \times 30 = \frac{1390fc}{3660fc}$

$a = 28.9''$

$T = C = \frac{3,655,000}{28.9} = 126,500\#$

$fc = \frac{126,500}{126.7} = 1000$ p.s.i.

$fs = \frac{126,500}{6.32} = 20,000$ p.s.i.

Girder Web Reinforcement

Sketch 14

Maximum shear occurs at the end with the entire span loaded.

$v = \frac{69,225}{15 \times 0.92 \times 32} = 157$ p.s.i.

Shear at center taken as 25 o/o of max.

$157 \times 0.25 = 39.2$ p.s.i.

Sheer taken by stirrups

$15 \times \frac{97 \times 91.5}{2} = 66,600\#$

$\frac{66,600}{6280} = 10.6$ stirrups required.

Use 12 stirrups from support to center.

Spacing

4 @ 5.0"

4 @ 8.0"

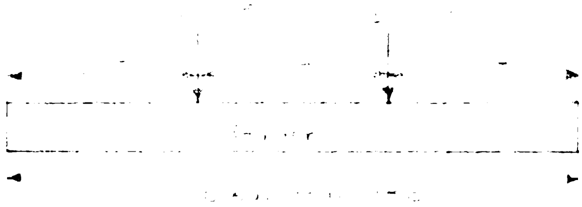
2 @ 10"

2 @ 12"

WALL BEAM DESIGN

Sketch 17

North and South wall beams will carry half the floor load of the of the other cross beams and no roof load. Therefore the floor



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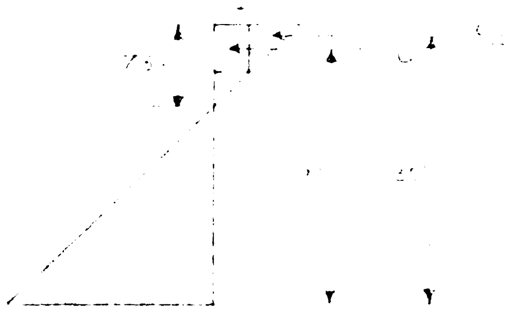

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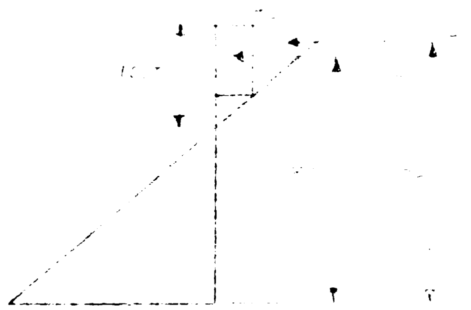

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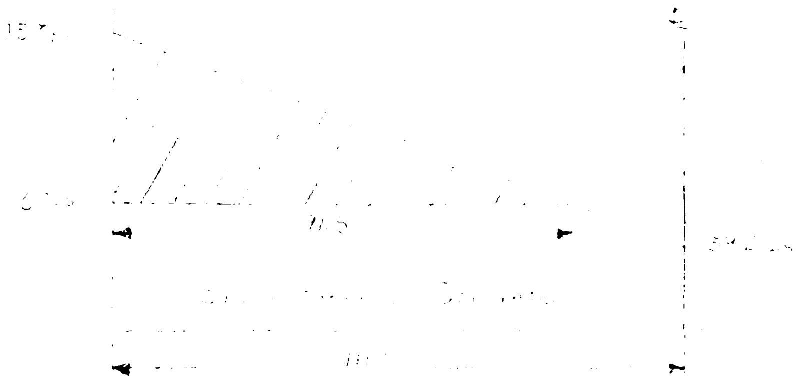


14

beam design will suffice for the North and South wall beams.

East and West Wall Beams

Assumed weight of stem = 500#/'

Wall load =1710#/'

2210#/'

$$\text{Max. } V = 32{,}300 + \frac{2210 \times 18.5}{2}$$

$$V = 52{,}800\#$$

$$\text{B.M.} = 32{,}300 \times 80 \times 2/3 + 1/10 \times 2210\,(18.5)^2 \times 12$$

$$= 2{,}707{,}000''\#$$

Since the B.M. and shear for the East and West wall beams are less than that used in the design of the interior girders, the previous girder design will be used for the East West wall beams.

COLUMN DESIGN

Sketches 18 and 19

All columns concentric - Axial loaded.

Columns 1, 7, 29, 35.

Sketch 20

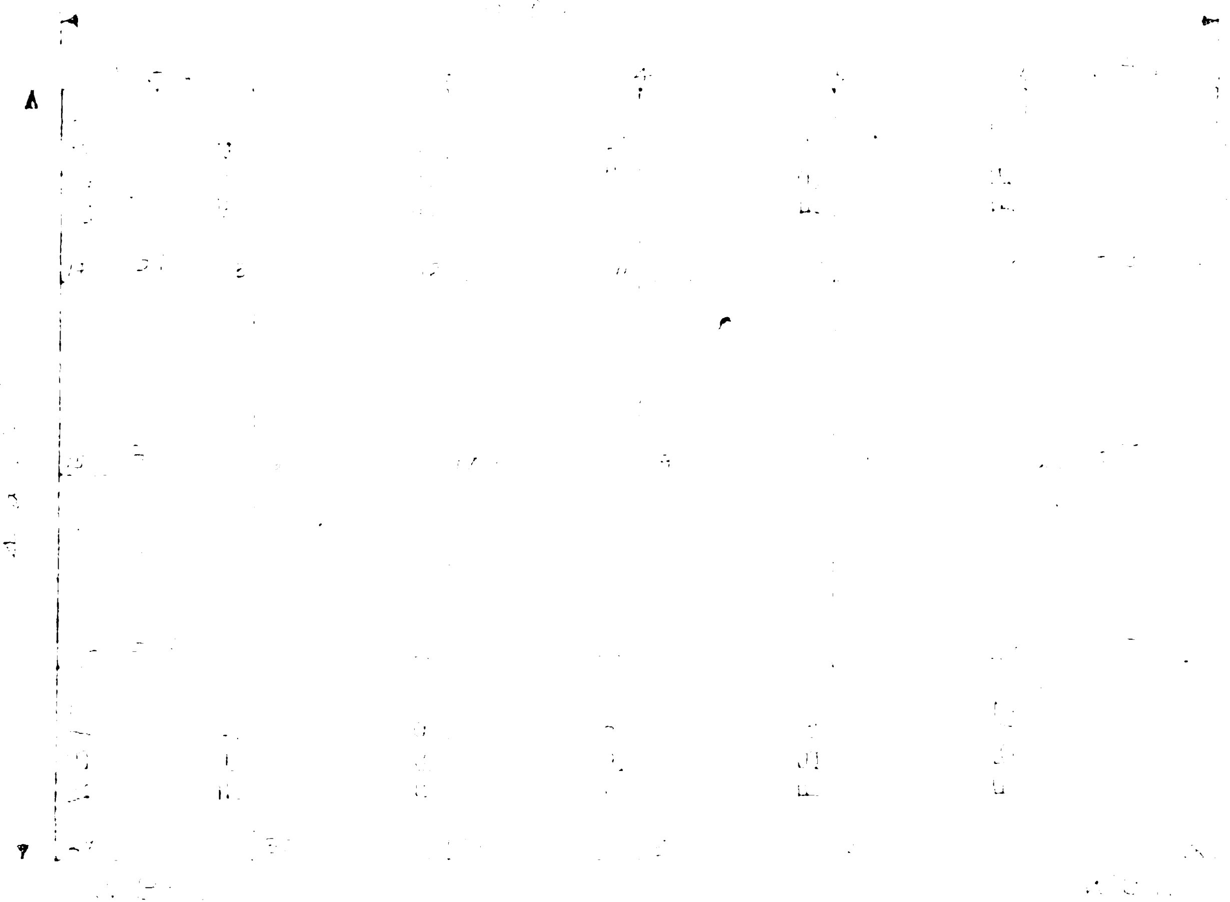
Maximum load will be on columns 29 and 35 due to the crane on the first floor.

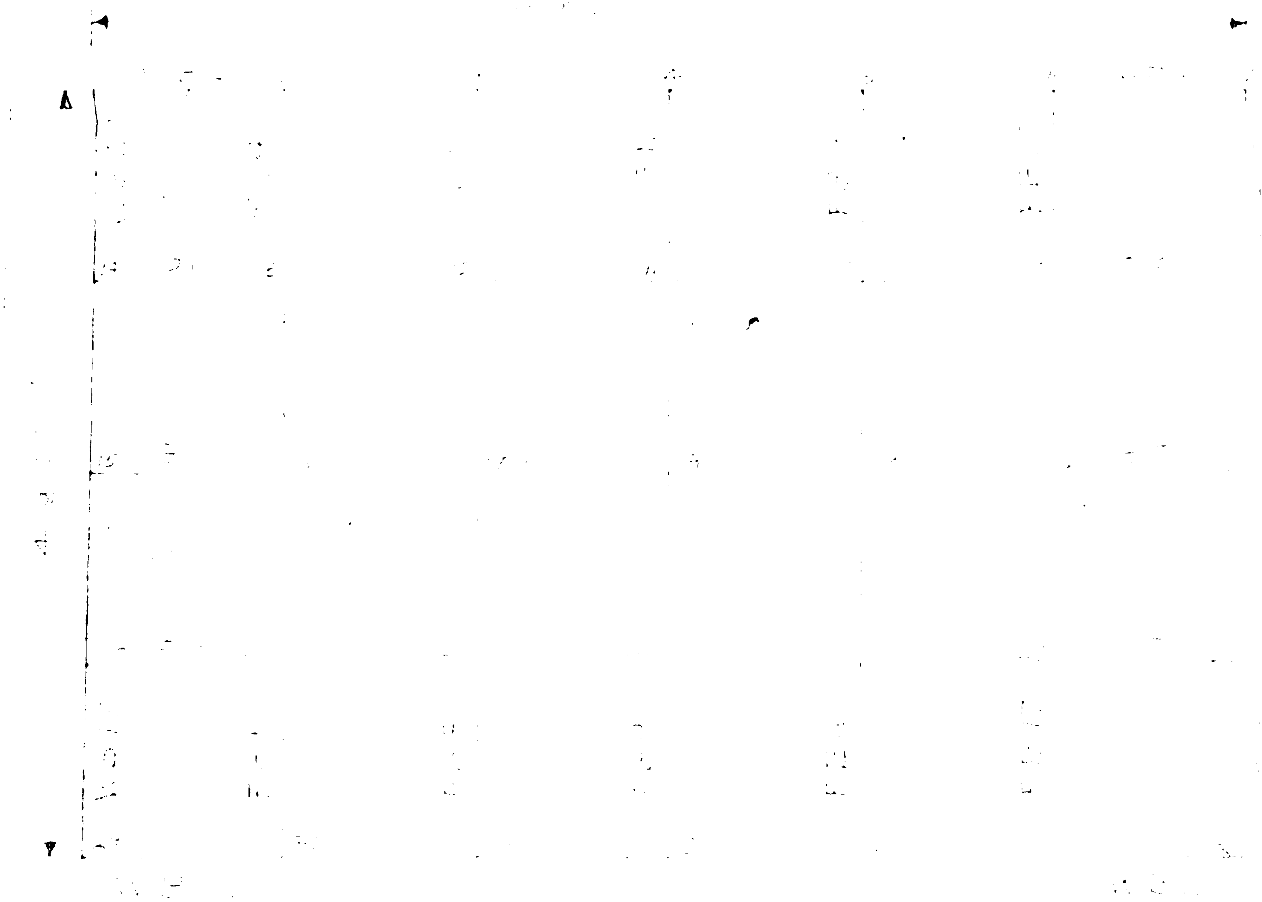
Girder load	= 69,225#
Floor beam load	= 32,300#
Crane load	= 3,000#
Column Wt.(assumed)	= 3,000#
N	=107,525#

From R.C.D.H. table #20

Use 10" x 14" column.

Load taken by concrete	= 95,000#
Load taken by steel	= 15,000#
Max. allowable load	=110,000#





Use 4 - 7/8" circular bars.

Tie spacing - Use the least of the following conditions.

1. 16 bar diameters = 14.0"
2. 48 tie diameters = 24.0"
3. least column **dimension** = 10.0" <u>Least</u>

Use ½" ties @ 10.0.

Check column weight

$$\frac{10 \times 14}{144} \times 14 \times 150 = 2100\# \qquad \underline{\text{O.K.}}$$

Columns 2, 3, 4, 5, 6, 30, 31, 32, 33, 34.

Sketch 21

Maximum load will be on columns 30, 31, 32, 33, and 34 due to the crane on the first floor.

Girder load = 2 x 69,225	=	138,450#
Floor beam load	=	32,300#
Crane load	=	3,000#
Column wt.(assumed)	=	3,500#
N	=	177,250#

Use 14" x 16" column.

Load taken by concrete	=	151,000#
Load taken by steel	=	30,000#
Max. allowable load	=	181,000#

Use 6 - 1" circular bars.

Use ½" ties @ 14.0".

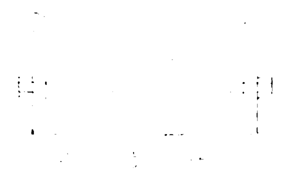
Check column weight.

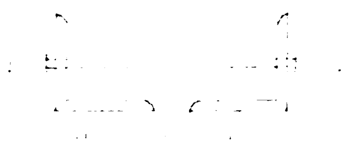
$$\frac{14 \times 16}{144} = 14 \times 150 = 3270\# \qquad \underline{\text{O.K.}}$$

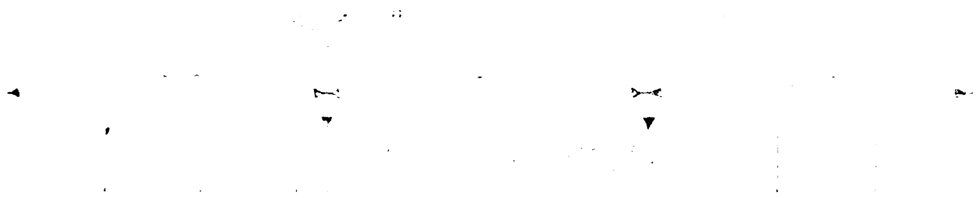
Columns 8, 14, 15, 21, 22, 28.

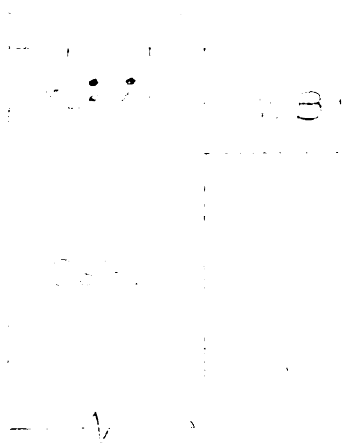
Sketch 22

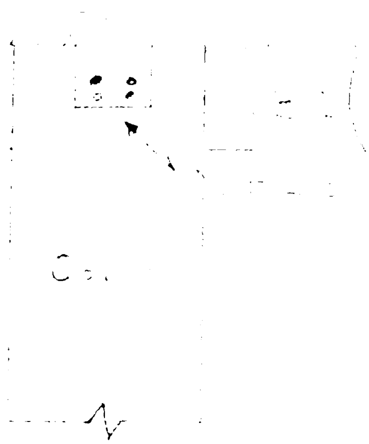
Maximum load will be on columns 22 and 28 due to the crane on the first floor.

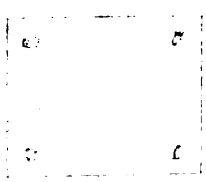


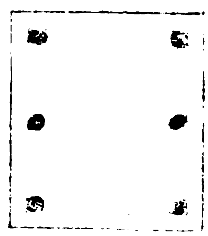












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Girder load	=	69,225#
Floor beam load = 2 x 32,300	=	64,600#
Crane load	=	3,000#
Column wt. (assumed)	=	3,000#
N		139,825#

Use 10" x 18" column.

Load taken by concrete	=	122,000#
Load taken by steel	=	19,000#
Max. allowable load	=	141,000#

Use 4 - 1" circular bars.

Use $\frac{1}{2}$" ties @ 10.0".

Check column weight.

$$\frac{10 \times 18}{144} \times 14 \times 150 = 2620\# \qquad \underline{\text{O.K.}}$$

Columns 9, 10, 11, 12, 13, 16, 17, 18, 19, 20, 23, 24, 25, 26, 27.

Sketch 23

Maximum load will be on columns 23, 24, 25, 26 and 27 due to the crane on the first floor.

Girder load = 2 x 69,225	=	138,450#
Floor beam load = 2 x 32,300	=	64,600#
Crane load	=	3,000#
Column wt. (assumed)	=	4,000#
N	=	210,050#

Use 16" x 18" column.

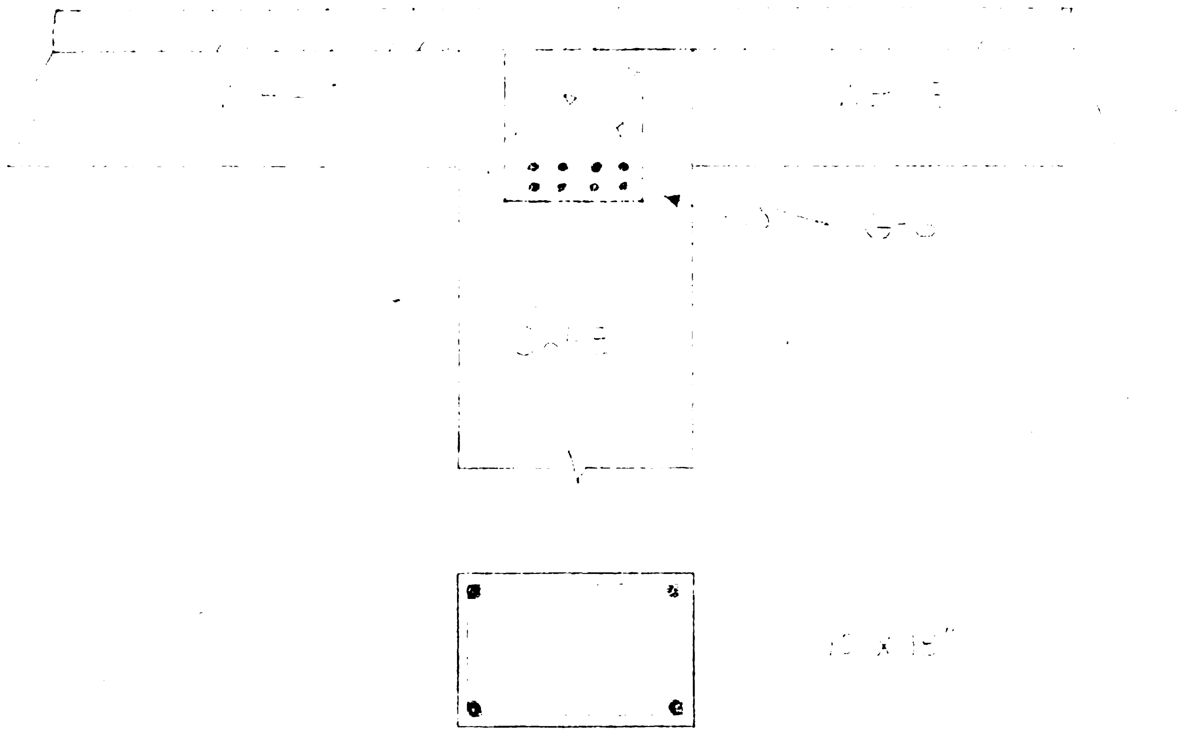
Load taken by concrete	=	194,000#
Load taken by steel	=	23,000#
Max. allowable load	=	217,000#

Use 6 - 7/8" circular bars.

Use $\frac{1}{2}$" ties @ 14.0".

Check column weight.

$$\frac{16 \times 18}{144} \times 14 \times 150 = 4200\# \qquad \underline{\text{O.K.}}$$



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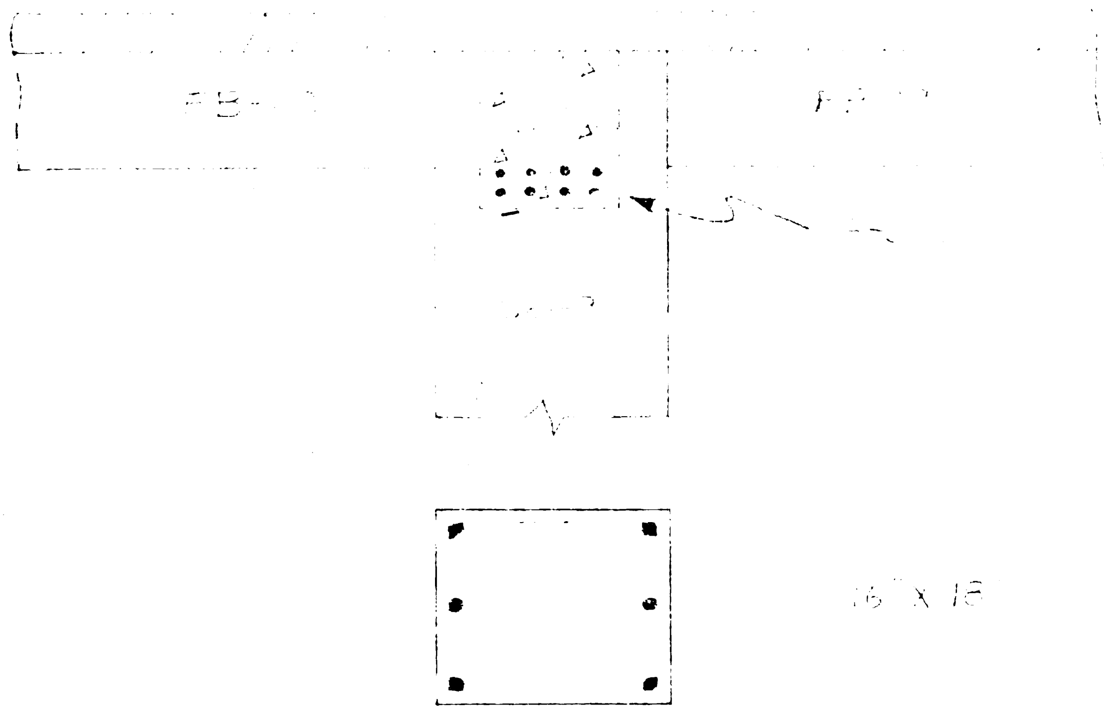


23

COLUMN FOOTING DESIGN

Sketch 24

Column footing 1.

Allowable soil pressure = 6000 p.s.f.

Footing weight will be assumed 6 o/o of live load.

Hooked, deformed bars will be used in all footings.

Column size 10" x 14"

LL = 107,525#

DL = 6,450#

TL = 113,975#

Area = $\frac{113,975}{6,000}$ = 19.0 sq. ft. required.

Use L = 4' - 6" A = 20.25 sq. ft.

Net pressure = $\frac{107,525}{20.25}$ = 5320 p.s.f.

B.M. = $5320 \left(\frac{14 \times 22}{144} \times \frac{22}{24}\right) + 5320 \left(\frac{(22)^2}{144} \times 0.6 \frac{22}{12}\right) = 30,000$'#

d = $\frac{M}{Kb} = \frac{30,000 \times 12}{236 \times 10}$ = 15.25" Use 16.0"

d = 16.0" h = 4.0 h + d = 20.0"

Check weight. 20.25 x $\frac{20}{12}$ 2 150 = 5060#

As = $\frac{M}{fsjd} = \frac{30,000 \times 12}{20,000 \times .866 \times 16}$ = 1.30 sq. in.

Use $\frac{1}{2}$" circular bars.

A = 0.2 sq. in.

$\frac{1.30}{0.2}$ = ~~6~~ 7 Use 7 bars @ 6.0" c.c.

Check Bond.

$u = \frac{V}{\Sigma ojd}$ = Neg pressure $\frac{(L^2 - (a+2d)^2)}{ojd} \times \frac{1}{4}$

$u = \frac{5320\ (20.25 - 12.25)}{11 \times .866 \times 16 \times 4}$ = 70 p.s.i. O.K.

Check shear.

$v = \frac{V}{bjd} = \frac{V}{4(a+2d)jd} = \frac{42,560}{168 \times .866 \times 16}$ = 18.3 p.s.i.

O.K.

Column footing 2.

Column size 14" x 16"

LL = 177,250#

DL = 10,650#

TL = 187,900#

Area = $\frac{187,900}{6,000}$ = 31.3 sq. ft. required

Use L = 5' - 8" A = 32.2 sq. ft.

Net pressure = $\frac{177,250}{32.2}$ = 5500 p.s.f.

B.M. = $5500(\frac{16 \times 27}{144} \times \frac{27}{24}) + 5500(\frac{(27)^2}{144} \times 0.6 \times \frac{27}{12})$ = 56,100'#

d = $\frac{56,100 \times 12}{236 \times 14}$ = 20.4" Use 21.0"

d = 21.0" h = 4.0" d+h = 25.0"

Check weight. $32.2 \times \frac{25}{12} \times 150$ = 10,100# O.K.

As = $\frac{56,100 \times 12}{20,000 \times .866 \times 21}$ = 1.85 sq. in.

Use 10 - $\frac{1}{2}$" circular bars @ 6.0" c.c.

Check bond.

u = $\frac{5500(32.2 - 21.8)}{15.7 \times .866 \times 21 \times 4}$ = 50 p.s.i. O.K.

Check shear

v = $\frac{5500 \times 10.4}{224 \times .866 \times 21}$ = 14.1 p.s.i. O.K.

Column footing 8.

Column size 10" x 18"

LL = 139,825#

DL = 8,400#

TL = 148,225#

Area = $\frac{148,225}{6,000}$ = 24.7 sq. ft. required.

Use L = 5.0' A = 25.0 sq. ft.

Net pressure $= \frac{139,825}{25} = 5590$ p.s.f.

B.M. $= 5590(\frac{18 \times 25}{144} \times \frac{25}{24}) + 5590 \; (\frac{(25)^2}{144} \times 0.6 \times \frac{25}{12}) = 48,500'\#$

$d = \frac{48,500 \times 12}{236 \times 18} = 13.7"$ Use 14.0"

$a = 14.0$ $h = 4.0"$ $h + d = 18.0"$

Check weight $25 \times \frac{18}{12} \times 150 = 5630\#$ O.K.

$As = \frac{48,500 \times 12}{20,000 \times .866 \times 14} = 2.40$ sq. in.

Use 13 - $\frac{1}{2}$" circular bars @ 4.0" c.c.

Check bond.

$$u = \frac{5590(25 - 10.1)}{20.4 \times .866 \times 14 \times 4} = 84.2 \text{ p.s.i.} \quad \text{O.K.}$$

Check shear.

$$v = \frac{5590 \times 14.9}{38 \times 4 \times .866 \times 14} = 45.2 \text{ p.s.i.} \quad \text{O.K.}$$

Column footing 9.

Column size 16" x 18"

LL = 210,050#

DL = 12,600#

TL = 222,650#

Area $= \frac{222,650}{6,000} = 37.1$ sq. ft. required

Use L = 6' - 2" A = 38.0 sq. ft.

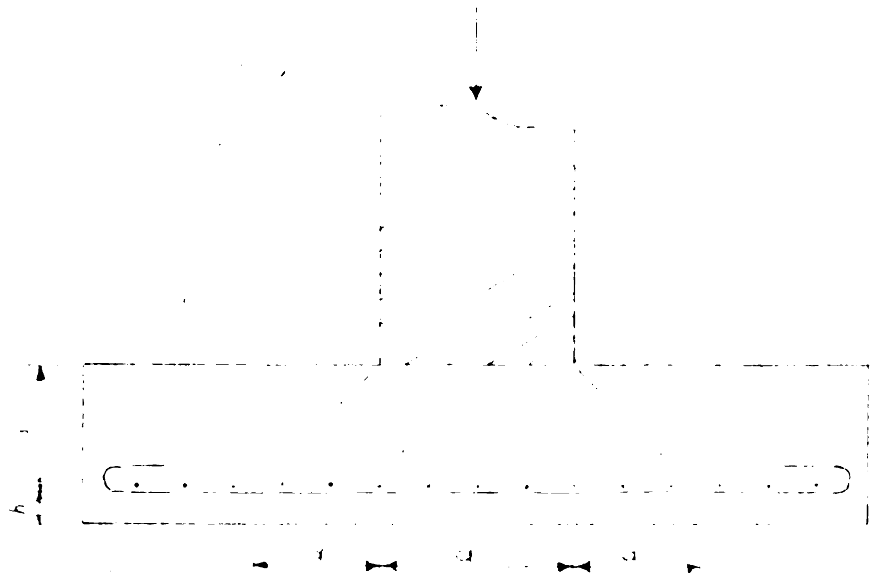
Net pressure $= \frac{210,050}{38.0} = 5520$ p.s.f.

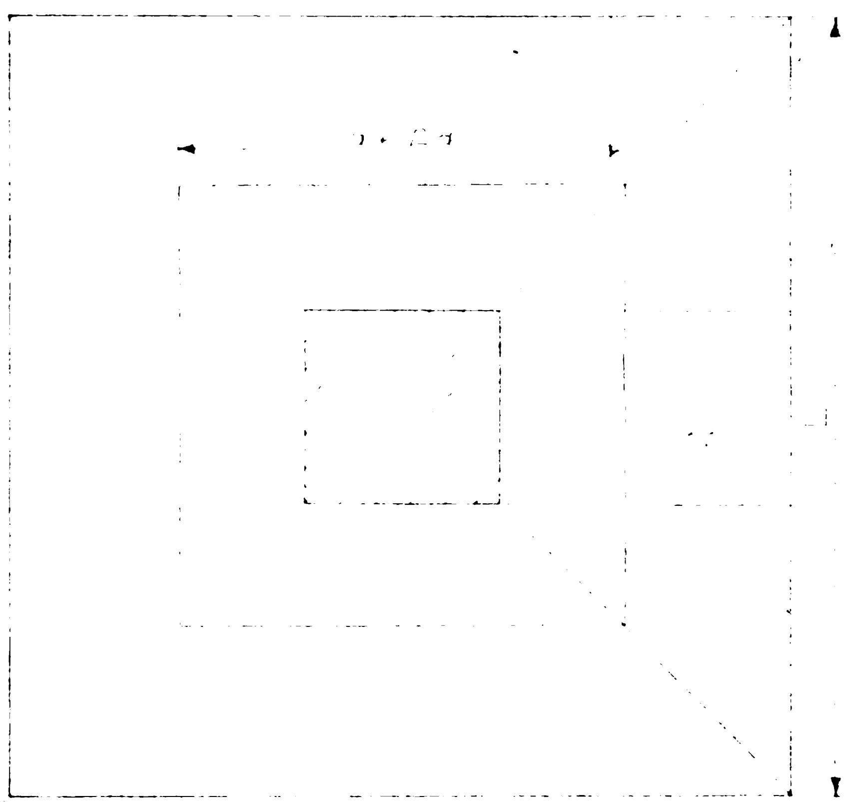
B.M. $= 5520(\frac{18 \times 29}{144} \times \frac{29}{24}) + 5520 \; (\frac{(29)^2}{144} \times 0.6 \times \frac{29}{12}) = 70,800'\#$

$d = \frac{70,800 \times 12}{236 \times 16} = 22.5"$ Use 23.0"

$a = 23.0"$ $h = 4.0"$ $h + d = 27.0"$

Check weight. $38 \times \frac{27}{12} \times 150 = 12,800\#$ O.K.





$$As = \frac{70,800 \times 12}{20,000 \times .866 \times 23} = 2.13 \text{ sq. in.}$$

Use 11 - $\frac{1}{2}$" circular bars @ 6.0" c.c.

Check bond.

$$u = \frac{5520(38 - 23.6)}{173 \times .866 \times 23 \times 14} = 45.7 \text{ p.s.i.} \quad \text{O.K.}$$

Check shear

$$v = \frac{5520 \times 11.4}{248 \times .866 \times 23} = 12.9 \text{ p.s.i.} \quad \underline{\text{O.K.}}$$

BASEMENT FLOOR DESIGN

Due to the expected use of the building the basement floor will be designed to carry heavier loads than the ground floor.

Max. wheel load expected = 6000#

Bearing area assumed = 10 sq. in.

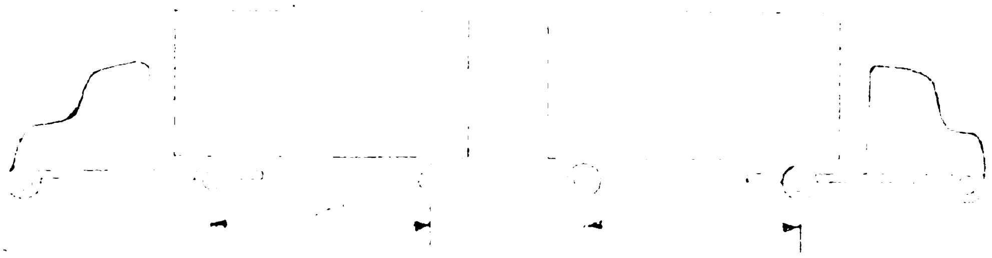
Unit pressure = 600 p.s.i. <u>O.K.</u> 1350 allowable.

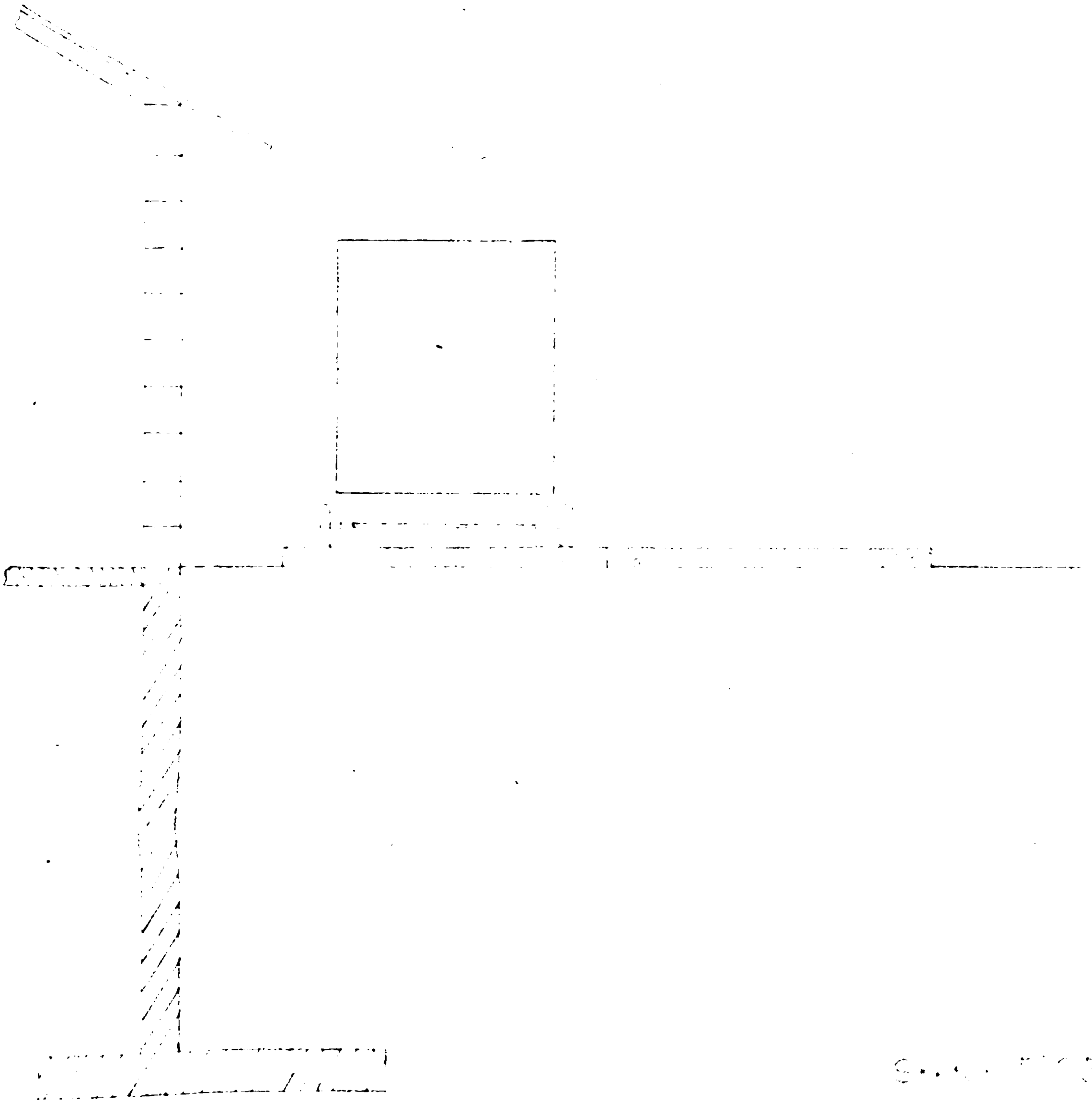
A subgrade of 6.0" of compacted stone will be prepared to receive the slab. A 6.0" plain concrete slab will be poured in alternate sections 20.0' x 20.0'. A $\frac{1}{2}$" filler will be used at the junction of the floor slab and all other members such as columns, walls, etc., and between the 20.0' sections.

RETAINING WALL DESIGN

Sketches 25, 26 and 27

Since the driveway is to be on the West side of the building the basement wall on that side will be a retaining wall in order to eliminate damage to the wall from driveway parking and traffic. The wall will be made 16.0' high, and 2.0' of the wall will be below grade as a precaution against frost damage. In order to compute the earth pressure against the wall, it is necessary to compute the equivalent surcharge for the maximum loads expected on the driveway. The largest vehicles expected to use the drive-





way are semi-trailers as shown in Sketch 25. The maximum condition expected is shown and the axle loads of 20,000# are considered to act uniformly on an area of 240 sq. ft.

Area = 40 x 6 = 240 sq. ft.

Unit load = $\frac{80,000}{240}$ = 333 p.s.f.

Weight of earth assumed 120 #/cu.ft.

$\frac{333}{120}$ = 2.78'   Use 3.0' surcharge.

The magnitude and point of application of the earth pressure will be determined by use of Rankine's Theory of Earth Pressure.

Sketch 26

Angle of repose for clay loam 36° 53'

$$P = \tfrac{1}{2}\, wh(h + 2h') \quad \frac{1 - \text{Sin } \phi}{1 + \text{Sin } \phi}$$

$$P = \tfrac{1}{2} \times 20 \times 16(16+6)\, \frac{1 - .602}{1 + .602} = 5300\#$$

$$Y = \frac{h^2 + 3\, h\, h^1}{3(h + 2\, h^1)} = \frac{(18)^2 + 3 \times 18 \times 3}{3(18+6)} = 6.75' \text{ from base}$$

Mo = 5300 x 6.75 = 35,800'#

To determine stem thickness at base

$$M = Rbd^2$$

$$d = \left(\frac{M}{Rb}\right)^{\frac{1}{2}} = \left(\frac{5300 \times 4.75 \times 12}{236 + 12}\right)^{\frac{1}{2}} = 10.5'' \quad \text{Use } 12.0''$$

$$d = \frac{V}{vjd} = \frac{5300}{12 \times .866 \times 60} = 8.52''$$

Stem thickness taken as 15.0". This will provide sufficient cover for stem steel. No batter will be used on the stem.

Design of base slab

The following dimensions will be assumed:

Toe = 3.0'

Heel = 5.0'

Thickness = 2.0'

$W = 120\ (5 \times 16) = 9{,}600 \times 6.75 = 64{,}800$

$W_1 = 1.25 \times 18 \times 150 = 3{,}380 \times 3.63 = 12{,}200$

$W_2 = 5 \times 2 \times 150 = 1{,}500 \times 6.75 = 10{,}150$

$W_3 = 3 \times 2 \times 150 = 900 \times 1.5 = 1{,}350$

$Rv = 15{,}380\#$ $Mr = 88{,}500'\#$

$88{,}500 - 35{,}800 = 52{,}700'\#$

$\frac{52{,}700}{15{,}380} = 3.43'$ from toe

Therefore, the Resultant acts within the middle third of the base.

$e = 4.63 - 3.43 = 1.2'$

$$S = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right) = \frac{15{,}380}{9.25}\left(1 \pm \frac{6 \times 1.2}{9.25}\right)$$

$= 2957$ p.s.f. max.

$= 367$ p.s.f. min.

Heel slab

Shear A-A				Moment A-A
9600		x 2.5	=	24,000
1500		x 2.5	=	3,750
+ 11,100#				+ 27,750'#
367 x 5	= 1835	x 2.5	=	4,590
1400 x 2.5	= 3500	x 1.67	=	5,840
	- 5335#			- 10,430'#

Shear A-A = 11,100 - 5335 = 5765#

Moment A-A = 27,750 - 10,530 = 17,320'#

$$d = \frac{V}{vjb} = \frac{5765}{80 \times .866 \times 12} = 9.25''$$

$$d = \left(\frac{M}{R}\right)^{\frac{1}{2}} = \left(\frac{17{,}320}{236}\right)^{\frac{1}{2}} = 8.58''$$

Base slab thickness changed to 16" (d = 12.0"). This will provide sufficient cover for steel.

$$As = \frac{17{,}320}{20{,}000 \times .866 \times 12} = 0.0832 \text{ sq. in./in.}$$

Use 1" circular bars @ 9.0" c.c. As = .0872 sq. in./in.

Steel placed in top of heel slab.

$$v = \frac{5765}{12 \times .866 \times 12} = 46.2 \text{ p.s.i.} \qquad \underline{\text{O.K.}}$$

$$u = \frac{46.2 \times 9}{3.14} = 132 \text{ p.s.i.} \qquad \underline{\text{O.K.}}$$

Toe slab

Shear B-B			Moment B-B
+ 900	x 1.5	=	+ 1350'#
2117 x 3 = 6351	x 1.5	=	9540
840 x 1.5 = 1260	x 1.0	=	1260
- 7611#			-10,800'#

Shear B-B = 7611 - 900 = 6711#

Moment B-B = 10,800 - 1350 = 9450'#

$$d = \frac{6711}{60 \times .866 \times 12} = 10.8"$$

$$d = \left(\frac{9450}{236}\right)^{\frac{1}{2}} = 6.34"$$

Toe slab will be made the same thickness as the heel slab 16.0" (d = 12").

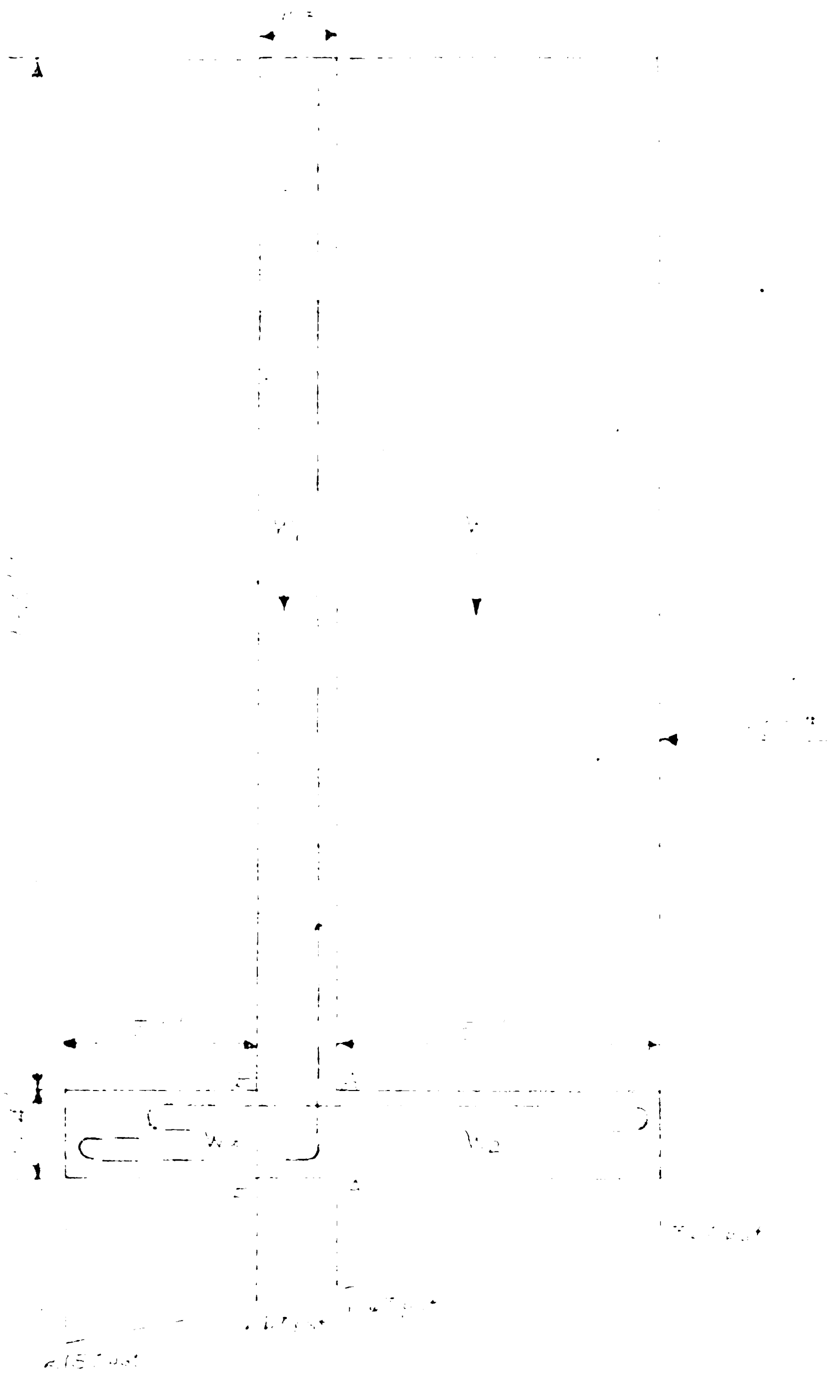
$$As = \frac{9450}{20{,}000 \times .866 \times 12} = 0.0454 \text{ sq. in./in.}$$

Steel should be placed in the bottom of the toe slab. Stem steel will be carried around for reinforcement in the toe slab. This will provide sufficient reinforcement for the toe slab and it will also provide enbedment length for the stem steel.

Stem

The stem will be 15.0" uniform thickness. Moments are computed for every 2.0' of stem from which the cut off points for the stem steel are determined.

h'	V#	M'#	As sq. in./ft.
2	80	53.5	0.00309
4	320	427	0.0205
6	720	1440	0.083



8	1280	3420	0.197
10	2000	6670	0.385
12	2880	11,550	0.666
14	3920	18,400	1.06
16	5300	28,300	1.64

Use 4 - 3/4" circular bars @ 3.0".

From this table the cutoff points can be determined. Sufficient length will be added for embedment.

$$L = \frac{f_c D}{4u} = 18.0"$$

For points of cutoff see Sketch 27.

In addition to the reinforcement steel, temperature steel will be placed in the stem, 0.15 o/o of cross section.

$A_s = 15 \times 12 \times .0015 = 0.27$ sq. in./ft.

Use $\frac{1}{2}$" circular bars @ 16" c.c. front = 0.15

Use $\frac{1}{2}$" circular bars @ 16" c.c. back = $\underline{0.15}$

0.30 sq. in./ft.

Factors of safety

$$\text{Overturning} = \frac{88{,}500}{35{,}800} = 2.47$$

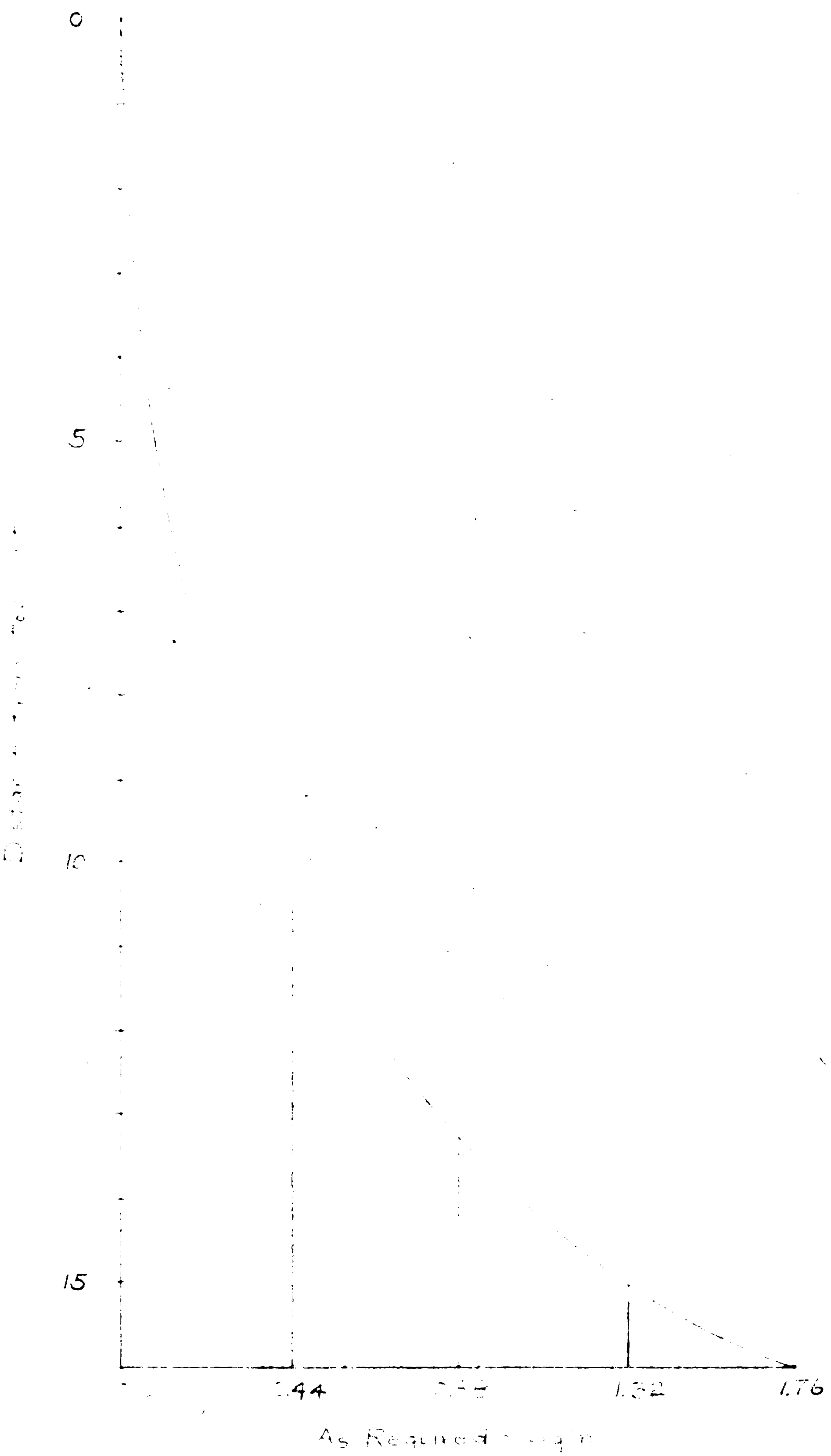
$$\text{Sliding} = \frac{15{,}380 \times \tan 25^\circ}{5300} = 1.36$$

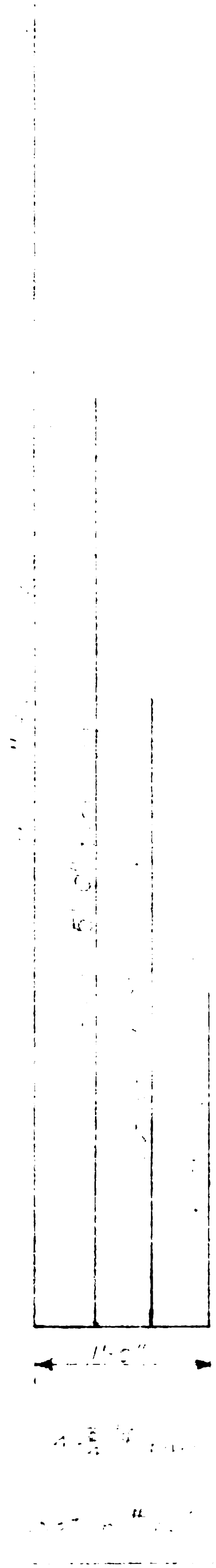
$$\text{Crushing} = \frac{6000}{2957} = 2.03$$

The retaining wall will be continuous for 60.0' sections. In order that the wall will frame into the other members, the wall will be poured flush with the outer face of the columns and wall beams. This will eliminate modification of the wall beams and columns along the West wall.

BASEMENT WALLS

The North, South and East basement walls will be poured concrete walls. Temperature steel will be placed in the walls, but no other


0
5
10
15
1.32
1.76



reinforcement will be used. The walls will be independent of the enclosing members such as the upper wall beams and columns. Therefore, it is safe to assume that there will be no transfer of loads from these members to the walls. The only direct load on the walls will be the earth pressure due to back fill.

The wall height will be 14.0' - 29" = 139" for the East wall and 14.0' - 20" = 148" for the North and South walls.

The wall thickness will be taken as 8.0" in order to provide a water-tight wall and to resist any stress caused by the back fill.

The only opening in the basement walls will be a door in the South wall. The door will be 14.0' high and 12.0' wide. Wall Beam #12 will serve as the top of the door hence no lintel is required.

Wall temperature steel

8.0 x 12 x .0015 = 0.144 sq. in./ft.

Use $\frac{1}{2}$" circular bars @ 16" c.c. in inside wall surface.

BASEMENT WALL FOOTINGS

For a 1.0' section of wall

Thickness = 8.0"

Height = 148.0"

Weight = $\frac{8 \times 148}{144} \times 1 \times 150$ = 1235#/ft.

Wall weight = 1235#/ft.

Footing weight (assumed) = 200#/ft.

T.L. = 1435#/ft.

Area = $\frac{1435}{6000}$ = .239 sq.ft. required.

Since such a small area is required, a practical size footing will be used without further investigation. Use a footing 2.0' wide and 8.0" thick. No reinforcement is necessary, but temperature steel will be used to prevent cracks from opening.

Use $\frac{1}{2}$" circular bars @ 9.0" c.c.

WALL POUR SCHEDULE

All columns and wall footings will be completed in one pour. Dowels of the same diameter as the vertical steel in the joining members will be placed in the footings. The dowels will extend at least 25 bar diameters beyond the footings. Keyways will be placed in the wall footings while the concrete is still in the plaster state. This will provide added anchorage for the walls to the footings.

The basement walls will be poured in two lifts. This is deemed necessary for proper puddling. The first lift willbe 8.0' high, and a keyway will be placed on the top surface of the first lift. The second lift will complete the wall to grade. Dowels will be used to anchor the wall to the floor slab. These dowels should extend 25 diameters into the wall and into the slab. This will necessitate bending the bars 90°. The temperature steel in the wall will extend 25 diameters into the joining columns in order to provide proper anchorage with these members.

The stem of the West wall, the retaining wall, will be poured in two lifts with the keyways provided in the same manner as for the other walls. The wall will be poured in two 60.0' sections with an expansion joint between the two sections. The expansion joint will be a keyway, doweled together, and with $\frac{1}{2}$" filler between the two sections. This type of joint should provide a flexible, water-tight connection. The stem will also be doweled to the West wall columns and wall beams.

STAIR DESIGN

Stairs located and dimensioned as shown in Sketch 28.

LL = 100 p.s.f.

DL = 75 p.s.f.

TL = 175 p.s.f.

Use 6.0" slab.

Total rise from top of floor to first landing - 7.0'.

Stairs with 10.0" run, 6" rise, and 1.0" nosing will be used.

Horizontal projection of stairway = 10.0'

Landing platform 4.0' long, 8.0' wide, 6.0" thick.

The stairway slab and platform slab will be designed as one slab, length = 14.0'.

$$V = \frac{wl}{2} = \frac{175 \times 14.0}{2} = 1225\#$$

$$B.M. = \frac{wl^2}{10} = \frac{175 \times (14.0)^2}{10} = 3430\#$$

$$d(\text{for sheer}) = \frac{1225}{12 \times .866 \times 60} = 1.97"$$

$$d(\text{moment}) = \left(\frac{3430 \times 12}{12 \times 236}\right)^{\frac{1}{2}} = 3.82"$$

Use d = 4.0"

$$As = \frac{3430 \times 12}{20{,}000 \times .866 \times 4.0} = 0.594 \text{ sq. in./ft.}$$

Use $\frac{1}{2}$" circular bars @ 4.0" c.c.

Check bond

$$u = \frac{1225}{4.7 \times .866 \times 4} = 75.2 \text{ p.s.i.} \qquad \underline{\text{O.K.}}$$

The lower and upper stairway slabs are fixed to the landing slab along the inside edge. The landing slab, 4.0' long, is fixed in the East wall. The stairway slab reinforcement is continued through the landing slab and anchored into the wall with hooks. This same reinforcement will be run along the 8.0' length of the landing slab.

A beam will be designed to support the inside edge of the platform slab. This beam will be supported by a short column at each end.

Stairway slab reaction = 1225 #/'

Platform slab reaction = $\frac{4 \times 75}{2}$ = 150 /'

Total load on landing beam = 1375 #/'

Assume stem weight = 50 #/'

T.L. = 1405#/'

$V = \frac{1405 \times 8}{2} = 5620\#$

With web reinforcement

$bd = \frac{5620}{180 \times .275} = 35.6$ sq. in. required

Let b = 6.0"

d = 9.0"

d/2.0 = 11.0"

d-t = 5.0"

Check stem weight

$\frac{5 \times 4}{144} \times 150 = 21.0\#$ <u>O.K.</u>

$B.M. = \frac{1405 \times 64}{8} = 11,250'\#$

$As = \frac{11,250 \times 12}{20,000 \times .92 \times 9} = 0.815$ sq. in. required

Use 5 - $\frac{1}{2}$" circular bars @ 2.0" c.c., two rows.

Check bond

$u = \frac{5620}{7.9 \times 0.92 \times 9} = 85.8$ p.s.i. <u>O.K.</u>

Check shear

Max. shear at end of beam = $\frac{5620}{6 \times .92 \times 9} = 113$ p.s.i.

Web reinforcement is required.

Max. center shear will be taken as 25 o/o of max. shear.

Vc = 0.25 x 113 = 38.3 p.s.i.

Total shear taken by stirrups

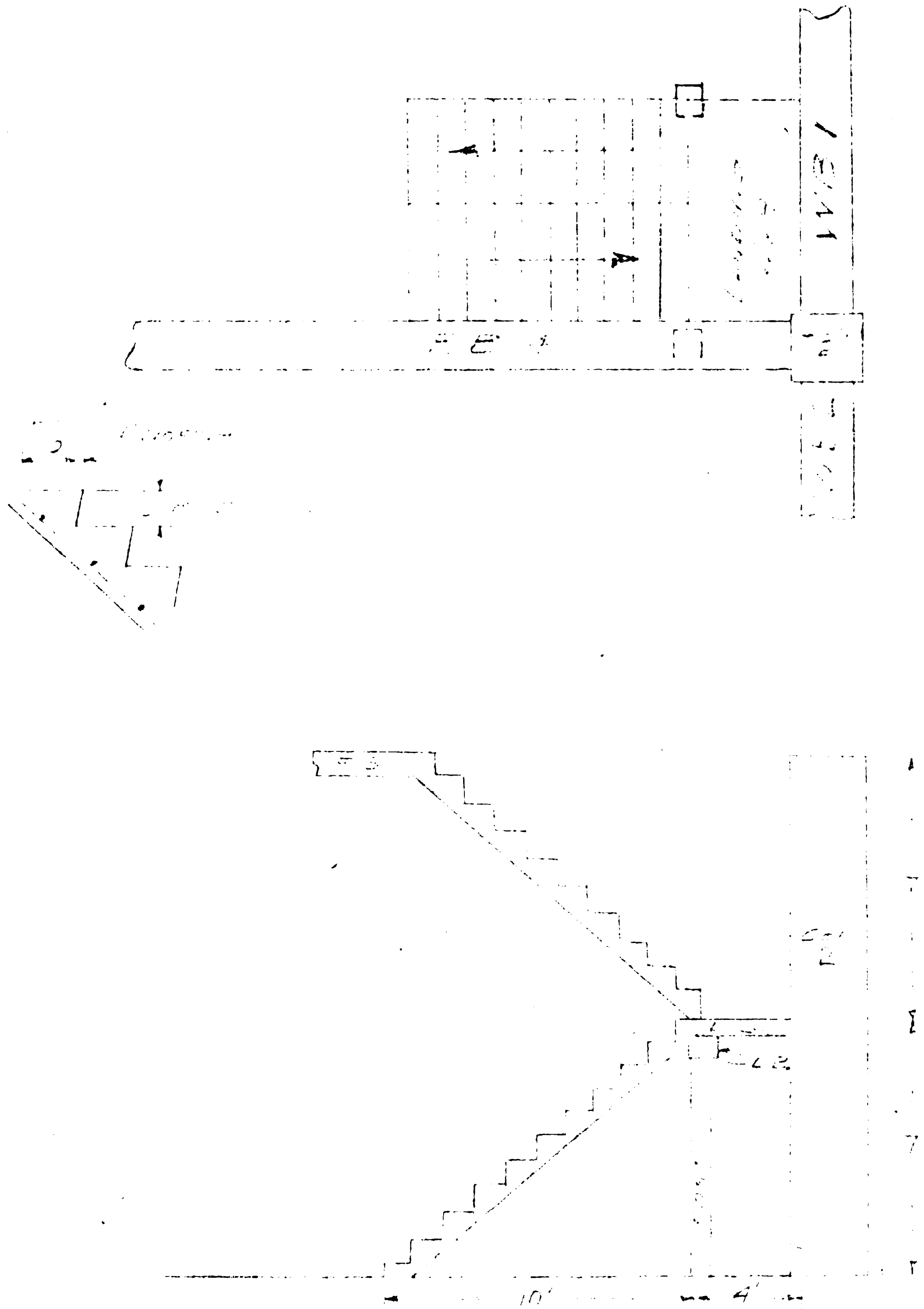
113 - 38.3 = 74.7 p.s.i.

113 - 60.0 = 53.0 p.s.i.

$\frac{74.7}{48} = \frac{53}{X}$ X = 34.0"

$V = \frac{53 \times 34}{2} = 900\#$

Using $\frac{1}{4}$" circular stirrups A = 0.05 sq. in.



SKETCH # 28

0.1 x 16,000 = 1600# taken by each stirrup

One stirrup required.

Max. apacing $= \frac{d}{2} = 4.5"$

Therefore $\frac{34}{4.5} = 7$

Use 8.0 stirrups @ 4.5" c.c. at each end.

Landing Post Design

Two 3" x 2 3/8" - 5.7# standard steel columns will be used.

Allowable concentric load for 7.0' height = 9900# <u>O.K.</u>

A 3/4" - 4 x 5 bearing plate will cap each end of the column .

Maximum beam reaction	=	5620#
Column weight = 7 x 5.7	=	40#
Total axial load	=	5660#

$\frac{5660}{4 \times 5} = 283$ p.s.i. compression stress upon floor. <u>O.K.</u>

Allowable compression stress = 1350 p.s.i.

Therefore no footing is required. The landing beam posts will be encased in concrete in order to provide a fireproof structure.

<u>CRANE SUPPORT DESIGN</u>

Sketch 29

A three ton overhead crane is to be placed on the first floor along the East wall. A four wheel crane will be used, crane to be designed by owner. The crane will operate on two rails which will be supported by steel columns. The columns will be embeded 12" into the concrete of the floor system, and the top ends of columns will be tied together by lateral bracing.

Six inches clearance will be allowed for the crane wheels and 2.0' clearance will be allowed for the crane mechanism.

Maximum load on the column will occur when one crane wheel is positioned over the column bracket. Due to insufficient data concerning the crane wheel spacing, the steel columns will be designed for the condition that one wheel is positioned over the bracket and the resulting moment will be computed for the weak axis of the column. The brackets will be placed so that the actual moment will be about the strong axis of the column.

A 5 x 5" - 18.5# light weight column will be checked for maximum moment about the weak axis.

Crane load = 1500#/wheel

Moment = 1.5 x 6 = 9.0" Kips.

Equivalent direct load = 9.0 x 1.56 + 1.5 = 15.36 Kips.

$\frac{fa}{FA} + \frac{fb}{FB}$ shall not exceed unity.

$\frac{l}{r} = \frac{144}{1.28} = 112.5$

FA = 10.81

$fa = \frac{P}{A} = \frac{15.36}{5.45} = 2.82$

FB = 20.0

$fb = \frac{M}{S} = \frac{9.0}{9.94} = 0.91$

$\frac{2.82}{10.81} \times \frac{0.91}{20.0} = 0.306$ less than unity O.K.

Allowable axial load for unsupported length of 12.0' equals 49.0 Kips. Therefore the column selected is adequate.

Bracket Design

From A.I.S.C. handbook

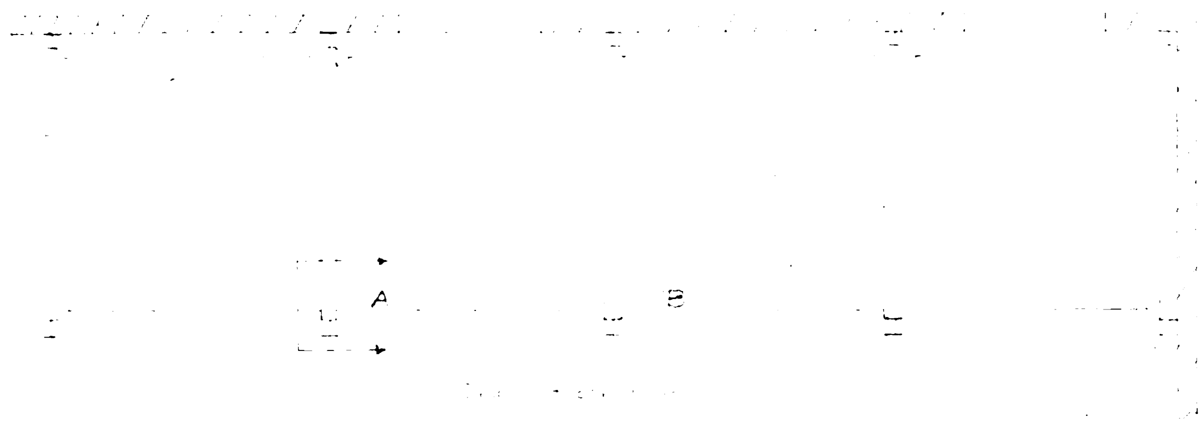
$P = 1.5^k$

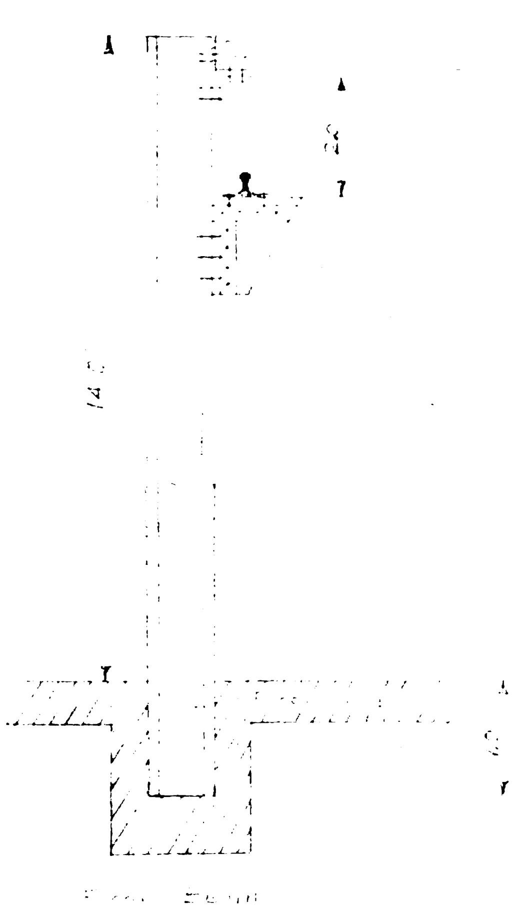
l = 6.0"

3/4" rivets

3.0" pitch

Allowable stress on one rivet = 15.0 x .4418 = 6.63 Kips.





Section-A

27

$$C = \frac{P}{2 \times S} = \frac{1.5}{2 \times 6.63} = 0.103$$

Use 3 rivets in each of two rows.

Use $\frac{1}{2}$" plate and 2 x 2 x $\frac{1}{2}$" angles for bracket.

The columns will be 15.0' long, 1.0' embeded in the floor system. The brackets will be placed 2.0' below the tops of the columns. The tops of the columns will be braced as shown in sketch 29. The columns will be tied together with 3 x 2 3/8" - 5.7# standard I bea s, and the top cross bracing will be 2 x 2 x 3/8" angles. Rail size and fastenings will be determined after the crane has been assigned.

DESIGN OF DRIVE

Sketch 30

A two lane concrete drive will be placed parallel to the West side of the building. The drive will be 3.0' from the building wall and will extend from the existing city street to the rear entrance of the building. The slab thickness will be determined by the use of Older's Theory.

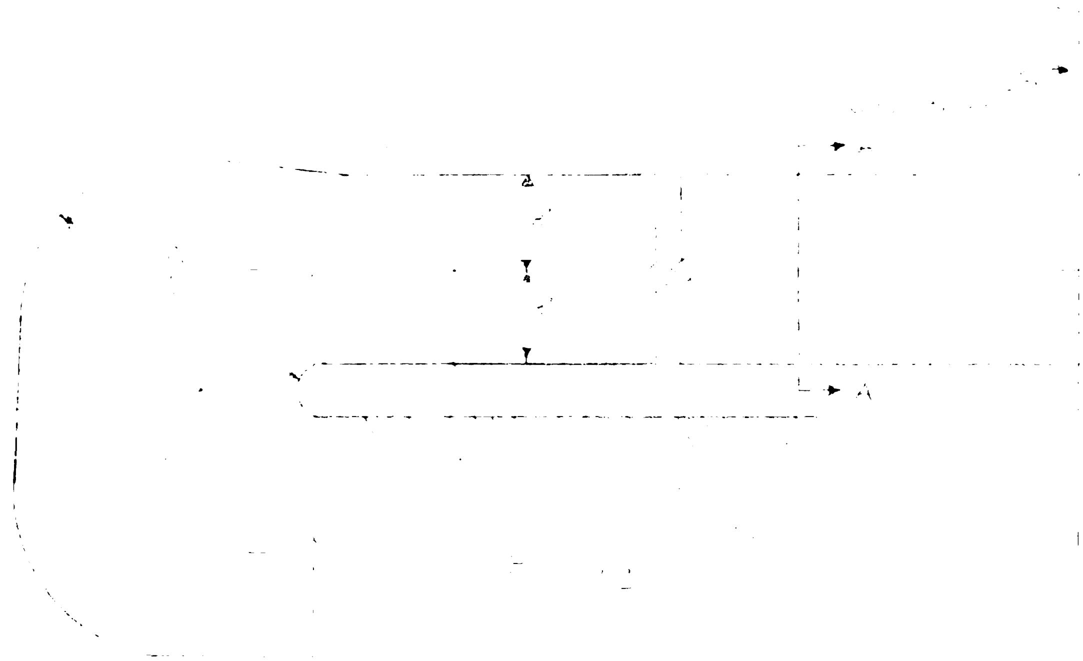
S = fluxural strength of concrete

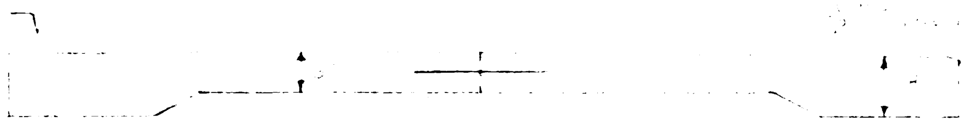
W = wheel load

d = slab thickness

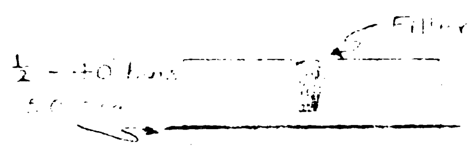
$$d = \left(\frac{1.5W}{S}\right)^{\frac{1}{2}} = \left(\frac{1.5 \times 10{,}000}{750}\right)^{\frac{1}{2}} = 4.47"$$

A 6.0" slab with thickened edges will be used. No reinforcement willbe placed in the slab. A transverse expansion joint will be placed at a point midway along the drive. Weakened plane contraction joints will be placed at points midway between the expansion joint and the ends of the slab. The total width of the drive will be 16.0', laid in two 8.0" slabs connected by a doweled longitudenal joint. A gutter will be placed in the outside edge of the left slab, in order to carry the run off from the slab away from the basement wall. The drive will be widened to 26.0' at the curve in order to facilitate entrance into the basement.

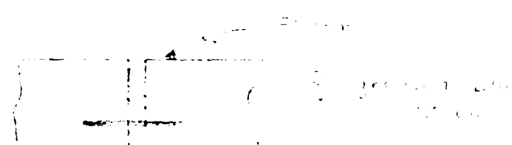




Section A-A



Contraction Joint



Expansion Joint

COST OF MATERIALS ESTIMATE

The unit prices for the construction materials used in this estimate were obtained from the May 1, 1947 issue of "Engineering News Record". The unit prices used are those listed therein for the Central Ohio area. Reference was also made to "Construction Estimates and Costs" by H.E. Pulver, published by McGraw-Hill Book Company in 1940.

Concrete Block Wall

4940 blocks @ \$0.17 = \$839.80

Cement Mortar. 1:4 mix used

0.25 cu. yd./100 blocks

49.40 x 0.25 = 12.35 cu. yds.

Cement

12.35 x 6.75 @ \$2.62/bbl. = \$ 54.70

Lime

12.35 x 70 @ \$30.00/Ton = \$ 13.00

Sand

12.35 x 1.0 @ \$1.15/Ton = \$ 21.10

Water

12.35 @ \$0.10/cu.yd. = \$ 1.30

Total Wall Cost = \$929.90

Concrete

3000# concrete will be used for all structures. The columns will be poured with a very plastic mix; 1:1 3/4: 2½ mix by volume, 6.0 gal./sack. The remaining sturctures will be poured with a 1:2:3 mix by volume, 6.0 gal/sack.

	S.G.	Wt.	Absolute Wt.
Cement	3.10	94#/sack	62.4 x 3.10 = 194#
Sand	2.65	110#/cu.ft.	62.4 x 2.65 = 165#
Gravel	2.65	100#/cu.ft.	62.4 x 2.65 = 165#
Water	1.00	62.4#/cu.ft.	

The yield will be computed for a 1.0 sack unit of each mix.

1:2:3 mix by volume.

Cement $\frac{94}{194}$ = 0.484 cu. ft.

Sand $\frac{110 \times 2}{165}$ = 1.333 cu. ft.

Gravel $\frac{100 \times 3}{165}$ = 1.818 cu. ft.

Water $\frac{6.0}{7.48}$ = 0.802 cu. ft.

Yield in concrete = 4.437 cu. ft.

1:1 3/4:2½ mix by volume. 6.0 gal./sack

Cement $\frac{94}{194}$ = 0.484 cu.ft.

Sand $\frac{110 \times 1.75}{165}$ = 1.167 cu. ft.

Gravel $\frac{110 \times 2.5}{165}$ = 1.515 cu. ft.

Water $\frac{6.0}{7.48}$ = 0.802 cu. ft.

Yield in concrete = 3.968 cu. ft.

First floor slab	4800 cu. ft.
T-Beams	2200 cu. ft.
Girders	1800 cu. ft.
Column footings	2400 cu. ft.
Basement floor slab	4800 cu. ft.
Basement walls	2200 cu. ft.
Retaining wall	3900 cu. ft.
Wall footings	400 cu. ft.
Stairway	75 cu. ft.
Driveway	1300 cu. ft.

Volume of 1:2:3 concrete = 23,875 cu. ft.

$\frac{23,875}{4.437}$ = 5380 units

Cement 1 x 5380 = 5380 sacks

Sand 1.333 x 5380 = 7170 cu. ft.

Gravel 1.818 x 5380 = 9780 cu. ft.

Columns 800 cu. ft.

800 cu. ft. of 1:1 3/4:2½ mix required

$\frac{800}{3.968} = 202$ units

Cement 1 x 202 = 202 sacks

Sand 1.167 x 202 = 236 cu. ft.

Gravel 1.515 x 202 = 306 cu. ft.

Total cement = 5582 sacks @ $ 2.62/bbl.	= $ 3660.00
Total sand = 7406 cu. ft. @ $ 1.15/Ton	= $ 407.00
Total gravel = 10,086 cu. ft. @ $ 2.50/Ton	= $ 1260.00
Total water = 6.0 x 5582 @ $ 0.15/1000 gal.	= $ 6.00
Total concrete cost	= $ 5333.00

Reinforcing Steel

Floor slab and stairs	30,000' - ½" Circular bars
T Beams	7,000' - 1" Square bars
Stirrups	2,520' - ½" Circular bars
Girders	4,800' - 1" Circular bars
Stirrups	6,000' - ½" Circular bars
Columns	1,800' - 7/8" Circular bars
	1,500' - 1" Circular bars
	2,800' - ½" Circular bars
Column footings	5,300' - ½" Circular bars
Retaining wall	1,040' - 1" Circular bars
	4,560' - 3/4" Circuler bars
	2,880' - ½" Circular bars
Basement wall	3,360' - ½" Circular bars
Wall footings	560' - ½" Circular bars

$\frac{1}{2}$" Circular straight bars 30,000'

2,800'

2,880'

3,360'

560'

39,600'

39,600 x .668 = 26,500#

26,500 @ \$ 3.20/100# = \$ 850.00

$\frac{1}{2}$" Circular bent bars 2520'

6000'

5300'

13,820'

13,820 x .668 = 9250#

9250 @ \$ 3.60/100# = \$ 335.00

3/4" Circular bars 4560'

4560 x 1.502 @ \$ 3.00/100# = \$205.00

7/8" Circular bars 1800'

1800 x 2.04 @ \$ 3.00/100# = \$110.00

1" Circular bars 4800'

1500'

1040'

7340'

7340 x 2.67 = 19,600#

19,600 @ \$ 3.00/100# = \$ 588.00

1" square bars 7000'

7000 x 3.4 @ \$ 3.00/# = \$ 714.00

Total reinforcing steel cost = \$ 2802.00

Filler

100 sq. ft. @ \$ 0.05/sq.ft. = \$ 50.00

Structural steel and rivets

Steel 2756# @ $ 4.32/100# = $ 120.00

Rivets 300 @ $ 5.25/100 = $ 15.75

Total = $ 135.75

Lumber

22,230 ft. B.M. @ $ 100.00/1000 ft. B.M. = $ 2223.00

Roofing

21,120# @ $ 2.26/100# = $480.00

Hardware

9 x $ 10.00/Truss = $ 90.00

Total Estimate = $ 12,043.65

From this estimate it is believed that all the materials required for construction of the building, with the exception of doors, windows, and miscellaneous fixtures, could be supplied for approximately $ 12,000.