A DESIGN OF A REINFORCED CONCRETE BUILDING AND SERVICE UTILITIES FOR A CHERRY FARM

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE
L. E. Sobkowski
1947

A Design of a Reinforced Concrete Building and Service Utilities for a Cherry Farm

A Thesis Submitted to

The Faculty of

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of

AGRICULTURE AND APPLIED SCIENCE

By

L. E. Sobkowski

Candidate for the Degree of

Bachelor of Science

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3/21/47
GIFT

DEDICATION

To my sons, Stevie and Joe

ACKNOWLEDGMENTS

I wish to thank the members of the Civil Engineering Staff for their assistance in this thesis, in particular, Professor C. A. Miller, for the time and effort spent with me to perfect the design both structurally and practicably.

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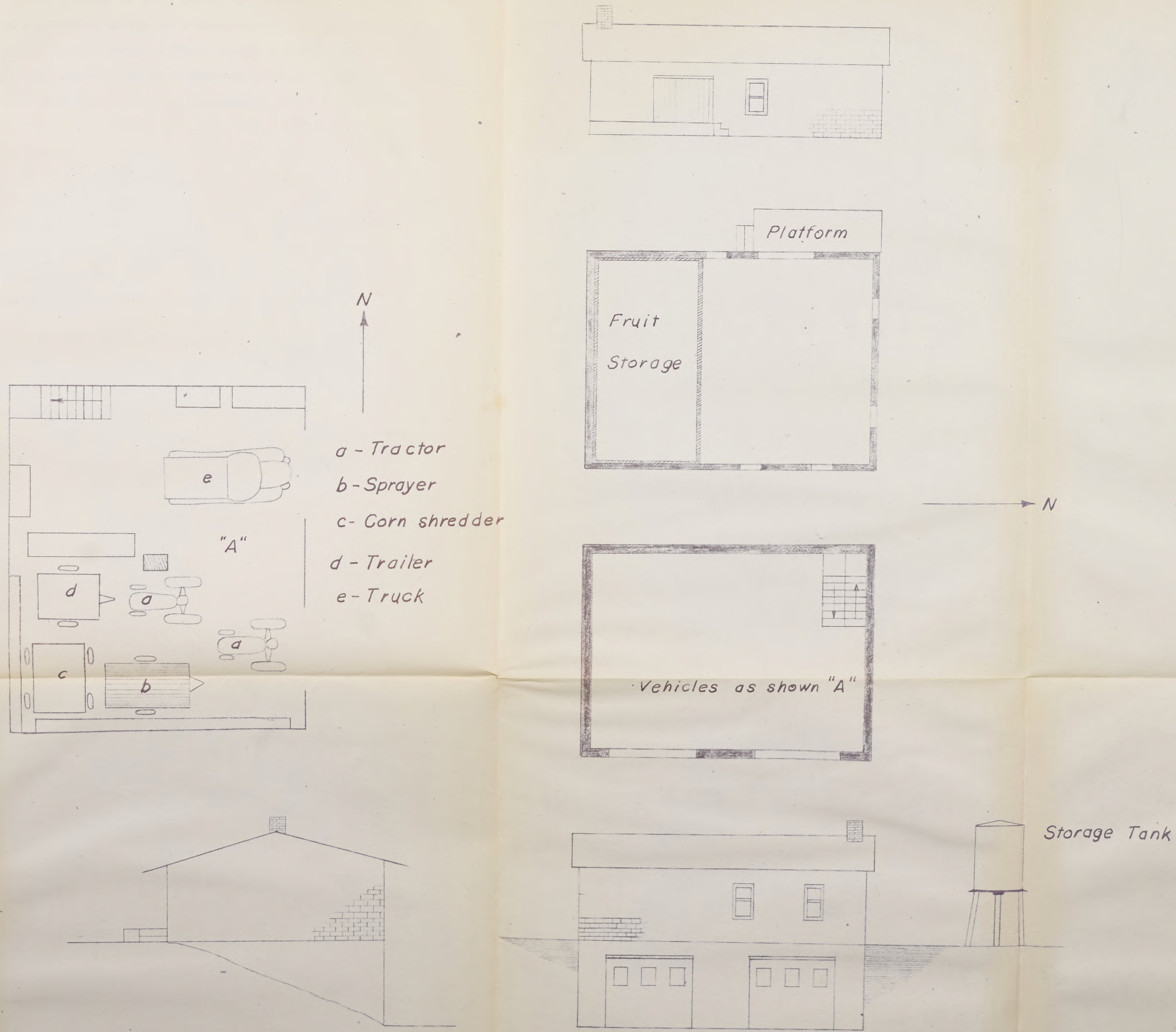


PROPOSED BUILDING

Dwg. No.1

PART I

Purpose and Scope of Thesis.

The purpose of this thesis is to present a practical and economical design of a building and service facilities for use by the owner, a fruit grower in Grand Traverse County. The owner plans an expansion in property and operations and thus I have taken the opportunity to gain some experience in practical design. Design methods used by the author are those set forth in the respective design courses as taught at Michigan State College. Thus it may be stated that much more labor had to be applied to this problem than would be necessary if handled by an experienced design engineer. Despite this fact, it is a worthwhile project in that the application of fundamentals is necessary before acquiring the tricks and short cuts of the trade.

The owner operates about 150 acres of cherry and apple orchards. This holding is to be increased in the near future and the required additions and changes to the present set-up constitute the object of this thesis.

A. To locate and design a fireproof, durable building to perform the following functions:

1. House the following listed tools and machinery:

   2 tractors

   1 pick-up truck

   1 flat trailer

1 corn shredder

1 Orchard sprayer unit

Work benches

Welding set

Woodworking machines

Lubrication facilities

2. Provide means for maintenance and repair of tools and machinery.

3. Stock all necessary spray materials for orchard pest and disease control.

4. Provide an inspection, weighing, and crating center for all seasonal fruits.

5. Provide a separate section of this building for extended storage of fruit. This section will be a part of the main building but of conventional storage design.

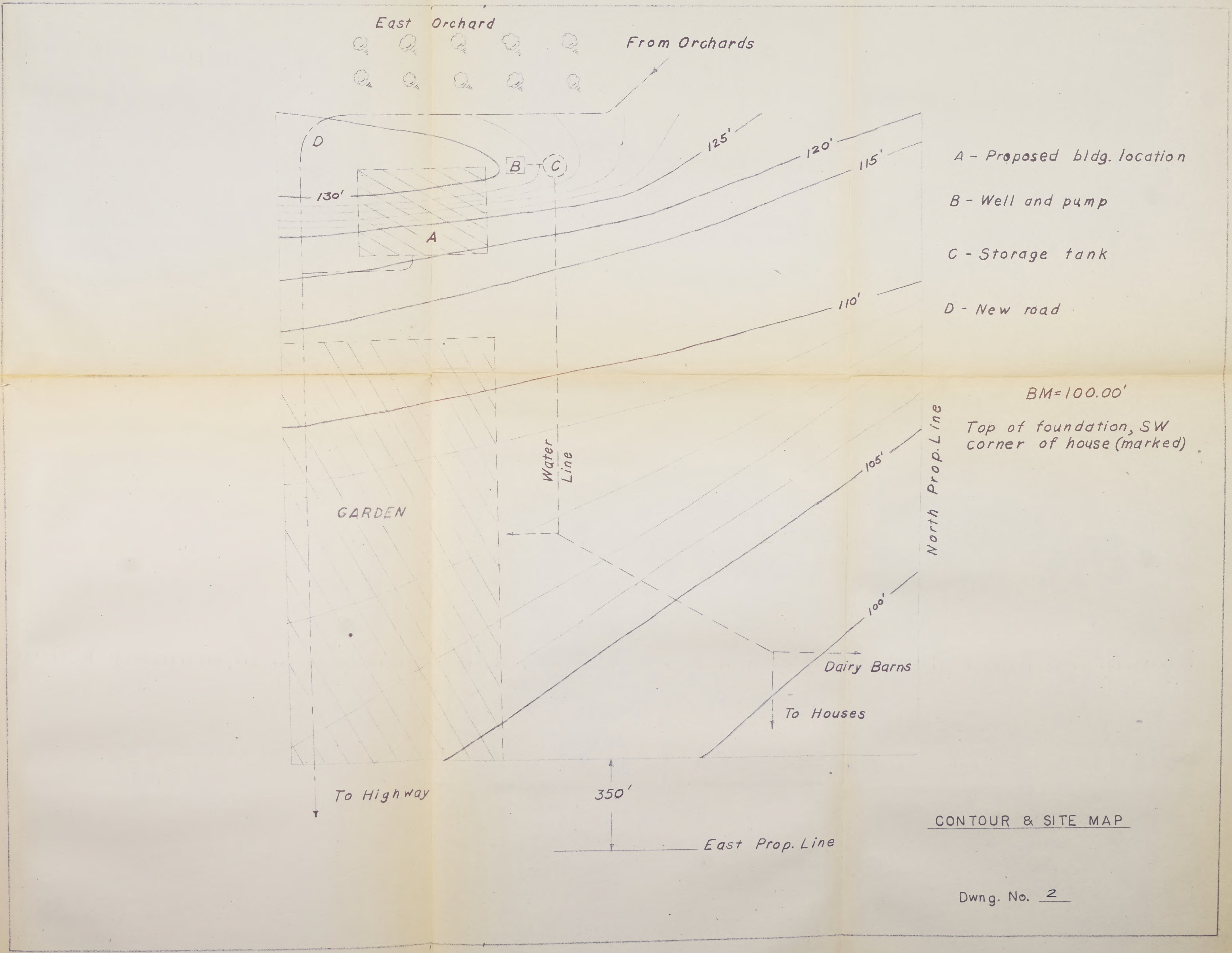
The owner desires this building to be located at the edge of an orchard, at the crest of a slight hill leading to the East boundary of this orchard.

(See Contour Map - Drawing No. 2)

B. To locate and design a well to be driven adjacent to this building.

C. Select the most efficient and economical method of pumping water from this well for the following purposes:

1. Water for orchard spray. Under the present plan of operation, a 400 gallon tank sprayer is being


East Orchard
From Orchards
D
B
C
A
130'
125'
120'
115'
110'
105'
100'
A - Proposed bldg. location
B - Well and pump
C - Storage tank
D - New road
BM=100.00'
Top of foundation, SW corner of house (marked)
North Prop. Line
Water Line
GARDEN
Dairy Barns
To Houses
To Highway
350'
East Prop. Line
CONTOUR & SITE MAP
Dwng. No. 2

used and at the height of the spraying season, water must be supplied at the rate of 400 gallons per hour.

2. Water for dairy stock and other farm animals. Predicted future needs are 100 gallons per day.
3. Fire protection needs. A main of suitable size installed to make available adequate supplies of water at a sufficient head.
4. Garden irrigation. Fresh vegetables comprise an important item of sale in the owner's farm market. In order to assure high quality and early appearance on the market, the owner requires a pipe line installed to the gardens and some means of effective distribution designed.

D. To locate and design:

1. An approach branch and turnaround from the existing orchard road to the upper story of the building.
2. A road from the lower story of the building running adjacent to the farm yard and houses, joining the highway at a point other than the existing drive. This will eliminate all heavy traffic through the farm and near the houses.

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 for the allied trades such as plumbing, heating, or wiring have not been mentioned or treated.

ALLOWABLE UNIT STRESSES USED

From ACI Building Code 1946

Using a concrete with f'_c = 3000 p.s.i.

n = 10

Steel - Billet, Hard Grade

f_s = 20,000 p.s.i.

COMPRESSION f_c 1350 p.s.i.

SHEAR v

(Beams)

W/o Web Reinf. - W/o S.A.	-	60 p.s.i.
W/o Web Reinf. - With S.A.	-	90 p.s.i.
With Web Reinf. - W/o S.A.	-	180 p.s.i.
With Web Reinf. - With S.A.	-	360 p.s.i.
One way footings	-	75 p.s.i.

BOND u

(2-way footings beams)

Plain Bars	-	120 p.s.i.
Deformed Bars	-	150 p.s.i.
Plain Bars (Hooked)	-	135 p.s.i.
Deformed Bars	-	168 p.s.i.

BEARING f_c

On full area	-	750 p.s.i.
1/3 Area or Less	-	1125 p.s.i.

O'ROURKE

SOIL PRESSURE

3 tons per sq. ft.

A.S.T.M. C 90-39

Bearing on masonry block - 1000 p.s.i.

PART II

Loading Platform Design -

Sketch #1

Class E concrete

$1:6\frac{1}{2}$ Vol. mix

f'_c = 3000 p.s.i.

f_s = 20,000 p.s.i.

f_c = 1350 p.s.i.

n = 10

Computing Min. Live Load

Apples to occupy a space 6' x 16' x 8' = 768 cu. ft.

w/cu. ft. apples = 35# (Bldg. Code Com. of the National Bureau of Standards November 1924)

768 cu. ft. x 35#/cu. ft. = 26,880# total live load.

Total Area = 18' x 6' = 108 p.s.f.

Unit L.L. = $\frac{26,880}{108}$ = 250 p.s.f.

Floor Load Recommended - 300 p.s.f.

Assume D.L. = 50 p.s.f.

Total Load = 350 p.s.f.

For a 1' section:

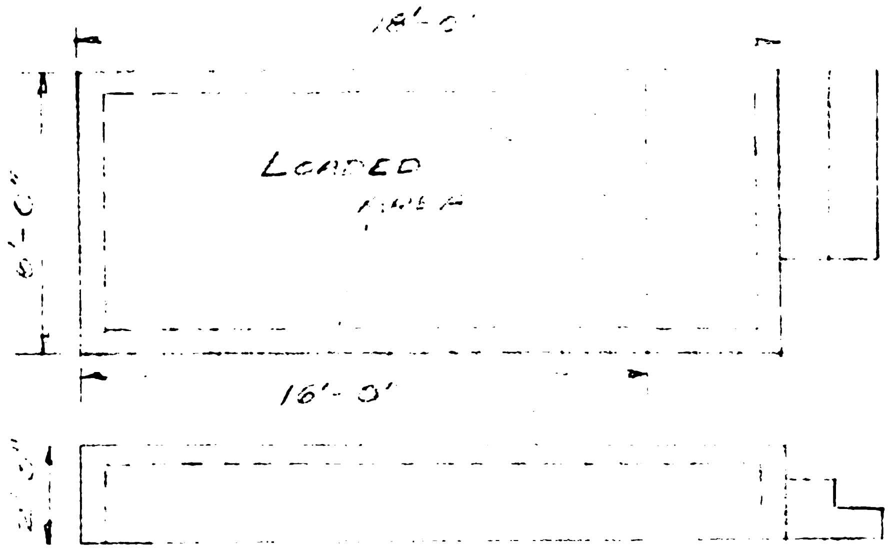
$V = \frac{wl}{2} = \frac{350 \times 6}{2}$ = 1050#/Lin. ft.

$BM = \frac{wl^2}{8} = \frac{350 \times 6^2}{8}$ = 1575'#

From table 2, RCDH, d = 3" RM = 2.12' K

$A_s = \frac{M}{ad}$ a = 1.44

$A_s = \frac{1.575}{1.44 \times 3}$ = .364 sq. in./ft.



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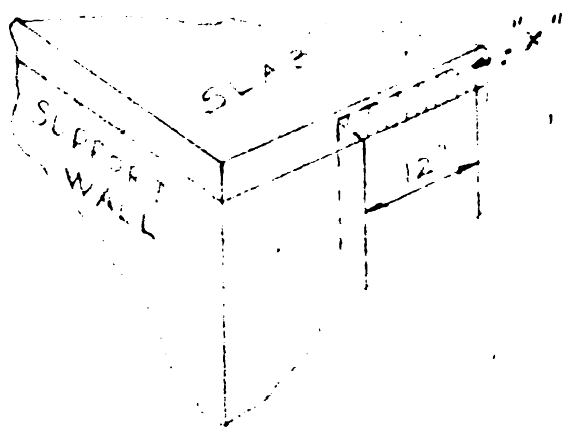

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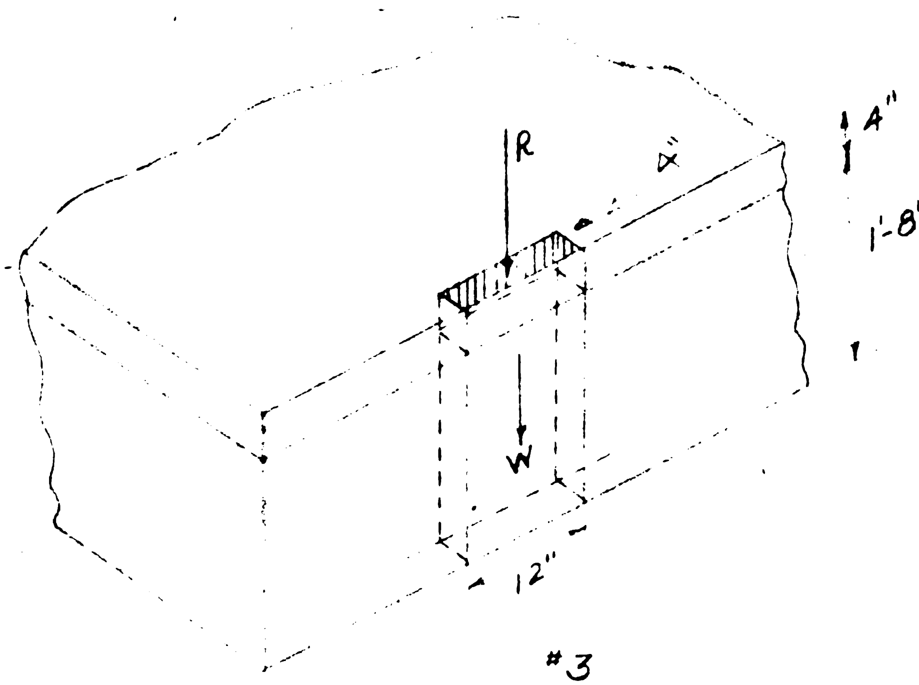


#3

SKETCHES #1-3

$$\Sigma o = \frac{V}{jdu} = \frac{1050}{7/8 \times 3 \times 150} = 2.67"$$

Table 3 RCDH

Use 3/8" Ø $3\frac{1}{2}$" c.c. spacing. o = 4.0

Use 1" cover, depth = d + c = 3" + 1" = 4"

Check weight - $\frac{4"}{12"}$ x 1' x 1' x 150' = 50 p.s.f. OK

SHEAR

$$v = \frac{V}{bjd} = \frac{1050}{12 \times 7/8 \times 3} = 33.4 \text{ p.s.i.} \quad \text{OK}$$

BOND

$$u = \frac{V}{\Sigma ojd} = \frac{1050}{4 \times 7/8 \times 3} = 100.2 \text{ p.s.i.} \quad \text{OK}$$

Check Bearing on Support - Sketch 2

R slab = 1050# Allowable Bearing Stress = 750 p.s.i.

ACI T 305

$$A = \frac{1050\#}{750 \text{ p.s.i.}} = 1.4 \text{ sq. in required}$$

A = 12 x , $x = \frac{1.4}{12}$ - .1165" thickness required

Use 4" thick support wall.

Check Earth Bearing at Base of Support Wall - Sketch 3

Allowable Bearing Stress - 8000 p.s.f.

(Ref. - O'Rourke Hdbk. - Compact, coarse sand or

stiff gravel)

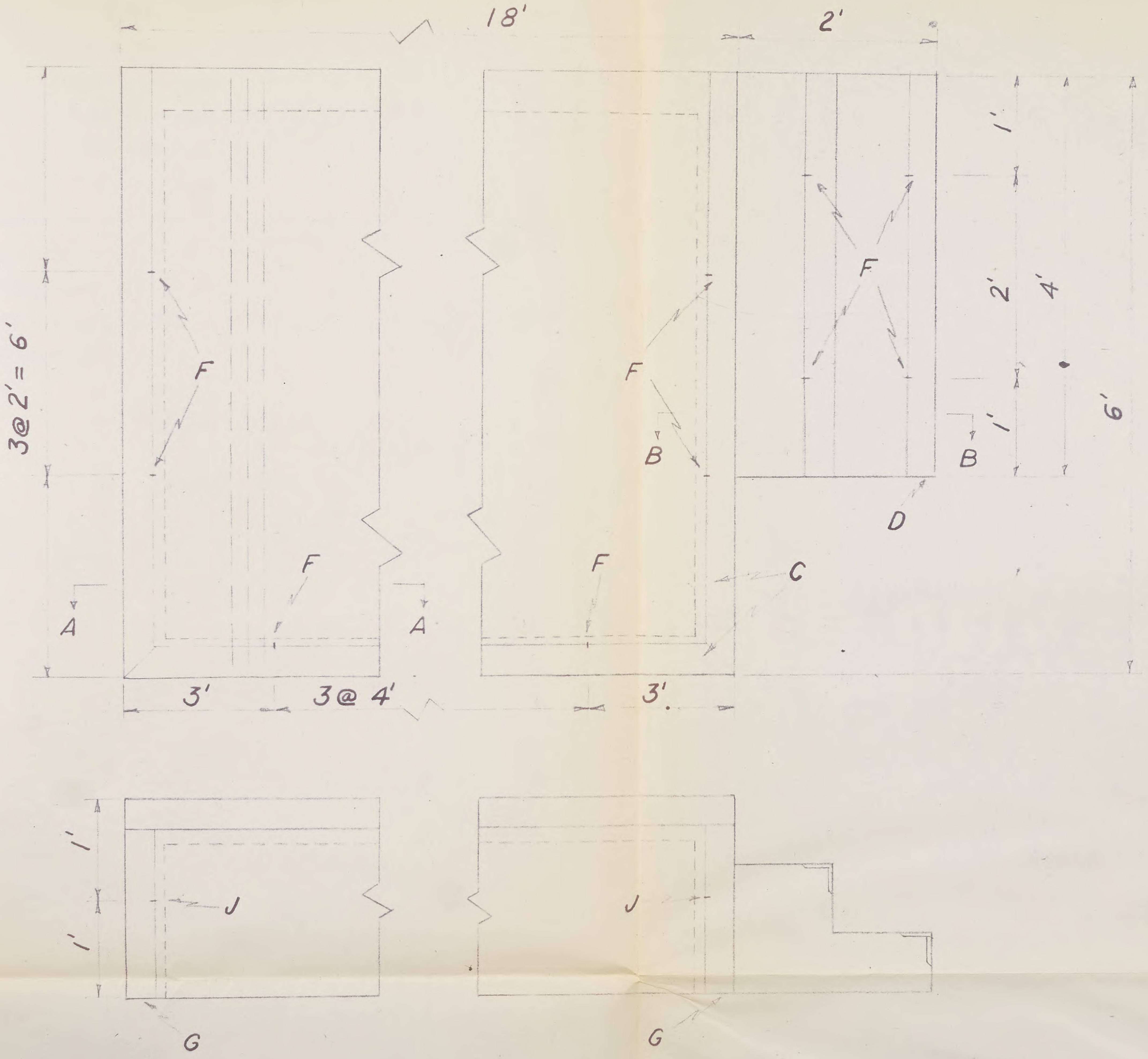
R = 1050#

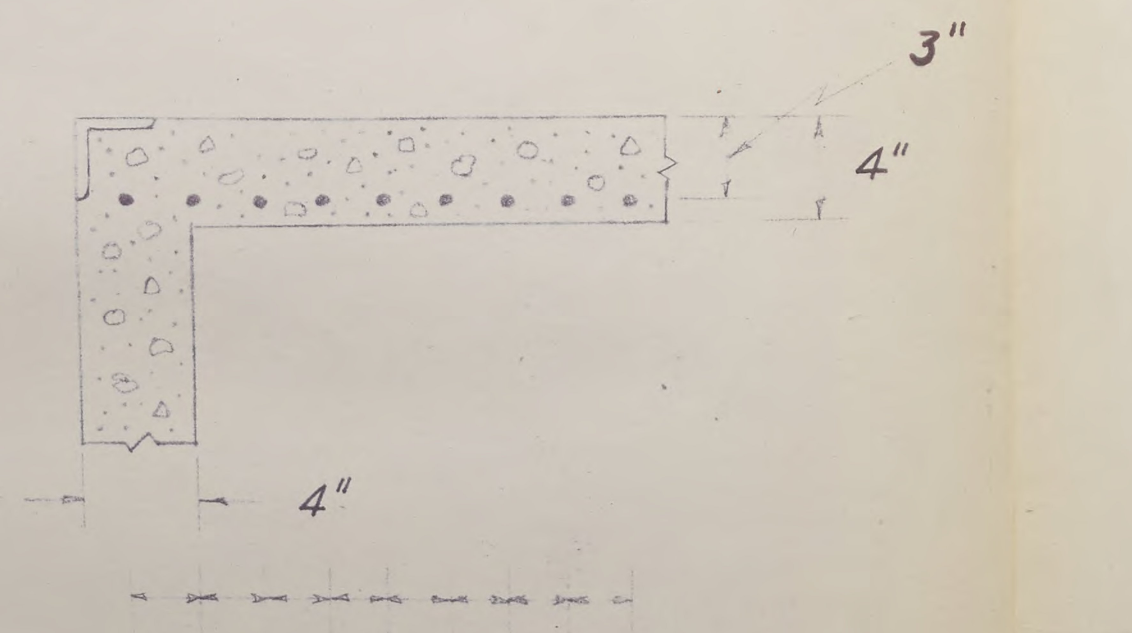
W_2 = 1 2/3 x 1/3 x 150 = 125# (Support)

Total Vert. Ld. = 1175#

Area in Bearing = $\frac{12" \times 4"}{144}$ = .33 sq. ft.

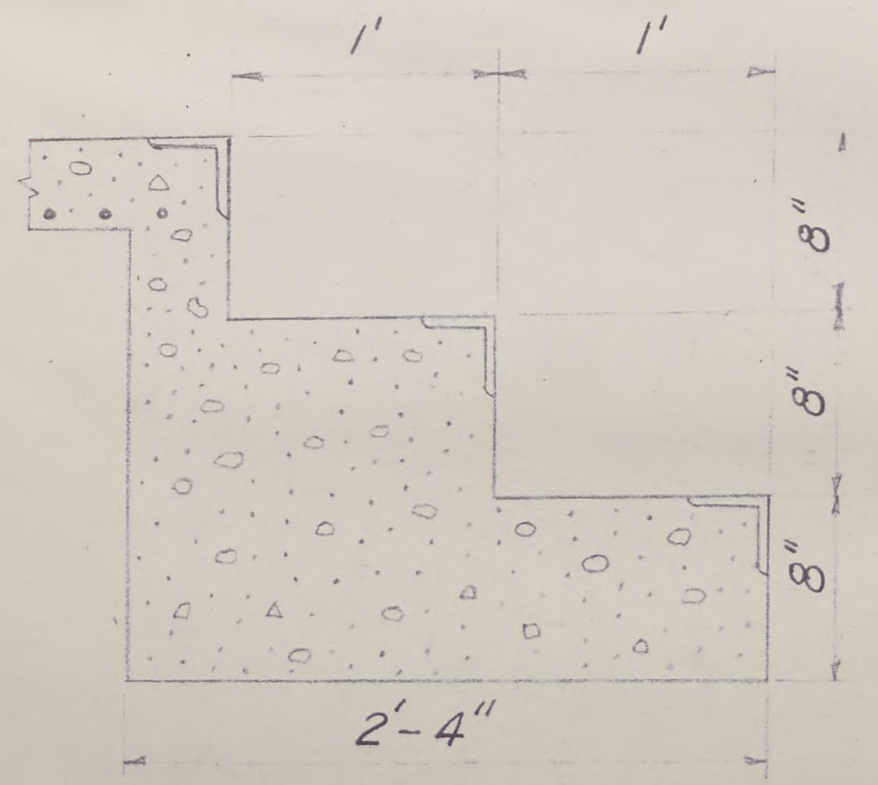
$\frac{1175\#}{.333}$ = 3530 p.s.f. OK



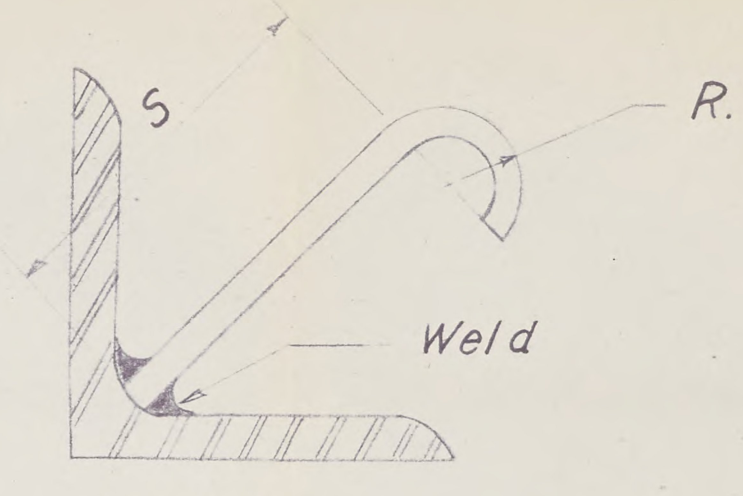


$\frac{3}{8}$" ϕ Bars $3\frac{1}{2}$" c.c.

Sect. A-A



Sect. B-B



Items E, H

Steel: Struc. Grade Bars, Plain

$\frac{3}{8}$" ϕ 78 5'-10"

Corner Plates

C - L 3×3×$\frac{1}{4}$ 1@18'-0" Reqd.
2 @ 6'-0" "

D - L 3×2×$\frac{1}{4}$ 2 @ 4'-0" "

G - L 3×3×$\frac{1}{4}$ 2 @ 1'-9" "

Hooks

E - Located as marked F
$\frac{1}{2}$" ϕ 12 @ $5\frac{1}{2}$" Reqd., S=$2\frac{1}{2}$" R=1"

H - Located as marked J
$\frac{1}{4}$" ϕ 2 @ $4\frac{3}{4}$" Reqd., S=2" R=$\frac{3}{4}$"

LOADING PLATFORM DETAILS

Dwng. No. 3

ROOF DESIGN

Sketch 4

Fink Truss

40' - 0" Span

8' - 0" Rise

16' - 0" C.C.

1' - 0 Eaves

This truss was selected from Timber Engineering Co. Truss Manual.

Total load = 40 p.s.f.

Loading will be checked to see that this is not exceeded.

$\frac{1}{2}$ Roof Area = (21.5' ≠ 1.0') x (48' ≠ 2) = 1125 s.f.

$\theta = 21^\circ 50'$

$\text{Sin } \theta = \frac{8}{21.5} = .3721$

$\text{Cos } \theta = \frac{20}{21.5} = .9302$

<u>LIVE LOADS</u>

Wind Load, $P_1 = 30$ p.s.f. (Horiz.)

$P = P_1 \frac{2 \text{ Sin. } \theta}{1 \neq \text{Sin.}^2 \theta}$

$= 30 \times \frac{2 \times .3721}{1 \neq .3721}$

P = 19.6 p.s.f. Say 20 p.s.f.

$P'' = P \cos \theta$

$= 20 \times .9302$

= 18.6 p.s.f.

Snow Load - 25 p.s.f. (Vertical)

It is unreasonable to expect a roof to be subjected to max. snow load and max. wind load simultaneously.

Therefore, the following load combination is judged adequate:

Dead Load + $\frac{1}{2}$ wind load on one side + full snow load on both sides

$$T.L. = D.L. + \frac{18.6}{2} + 25$$

$$= D.L. + 34.3 \text{ p.s.f.}$$

<u>DEAD LOADS</u>

From Truss Design:

501 FBM @ 2580 #/1000 FBM = 1290 # Truss Wt.

Roofing - Asbestos Shingles 200#/100 sq. ft.

200 x 11.25 = <u>2250#</u>

Felt 25#/100 sq. ft.

25 x 11.25 = 280#

Total Dead Load = $\frac{1290}{2}$ + 2250 + 280 = 3175# (For $\frac{1}{2}$ Roof)

$\frac{1}{2}$ Horizontal Area = 21 x 50 = 1050 sq. ft.

Unit Dead Load = $\frac{3175\#}{1050 \text{ sq. ft.}}$ = 3.02 p.s.f.

Total Load = 34.3 + 3.02 = 37.32 p.s.f.

Use <u>40 p.s.f.</u>

Reaction = T.L. x $\frac{\text{Area}}{2}$ = 40 x 1050 = 42,000#

Use 4 trusses 16' - 0" c. to c.

$\frac{42,000}{4}$ = 10,500# Each wall truss reaction

Length = 48'

$\frac{42,000}{48}$ = 875#/Lin. ft. of wall - Unit Live Load

Gross Area of 8 x 8 x 16 Concrete Block

15 3/4" x 7 3/4" = 122 sq. in.

1000 p.s.i. Allowable Bearing

ASTM C90-39

$122 \text{ sq. in} \times \frac{12}{16} = 91.5$ sq. in. per lin. ft.

$\frac{875}{91.5} = 9.6$ p.s.i. Bearing on Top Course OK

UPPER WALL DEAD LOAD

8 x 8 x 16 Concrete Block

Heavy load bearing

$\frac{1}{4}$" joint - 50#

10' wall Height = 120"

$\frac{120}{8} = 15$ courses

40' wall length = 480"

$\frac{480}{16} = 30$ blocks

48' wall length = 576"

$\frac{576}{16} = 36$ blocks

450 block per 400 sq. ft. of wall area

$\frac{450}{400} = 1.125$ block per sq. ft. of wall

North Wall

40' Long — 2 - 3 x 5 windows

South Wall

40' Long — No windows

East Wall

48' Long — 2 - 3 x 5 windows

West Wall

48' Long — 1 - 3 x 5 window

1 - 7 x 8 door

	N.W.	S.W.	E.W.	W.W.
Area Sq. ft.	470	500	450	409
No. Block	530	563	506	460
Tot. Wt. #	26,500	28,150	25,300	23,000
Unit Wt.#/Lin.'	663	704	527	480

Check East Wall for Bearing on Bottom Course

D.L. - 527 #/Lin. Ft. 91.5 sq. in. per lin. ft.

L.L. - 875 Area of Block

T.L. -1402 #/Lin. Ft.

$\frac{1402 \text{ \#/ft.}}{91.5 \text{ sq. in./ft.}} = 15.3$ p.s.i. OK

Check South Wall

D.L. - 704 #/Lin. Ft.

L.L. - 0

T.L. - 704 #/Lin. Ft.

$\frac{704\text{\#/ft.}}{91.5 \text{ sq. in./ft.}} = 7.7$ p.s.i. OK

WINDOW LINTEL DESIGN

East Wall Windows Sketch 5

3 courses at top

3 x 50 = 150#

$150 \times \frac{12}{16} = 113$#/Lin. Ft.

8" = .67' width

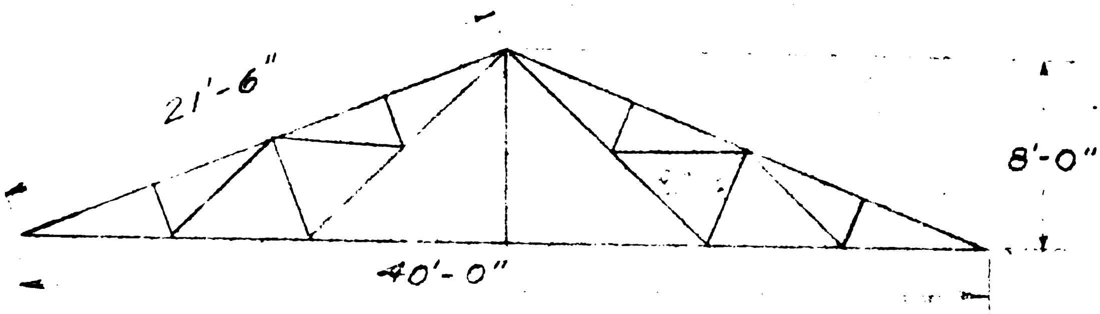
1' x .67' = .67 sq. ft.

L.L. - 875#/Lin. Ft.

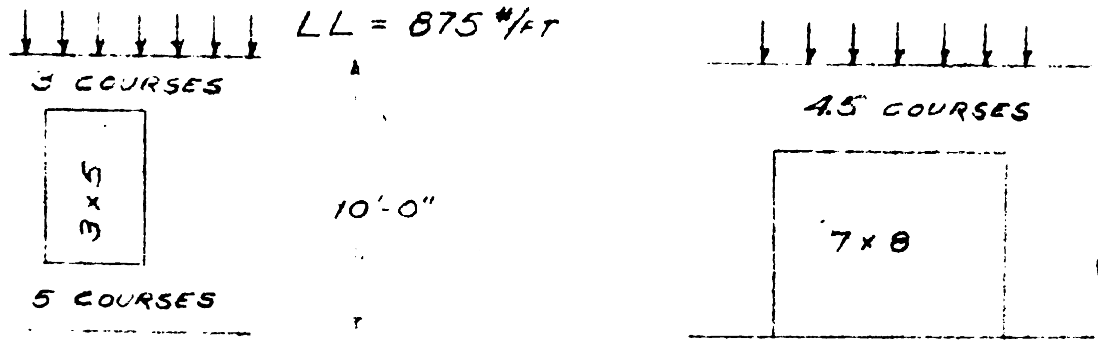
D.L. - 113'

T.L. - 988#/Lin. Ft.

$w = \frac{988\text{\#/Lin. Ft.}}{.67 \text{ sq. ft.}} = 1480$ p.s.f.

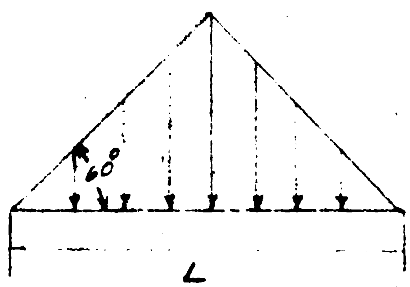


#4 ROOF TRUSS

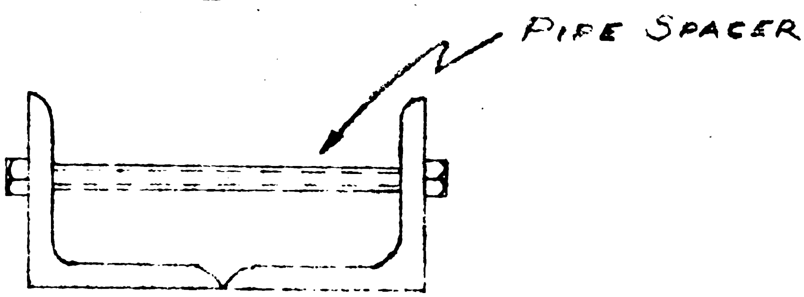


#5 WINDOW

#6 DOOR



#7 TRIANGULAR LOADING



#8 LINTEL DESIGN

SKETCHES #4-8

Sketch 7

$$\text{Max. B.M.} = \frac{wL^3}{24} = \frac{1480 \times (3.25)^3}{24} = 2120'\#$$

$$S = \frac{M}{f} = \frac{2120 \times 12}{20{,}000} = 1.275 \text{ in.}^3$$

Use 2 - 4 x 4 x $\frac{1}{4}$ L's $S = 1.1 \text{ in.}^3$ $2S = 2.2 \text{ in.}^3$

Sketch 8

This design will suffice for all windows in the upper story.

DOOR LINTEL DESIGN

West Wall Upper Story -

Sketch 6

$4.5 \times 50 = 225\#$

$225 \times \frac{12}{16} = 169\#/\text{Lin. Ft. D.L.}$

D.L. = 169#/Lin. Ft.

L.L. = 875

T.L. = 1044#/Lin. Ft.

$$w = \frac{1044}{.67} = 1560 \text{ p.s.f.}$$

$$\text{Max. B.M.} = \frac{wL^3}{24} = \frac{1560 \times (8.33)^3}{24} = 39{,}200'\#$$

$$S = \frac{39{,}200 \times 12}{20{,}000} = 23.5 \text{ in.}^3$$

Use 2 - 8 x 4 x 7/8 L's $S = 12.5$ $2S = 25 \text{ in.}^3$

Sketch 8

UPPER FLOOR SLAB -

Sketches 9 & 10

Live Load on Storage Section

Area 16' x 40' = 640 sq. ft.

Vol. of storage, considering apples in bulk 8 ft. high.

15' x 38' x 8' = 4550 cu. ft.

Weight - Vol. x unit Wt. of apples

W = 4550 x 35 = 160,000#

$$\frac{160{,}000}{640} = 250 \text{ p.s.f.}$$

Floor Load Recommended - 300 p.s.f.

Assume a 6" thickness

Wt. = 1' x .5' x 1' x 150 = 75 p.s.f.

L.L. = 300 p.s.f.

D.L. = 75

T.L. = 375 p.s.f. or for 1' ft. section 375#/Lin. Ft.

Considering this a simply supported, uniformly distributed loaded beam:

$$\text{Max. B.M.} = \frac{wL^2}{8} = \frac{375 \times 64}{8} = 3000'\#$$

$$V = \frac{wL}{2} = \frac{375 \times 8}{2} = 1500\#$$

From Table 2 R.C.D.H.

$d = 3\frac{3}{4}$"

Changing thickness of slab to 5"

B.M. = 2900'#   d = $3\frac{1}{2}$"   V = 1450

$$A_s = \frac{M}{a\,d} \qquad a = 1.44$$

$$= \frac{2.9}{1.44 \times 3.5} = .575 \text{ sq. in./ft. Reqd.}$$

Table 3 R.C.D.H.

Use $\frac{1}{2}$" ø 4" spacing $\Sigma o = 4.7$"

SHEAR

$$v = \frac{V}{bjd} = \frac{1450}{12 \times 7/8 \times 3\frac{1}{2}} = 40 \text{ p.s.i. OK}$$

BOND

$$u = \frac{V}{\Sigma o j d} = \frac{1450}{4.7 \times 7/8 \times 3\frac{1}{2}} = 101 \text{ p.s.i.} \quad \underline{OK}$$

T - BEAM DESIGN

Sketch 11

Span 20' C. to C.

Assume 19' Clear Span

L.L. = 375 p.s.f. x 8 ft. = 3000#/ft.

D.L. = Assume Stem Wt. = 200#/ft.

T.L. = 3200#/ft.

$$V = \frac{wl}{2} = \frac{3200 \times 19}{2} = 30{,}400\#$$

Allowable v = .06 f'_c = 180 p.s.i. with web reinforcement

$$v = \frac{V}{b'dj} \qquad b'd = \frac{30{,}400}{180 \times 7/8} = 193 \text{ sq. in.}$$

Let b' = 10" d + 3 = 25"

d = 22" d - t = 22 - 5 = 17"

Checking for weight of stem

$$W = \frac{10 \times 18}{144} \times 150 = \underline{188\#/ft.} \quad \underline{OK}$$

Assume j = .92

$$BM = \frac{wl^2}{10} = \frac{3200 \times 19 \times 19 \times 12}{10} = 1{,}390{,}000"\#$$

BM = Tjd

$$T = \frac{1{,}390{,}000}{.92 \times 22} = 68{,}700\#$$

$$A_s = \frac{T}{f_s} = 3.43 \text{ sq. in. per ft. Reqd.}$$

Use 4 - 1" ф Bars, 2 rows Area = 4.00 sq.in.

BOND

$$u = \frac{V}{\Sigma o j d} = \frac{30{,}400}{4 \times 4 \times .92 \times 22} = \underline{94 \text{ p.s.i.}} \quad \underline{OK}$$

120 p.s.i. All. Art. 306 ACI

REVIEW OF T-BEAM DESIGN

Sketch 12

To select flange width "b", the <u>least</u> of these three conditions is chosen:

1. $\frac{\text{Span}}{4} = \frac{20 \times 12}{4} = \underline{60"}$ *LEAST
2. 8 x Thickness x 2 + 10" = <u>74"</u>
3. (8'-0") - (0'-10") = 7'-2" clear span

   2(3'-7") + 10 = <u>96"</u>

$\underline{\Sigma M_{na} = 0}$

$(50 \times 4)(x - 3) = 40(22 - x)$

$x = 6.17"$

$\frac{z}{2.17} = \frac{f_c}{6.17}$

$z = .352 f_c$

$f_c - z = .648$

<u>Comp.</u>	<u>C</u>	<u>Arm</u>		<u>Mom</u>
$C_1 = .352 f_c \times 4 \times 60 =$	$84.5 f_c$	x 20	=	$1690 f_c$
$C_2 = \frac{1}{2} \times .648 f_c \times 4 \times 60 =$	$\underline{77.8 f_c}$	x 20.67	=	$\underline{1610 f_c}$
	$C_1 + C_2 = C = 162.3 f_c$		=	$3300 f_c$

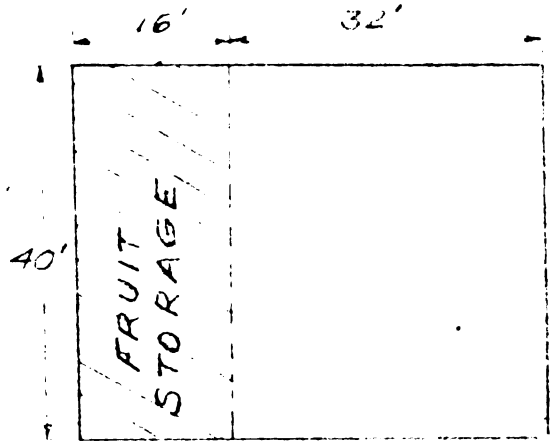
$3300 f_c = 1,390,000"\#$

$f_c = \underline{420 \text{ p.s.i.}}$ <u>OK</u>

$C = 162.3 \times 420 = 68,000 \#$

$C = T = A_s f_s$

$f_s = \frac{68,000}{4} = \underline{17,000}$ p.s.i. <u>OK</u>



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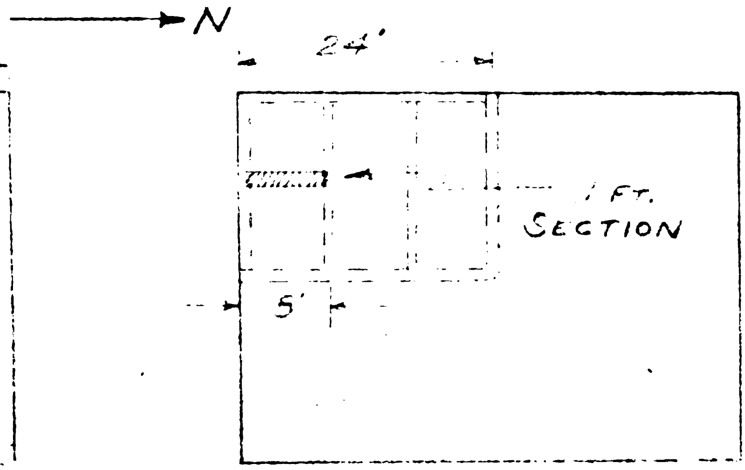

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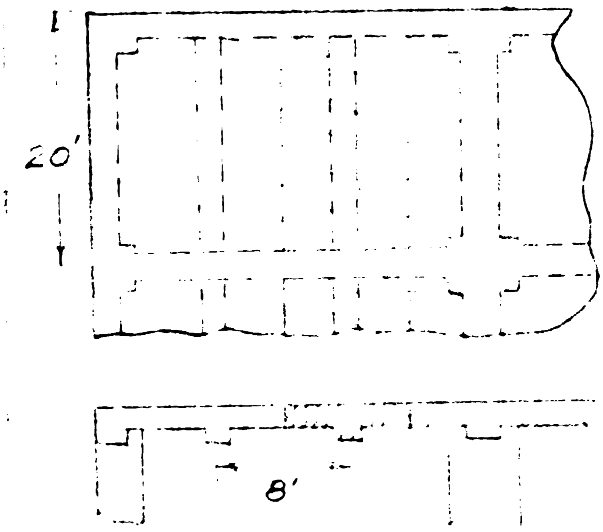

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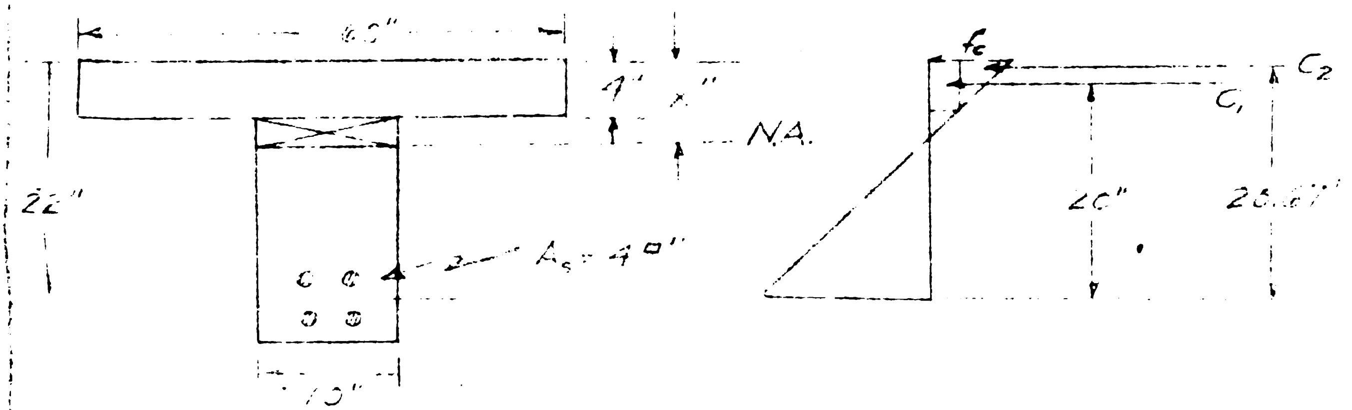


#12

SKETCHES #9-12

CHECK OF T-BEAM AT SUPPORTS

To locate neutral axis: Sketch 13

$\Sigma M_x = 0$

$A_1 (x - 2) + A_2 (x/2) = A_3 (22 - x)$

$x = 8.2"$

$z = f_c \frac{3}{8.3}$

$= .476 f_c$

$f_c - z = .524$

$C_s = 36 \times .476 f_c = 17.1 f_c$

$C_c = \frac{1}{2} \times .524 \ f_c \times 10 \times 8.3 = 21.8 f_c$

$T = C_s + C_c = 38.9 f_c$

$C_s \times 20$	$= 17.1 f_c \times 20$	$= 342 f_c$
$C_c \times 20.67$	$= 21.8 f_c \times 20.67$	$= 552 f_c$
	$38.9 f_c$	$894 f_c$

a = 23" Moment Arm of Resultant C

$T = C = \frac{BM}{a} = \frac{1,390,000}{23} = 60,400\#$

$f_c = \frac{60,400}{38.9} = \underline{1330}$ p.s.i. <u>OK</u>

$f_s = \frac{60,400}{4} = \underline{15,100}$ p.s.i. <u>OK</u>

T - BEAM WEB REINFORCEMENT

Sketch 14

The maximum shear, 30,400# occurs at the ends when the entire span is loaded with the uniformly distributed load - 3000 #/ft.

$$v_m = \frac{30,400}{10 \times .92 \times 22} = 150 \text{ p.s.i.}$$

The center shear will not be computed but will be calculated as 25% of maximum shear stress.

$$v_c = 150 \times .25 = 37.5 \text{ p.s.i.}$$

$$x = 114" \times \frac{90}{112.5} = 91"$$

V taken by Stirrups $\frac{90 \times 91}{2} \times 10 - 41,000\#$

Use $\frac{1}{2}$" ø Stirrups A = .1963 sq. in. x 2 = .3926 sq."

16,000 p.s.i. for web reinf. (J.C. Spec.)

.3926 x 16,000 = 6280# taken by each Stirrup

$\frac{41,000}{6280} = 6.5$ Use 7

$\frac{91"}{7} = 13"$ spacing

but $\frac{d}{2} = 11"$, therefore use 9 Stirrups

Spacing:

From end to center

1 @ 5"

8 @ 10"

PLACEMENT OF NEGATIVE MOMENT STEEL

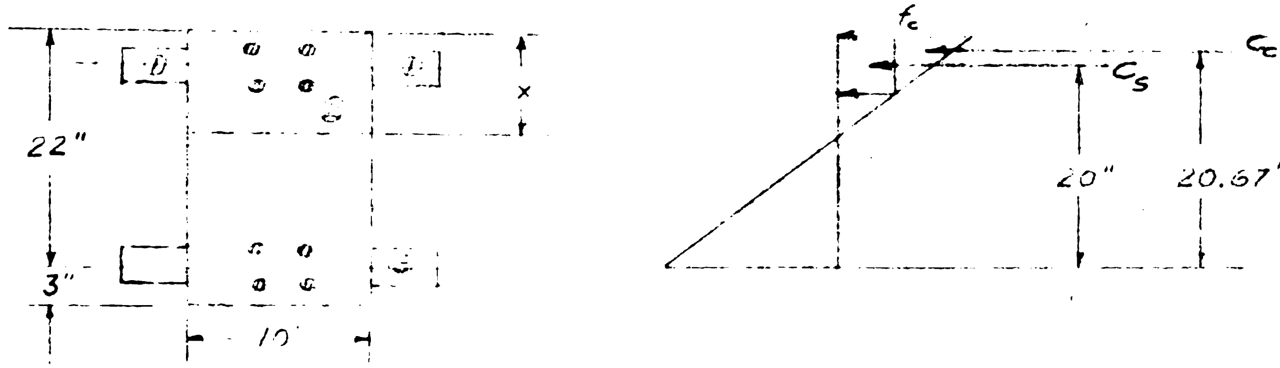
Sketch 15

Reinforcing steel, same size and spacing as that used for tension, will be placed over supports.

Length to be governed by general rule:

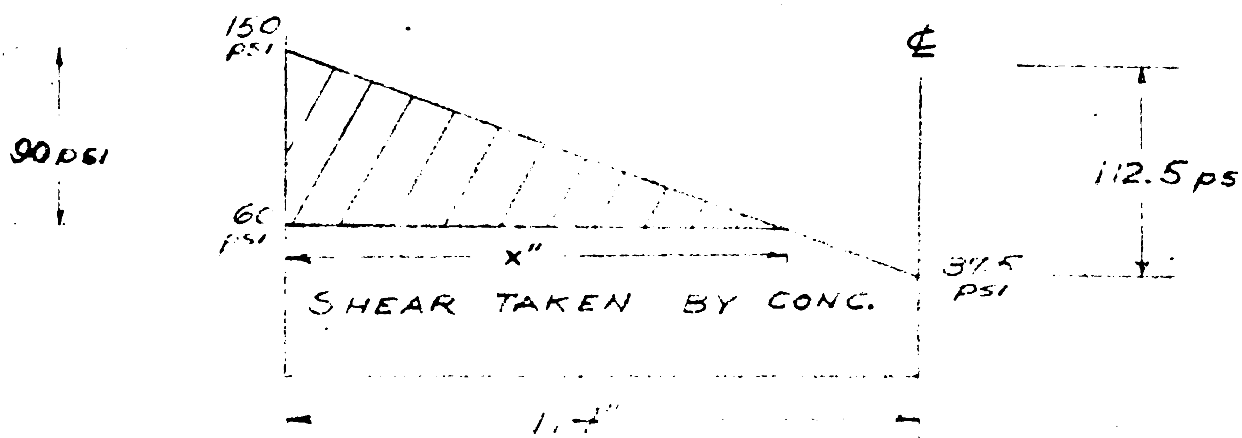
$\frac{1}{4}$ clear span on each side of support.

b = 10"



① $(n-1)A_s = 36$ □"
② 10x
③ $nA_s = 40$ □"

#13



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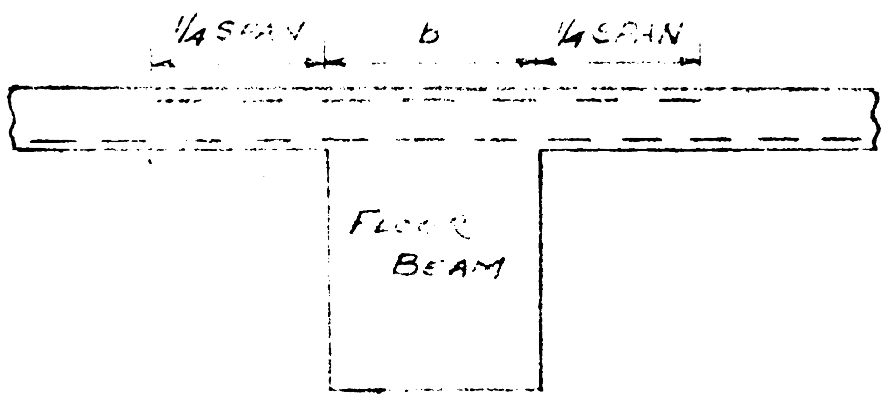


#15

SKETCHES #13-15

Supports - 8'-0" c.c.

Clear Span - (8'-0")-(0'-10")

- 7'-2"

$\frac{7'-2"}{4}$ = <u>22" Length</u>

GIRDER DESIGN

Sketch 16

Considering this beam simply supported

BM at center = 60,800# x 8' = 486,400'# = $1/8\ wl^2$

but BM = $1/10\ wl^2$

$\therefore$ Actual BM at center = 486,400 x $\frac{8}{10}$ x 12 =

4,670,000"#

Weight of Stem, assumed - 400 #/ft.

BM = $1/10\ wl^2$ = 1/10 x 400 x 21 x 21 = 17,640#'

= 211,680#"

Total BM @ center line = 211,600 + 4,670,000

= 4,881,680 #"

V = 60,800 + $\frac{wl}{2}$ = 65,000#

b'd = $\frac{V}{180 \times j}$ = $\frac{65,000}{180 \times 7/8}$ = 413 sq. in.

Use b' = 14"   d + 3 = 33"

d = 30"   D - 4 = 29"

Checking weight of stem

$\frac{29 \times 14}{144}$ x 150 = 420#/' <u>OK</u>

Assume j = .92

BM = Tjd

$T = \frac{4,881,680}{.92 \times 30} = 177,000\#$

$A_s = \frac{T}{f_s} = \frac{177,000}{20,000} = 8.85$ sq. in.

Try $1\frac{1}{4}"$ ø A = 1.56 sq. in.

$\frac{8.85}{1.56} = 5 +$ Bars

Use 6 - $1\frac{1}{4}"$ ø 2 rows $3\frac{1}{2}"$ spacing

A = 9.36 sq. in. $\Sigma 0 = 6 \times 5 = 30"$

BOND

$u = \frac{V}{\Sigma ojd} = \frac{65000}{30 \times .92 \times 30} = 78.5$ p.s.i. OK

REVIEW OF T-BEAM DESIGN - GIRDER

Sketch 17

To select flange width "b"

1. $\frac{Span}{4} = \frac{24 \times 12}{4} = 72"$ * Least

2. 8 x thickness x 2 + 14 = 78"

3. Clear span = 21' x 12 = 252"

To find neutral axis:

$\Sigma M_x = 0$

$(72 \times 4)(x - 2) = (93.60) \times (30 - x)$

$x = 8.9"$

$C_1 = (72 \times 4)\ .55\ f_c = 158.5\ f_c \times 28" = 4430\ f_c$

$C_2 = \frac{1}{2} \times .45\ f_c \times 72 \times 4 = 64.75\ f_c \times 28.67" = 1855\ f_c$

$C_1 + C_2 = C = 223.25\ f_c$ $6285\ f_c$

$R_m = 4,881,680$ "#

$f_c = \frac{4,881,680}{6285} = 777$ p.s.i. OK

$T = C = 223.25 \times 777 = 173{,}500$ #

$f_s = \frac{T}{A_s} = \frac{173{,}500}{9.6} = \underline{18{,}050}$ p.s.i. OK

CHECK OF GIRDER OVER SUPPORTS

Sketch 18

To locate neutral axis:

$$\underline{\Sigma M_x = 0}$$

$$\left(\frac{x}{2}\right)(14\,x) + (84.24)(x - 2) = (93.6)\ (30 - x)$$

$$x = 11.5''$$

$$RM = C_1\ d_1 + C_2\ d_2$$

$$= (.65\ f_c \times 72 \times 4)\ 28 + (\tfrac{1}{2} \times .35\ f_c \times 72 \times 4)\ 28.67$$

$$= 5240\ f_c + 1450\ f_c = 6690\ f_c$$

$f_c = \frac{4{,}881{,}680}{6690} = 730$ p.s.i. OK

$$C = C_1 + C_2 = 187\ f_c + 50.4\ f_c = 237.4\ f_c$$

$C = 237.4 \times 730 = 173{,}500\# = T$

$f_s = \frac{T}{A_s} = \frac{173{,}500}{9.36} = \underline{18{,}500\text{ p.s.i.}}$ OK

WEB REINFORCEMENT

Sketch 19

$v_m = \frac{V}{bjd} = \frac{65{,}000}{14 \times .92 \times 30} = 168.5$ p.s.i. max. at ends.

$v_c = v_m \times .25 = 42.1$ Say 42 p.s.i. at center.

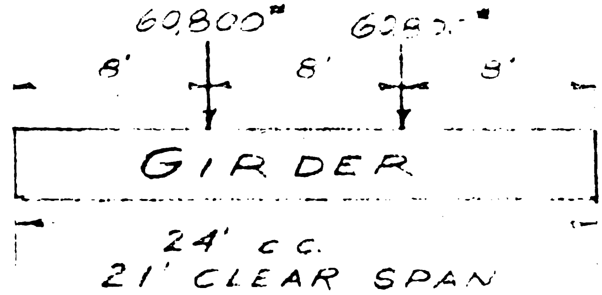
Shear taken by Stirrups $= \frac{108 \times 108.5 \times 14}{2} = 82{,}000\#$

3/4" ø Bars A = .44 sq. in. x 2 = .88 sq. in.

16,000 p.s.i. for web reinforcement.

16,000 x .88 = 14,100# each Stirrup

$\frac{82{,}000}{14{,}100} = 5 +$ Use 6



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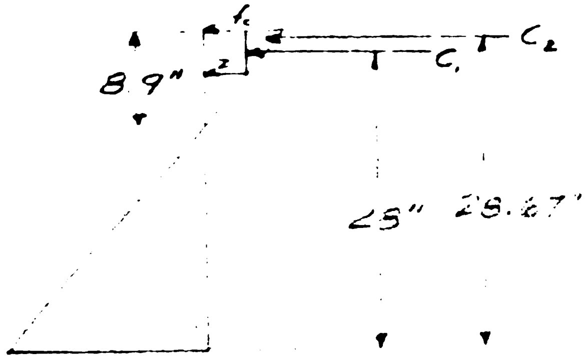

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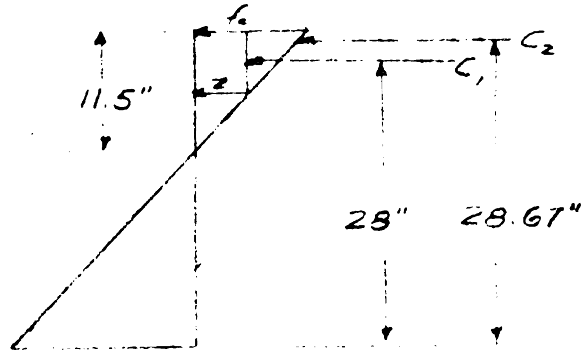

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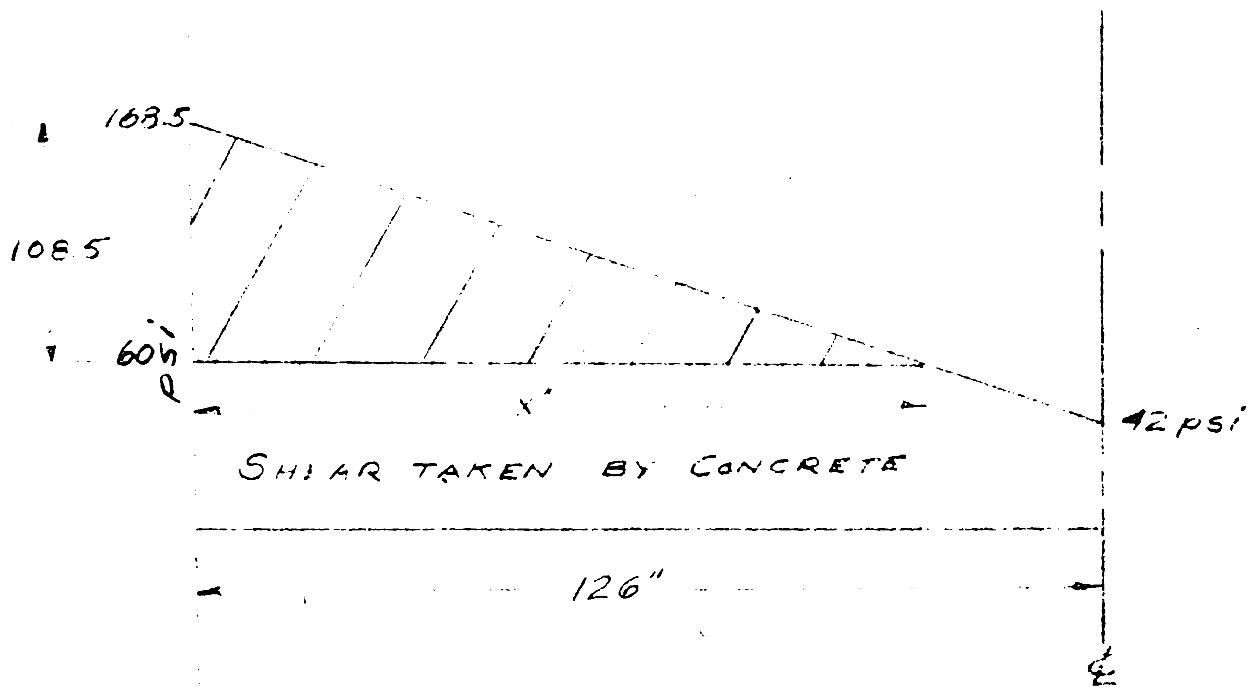


#19

SKETCHES #16-19

Minimum Spacing - $\frac{d}{2}$ = 15"

$\frac{108}{15}$ = 7.2 Reqd.   Use 8

Spacing:

From end to center.

1 @ 3"

7 @ 15"

EAST & WEST WALL BEAM DESIGN - Sketch 20

M.B.M. @ Center = 30,400# x 8' = 243,200'#(Couple)

M.B.M. = $1/8\ wl^2$

but B.M. $1/10\ wl^2$ may here be used.

Therefore 243,200 x $\frac{8}{10}$ x 12 = 2,335,000"#

Assumed weight of Stem = 300#/ft.

B.M. = $1/10\ wl^2$ = $1/10(300 + 480)(21)^2$ = 33,200"#

B.M. = 398,000"#

Total B.M. at center = 2,733,000"#

V = 30,400 + $\frac{wl}{2}$ = 30,400 + $\frac{780 \times 21}{2}$ = 38,600#

b'd = $\frac{V}{vj}$ = $\frac{38,600}{180 \times 7/8}$ = 245 sq. in.

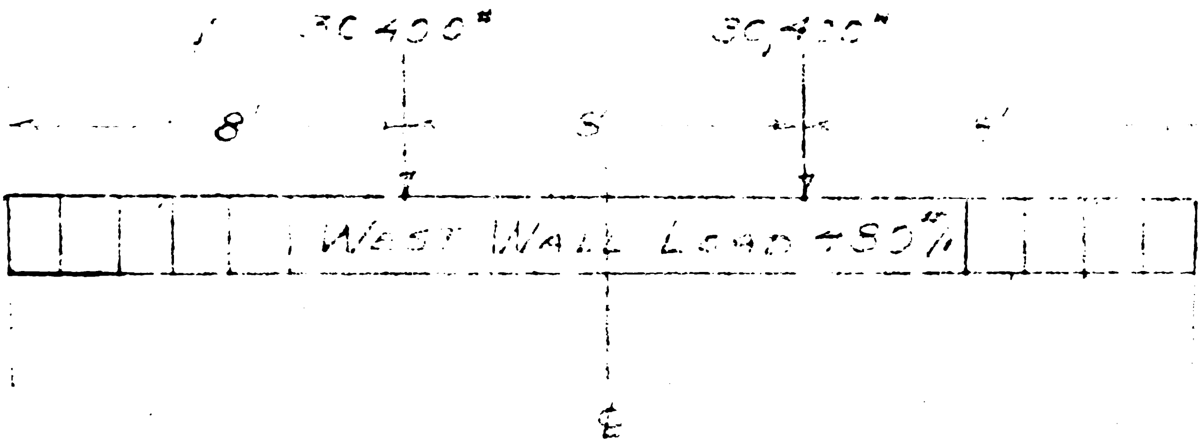
It is evident that this wall beam will be smaller than the girders, therefore computations will be discontinued and the girder design will stand for the wall beam design.

All girders and wall beams 2, 3, 12 and 13 - Same design.

NORTH & SOUTH WALL BEAM DESIGN - Sketch 21

LL (Masonry) = 704#/ft.

704 x $\frac{8}{12}$ = 470 p.s.f.



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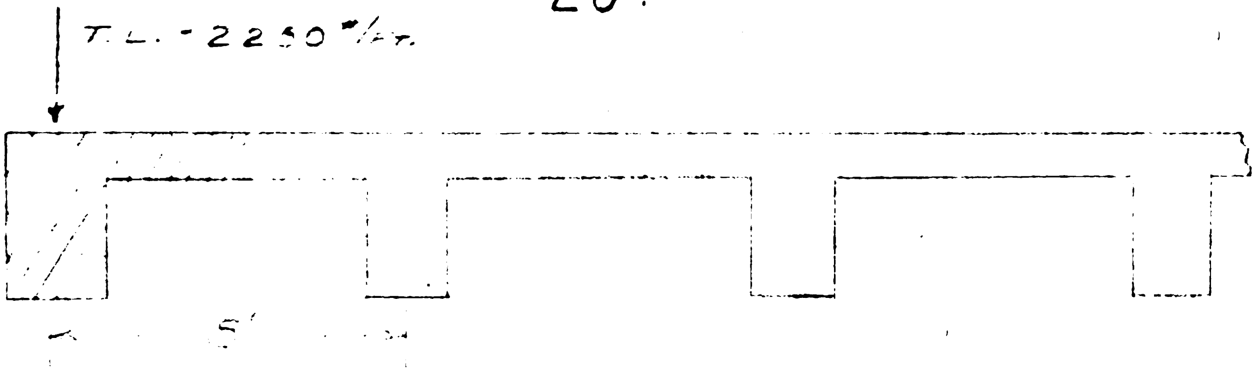

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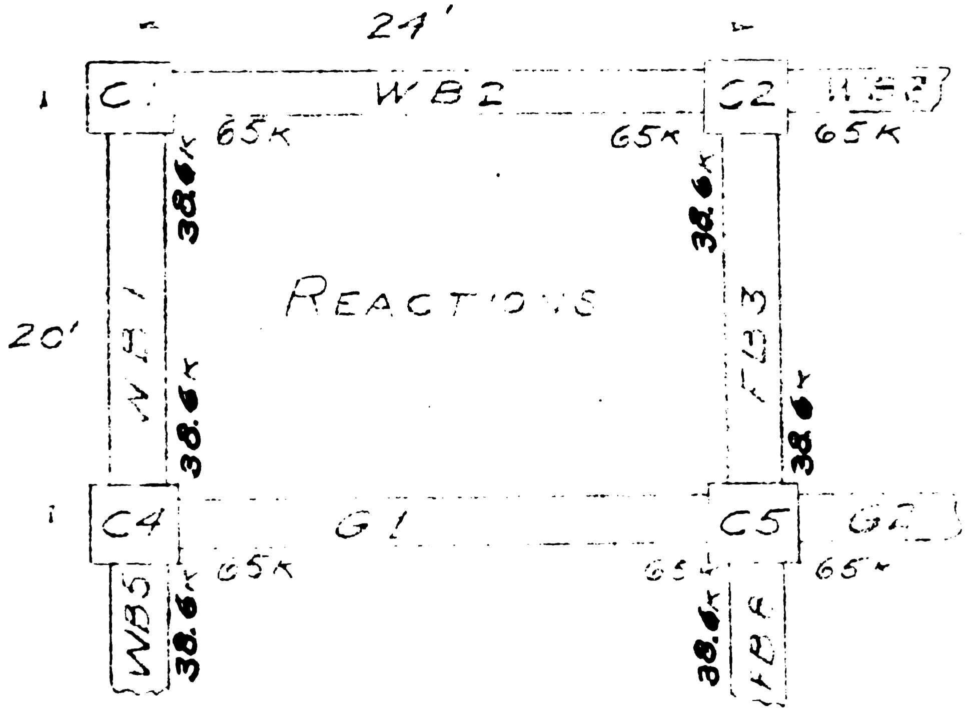


#22

SKETCHES #20-22

LL (Floor slab) $= \frac{4}{12} \times 150 = 50$ p.s.f.

Tot. LL = 520 p.s.f. = 520 x 4' = 2080#/ft.

D.L. = Assumed wt. of Stem = 200#/ft.

T.L. = 2280#/ft.

$V = \frac{wl}{2} = \frac{2280 \times 19}{2} = 21,600\#$

Allowable v (Web Reinf.) = 180 p.s.i.

Assume j = 7/8

$b'd = \frac{V}{vj} = \frac{21,600}{180 \times 7/8} = 137.5$ sq. in.

$B.M. = \frac{wl^2}{10} = \frac{2280 \times (19)^2 \times 12}{10} = 937,000"\#$

Since B.M. and V are both less than used in design of the floor beams, that same design will be adequate for wall beams 1, 4, 5, and 11.

Sketch 22 - COLUMN DESIGN

All Columns - Concentric, Axial Loading

Col. 1 Sketch 23

$f'_c = 2000$ p.s.i., $n = 15$

N = 103.6 kips

In order for the joining members to frame into this column, the dimensions of Col. 1 are set at 10' in. x 14 in.

Area = 140 sq. in.

$f_c = 2000$ p.s.i.

$N + W = f_c A \left[1 + (n - 1) p\right]$

$103,600 + \frac{140}{144} \times 150 \times \frac{111}{12} = .225 \times 2000 \times 140 \left[1 + 14p\right]$

$105,100 = 63,000 + 880,000p$

$p = .048$

$A_s = A \times p = 140 \times .048 = 6.7$ sq. in.

Use 1 1/8 A = 1.27 sq. in. x 6 = 7.64 sq. in.

For tie spacing, the least of these three conditions:

$\frac{1}{2}$" ø ties

1. Not over 16 bar diameters 16 x 1.125" = 18"
2. 48 tie diameters 48 x $\frac{1}{2}$ = 24"
3. Least dimension of column 10" *Least

Reference P. 1104b ACL 318-41

This design will apply to Cols. 1, 3, 7 and 9.

Col. 2 Sketch 24

$N = 2 \times 65\ k + 38.6 + W$

$= 168.6\ k + W$

Let Col. dimensions be 14" x 16.5"

Area = 231 sq. in.

$p = .04$

$A_s = 231 \times .04 = 9.24$ sq. in.

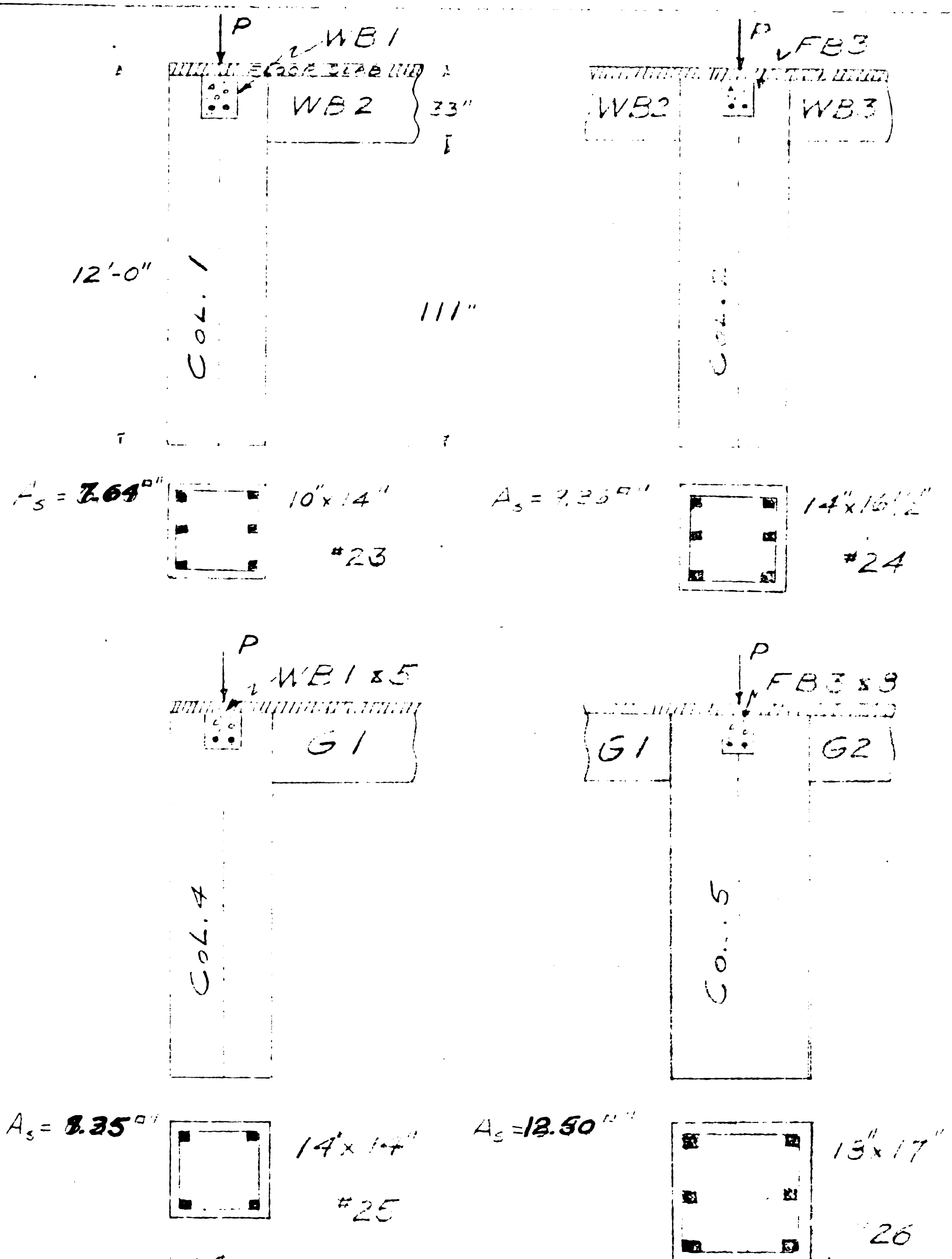
Use $1\frac{1}{4}$" A = 1.56 sq. in. x 6 = 9.36 sq. in.

$\frac{1}{2}$" ø ties

For tie spacing, the least of these three conditions:

1. 16 x 1.25 = 20"
2. 48 x $\frac{1}{2}$" = 24"
3. Least dimension of column = 14" *Least

This Design will apply to Cols. 2 and 8



½" Φ TIES USED THROUGHOUT
2" COVER " " "

SKETCHES #23-26

Col. 4 Sketch 25

$N = 77.2 + 65k = 142.2$

Let the column dimensions be 14" x 14"

Area = 196 sq. in.

$142{,}000 + \frac{196}{144} \times 150 \times \frac{111}{12} = 450 \times 196\ (1 + 14p)$

$p = .0452$

$A_s = 196 \times .0452 = 8.85$ sq. in.

Use 6 - 1¼" Area = 9.35 sq. in.

½" ⌀ ties

Ties spaced 14".

This design will apply to Cols. 4 and 6.

Col. 5 Sketch 26

$N = 2(65 + 38.6) = 207.2$ kips

Try 18" x 17" column.

$W = \frac{306}{144} \times 150 \times \frac{111}{12} = 2950$

$210{,}150 = 450 \times 306\ (1 + 14p)$

$\frac{72{,}650}{1{,}930{,}000} = p = .0377$

$A_s = 306 \times .0377 = 11.50$ sq. in.

Use 8 - 1¼" $A_s = 12.5$ sq. in.

½" ⌀ ties spaced 17"

COLUMN FOOTINGS

Col. Footing 1 Sketch 27

Col. Size 10" x 14" p = 105.1 kips

Allowable Soil Pressure = 6000 p.s.f.

LL = 105,100#

DL = 6,306# (6% LL)

TL = 111,406#

$A = \frac{111,406}{6000} = 18.6$ sq. ft. Required

4'5" x 4'5" = 19.36 sq. ft.

L = 4.4'

Net Press. $w = \frac{105,100}{19.36} = 5430$ p.s.f.

B.M. = $5430\ (1.5 \times 1.67^2 \times .6 + \frac{14 \times 20^2}{144 \times 24} = 23,400$'#

Min. $d = \frac{M}{Kb}$ Let b = least dimension of column = 10"

$d = \frac{23,400 \times 12}{236 \times 10} = 10.9"$ Say 11"

Let h = 4" (Cover)

h + d = 15" Weight = $4.4^2 \times \frac{15}{} \times 150 = 3250\#$ OK

$M = A_s f_s jd$ -

$A_s = \frac{M}{f_s jd} = \frac{23,400 \times 12}{20,000 \times .866 \times 11} = 1.475$ sq. in.

Use ½" ∅ bars A = .1963 sq. in.

$\frac{1.475}{.1963} = 7 +$ bars. Use 8

Spacing 6" c.c.

BOND STRESS

$u = \frac{V'}{o jd} = \frac{5430\ (4.4^2 - \frac{(10 + 22^2)}{144}) \times .25}{1.57 \times 8 \times .866 \times 11} = 120$ p.s.i. OK

Allowable bond stress = $f'_c \times .056 = 168$ p.s.i.

Deformed bars, hooked.

Check for Diagonal Tension

$$v = \frac{w\,L^2 - (a + 2d)^2}{2\,jd\quad(2a + 4d)} = 47.2 \text{ p.s.i.} \qquad \underline{\text{OK}}$$

Col. Footing 2 Sketch 28

Col. Size = 14" x 16.5" A = 231 sq. in. P = 168,600#

L.L. = 168,600#

D.L. = 10,100#

T.L. = 178,700#

$$A = \frac{178{,}700}{6000} = 29.8 \text{ sq. ft. Reqd.}$$

L = 5.5'

L^2 = 30.3 sq. ft.

$$\text{Net pressure } w = \frac{168{,}600}{30.3} = 5630 \text{ p.s.f.}$$

$$\text{B.M.} = 5630\left(\frac{24 \times 13\frac{1}{4}}{144} \times .6 \times 2 + \frac{14 \times 24}{144} \times 2 \times .5\right)$$

= 28,000'#

$$\text{Min. } d = \sqrt{\frac{28{,}000 \times 12}{236 \times 14}} = 9.5" \qquad \text{Say } 10"$$

Let h = 5" (Cover)

h + d = 15"

$$A_s = \frac{28{,}000 \times 12}{20{,}000 \times .866 \times 10} = 1.94 \text{ sq.in.}$$

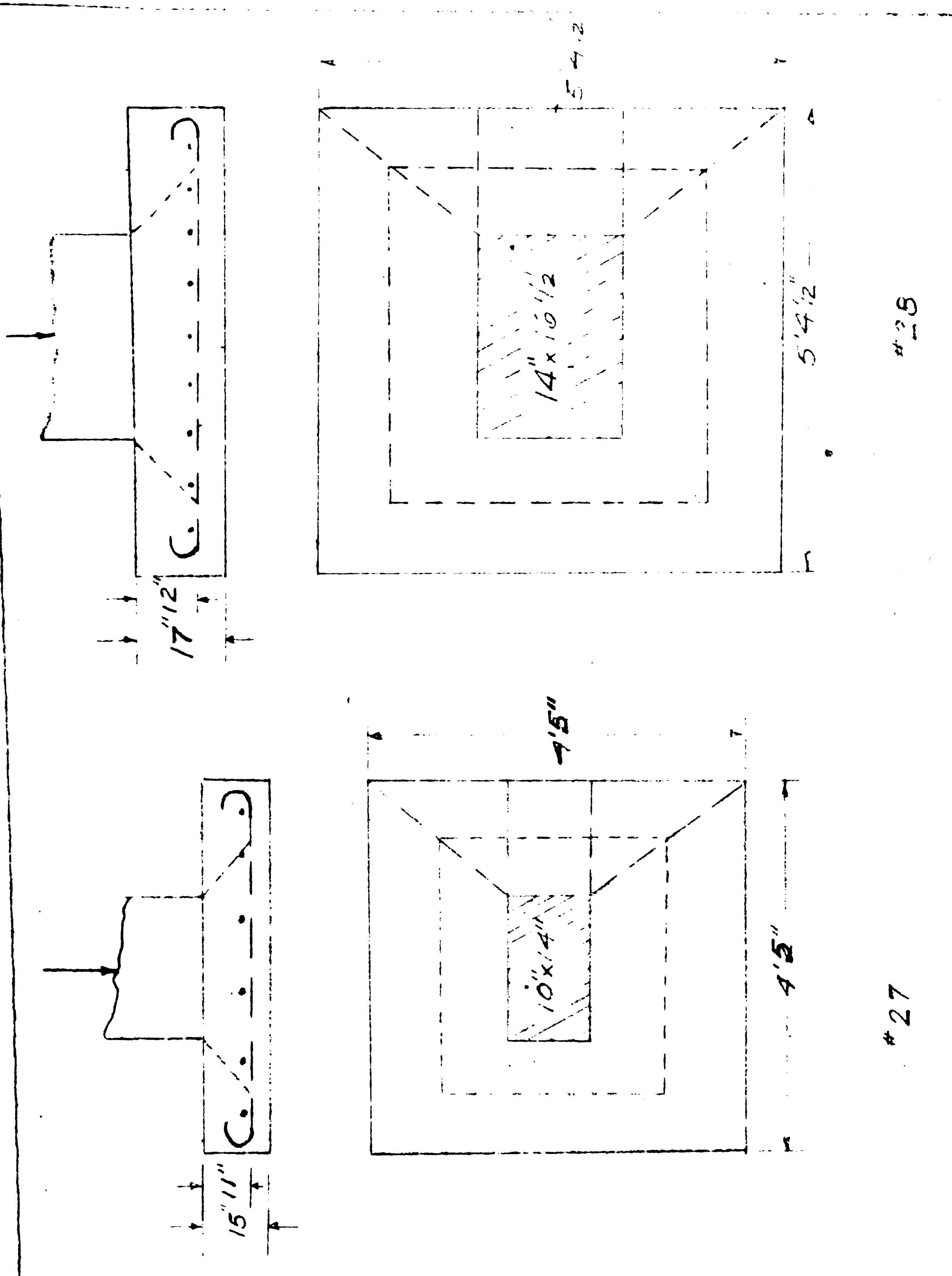
Use 3/8" ø A = .11 sq.in.

$$\frac{1.94}{.11} = 17 + \quad \text{Use 18 bars. Spacing 3" c.c.}$$

BOND STRESS

$$u = \frac{5630\left(5.5^2 - \frac{(14 + 20)^2}{12}\right) \times .25}{18 \times 1.18 \times .866 \times 10} = 168 \text{ p.s.i. } \underline{\text{OK}}$$

Use deformed, hooked bars.



SKETCHES #27-28

DIAGONAL TENSION

$v = 98$ p.s.i. Too high

Therefore, d must be increased.

let $d = 12"$

Then $v = 68$ p.s.i. CK

Allowing A_s to remain unchanged is on the coservative side and decreases the bond stress.

Col. Footing 4 Sketch 29

Col. size 14 x 14 $P = 142,200\#$

L.L. - 142,200#

D.L. - 8,540#

T.L. - 150,740#

$A = \frac{150,740}{6000} = 25.0$ sq.ft. Reqd.

Use $L = 5.0'$

Net pressure = $w = \frac{142,200}{25} = 5670$ p.s.f.

$B.M. = 5670(1.79^2 \times .6 \times 1.79 + 1.17 \times 1.79^2 \times .5)$

$= 30,000'\#$

Min. $d = \sqrt{\frac{30,000 \times 12}{236 \times 14}} = 10.5$ Say 11"

$h = 4"$ Cover

$h + d = 15"$

$A_s = \frac{30,000 \times 12}{20,000 \times .866 \times 11} = 1.89$ sq.in.

Use 3/8" ∅ $A = .11$ sq.in.

$\frac{1.89}{.11} = 17 +$ Use 18 bars · Spacing $2\frac{1}{2}"$ c.c.

<u>BOND STRESS</u>

$$u = \frac{5670\left(25 - \frac{(14 + 22)^2}{12}\right) \times .25}{13 \times 1.18 \times .866 \times 11} = 94 \text{ p.s.i. } \underline{\text{OK}}$$

Use deformed hooked bars.

<u>DIAGONAL TENSION</u>

$v = 55.5$ p.s.i. <u>OK</u>

Col. footing 5   Sketch 30

Col. size = 13" x 17"   P = 207,200#

L.L. - 207,200#

D.L. - 12,400#

T.L. - 219,600#

A = 219,600/6000 = 36.5 sq.ft.

L = 6.0'

Net pressure w = 207,200/36 = 5830 p.s.f.

$B.M. = 5830(2.125^3 \times .6 + 1.5 \times .5 \times 2.125^2) = 39{,}800'\#$

$$\text{Min. } d = \sqrt{\frac{39{,}800 \times 12}{236 \times 18}} = 10.6'' \quad \text{Say } 11''$$

h = 4" Cover

h + d = 15"

$$A_s = \frac{39{,}800 \times 12}{20{,}000 \times .866 \times 11} = 2.51 \text{ sq.in.}$$

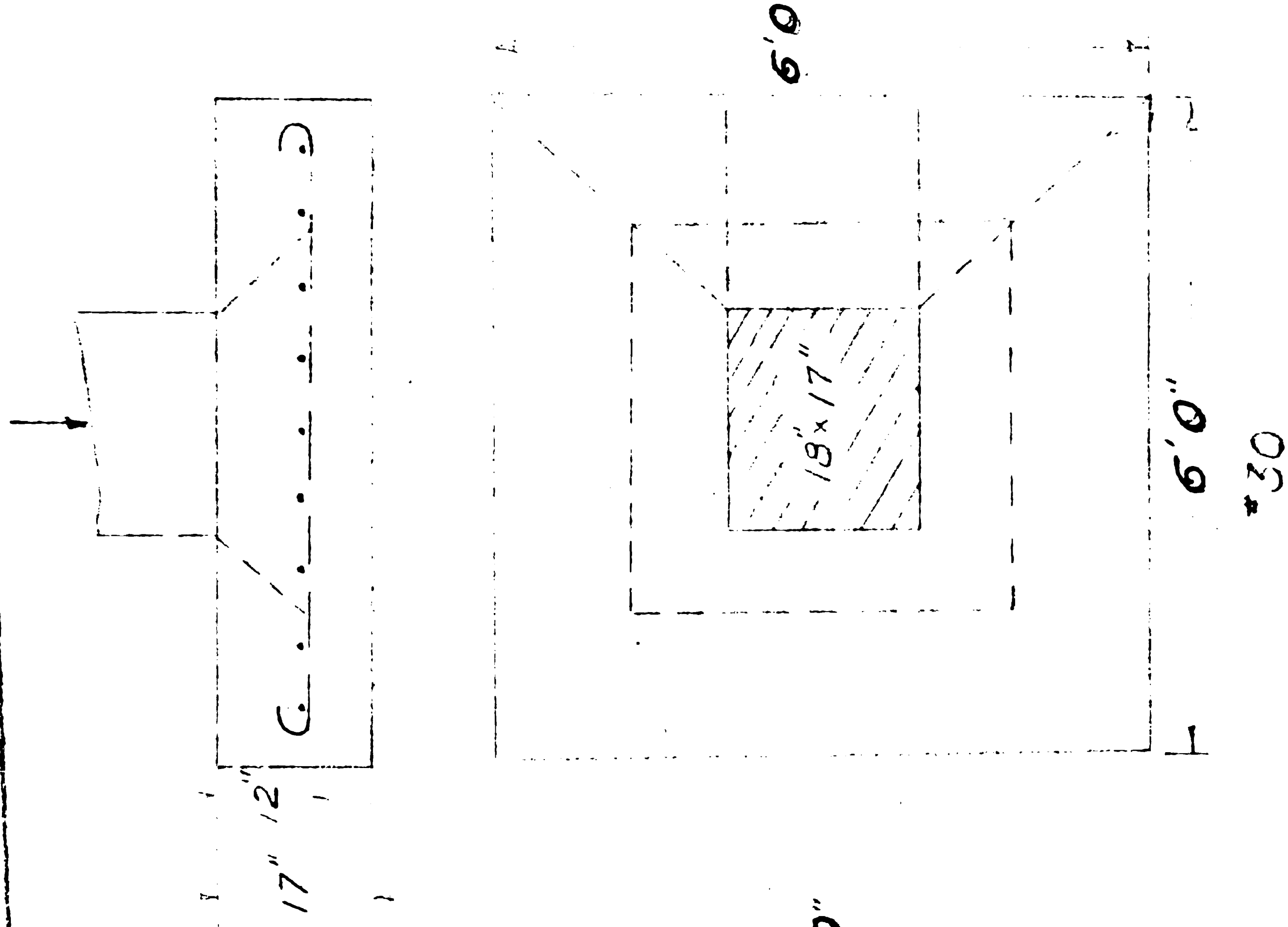
Use $\frac{1}{2}$" ø bars A = .196 sq.in.

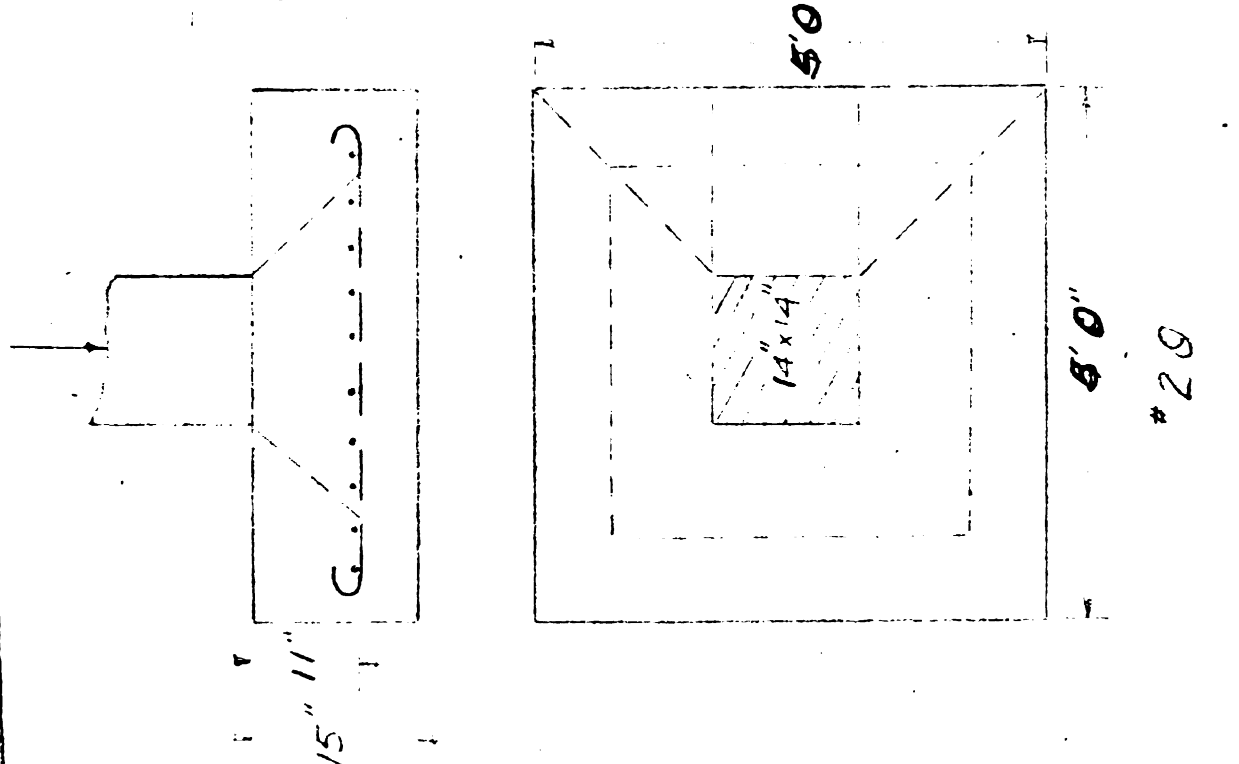
2.51/.196 = 12 +   Use 13 bars   spacing $4\frac{1}{2}$" c.c.

<u>BOND STRESS</u>

$$u = \frac{5830(36 - (17 + 22/12)^2 \times .25}{13 \times 1.57 \times .866 \times 11} = 165 \text{ p.s.i. } \underline{\text{OK}}$$

Use deformed, hooked bars.





SKETCHES #29-30

Diagonal Tension

$v = 87$ p.s.i. Too high

Increase d Let d = 12"

$v = 72$ p.s.i. OK

FLOOR DESIGN -

From an analysis of all equipment to be stored in this building, the max. wheel loading to be expected is 1250#.

Using a 3000# concrete, the max. extreme fiber stress in compression, $f_c = 1350$ p.s.i.

The bearing area of a Ford tractor tire may be taken as 10 sq. in.

$$\frac{1250}{10} = 125 \text{ p.s.i.} \quad \text{OK}$$

A sub grade of a 6 inch layer of compacted cinders will be prepared to receive the slab.

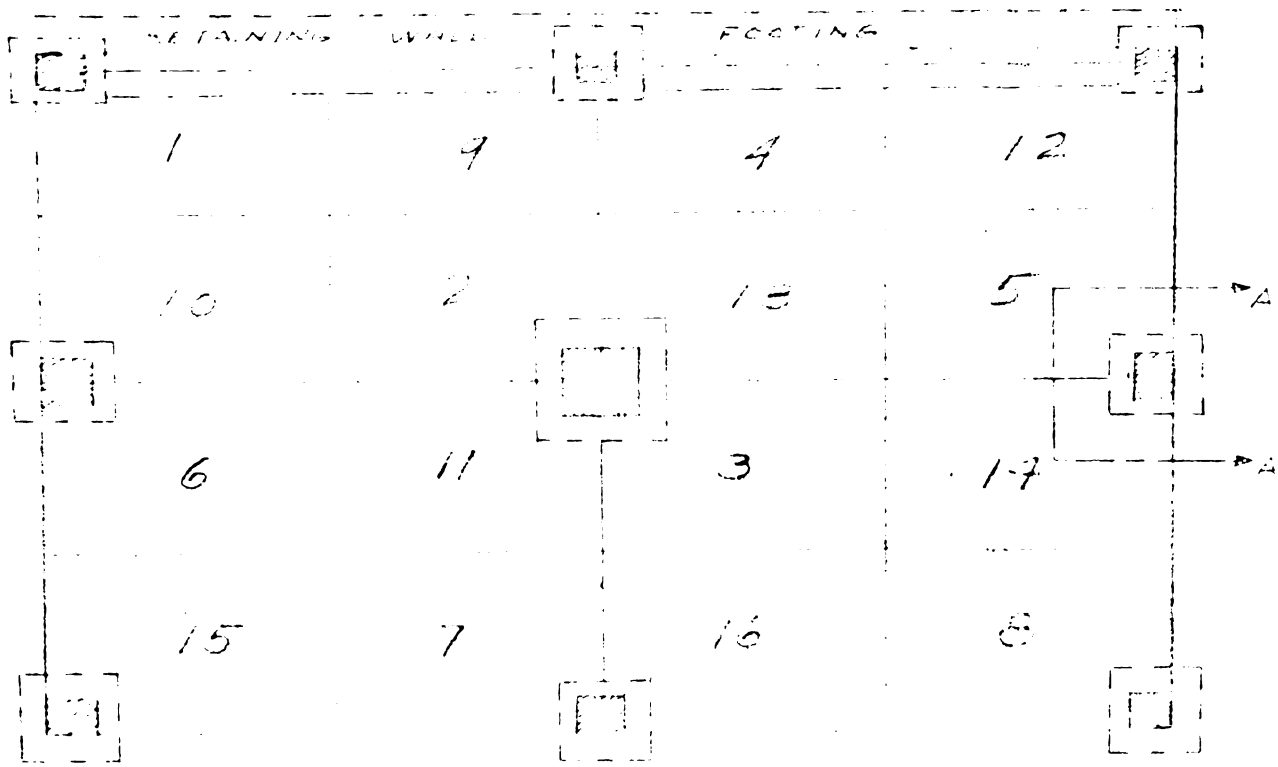
A 6" unreinforced concrete slab will be poured in alternate sections as illustrated in Sketch 31.

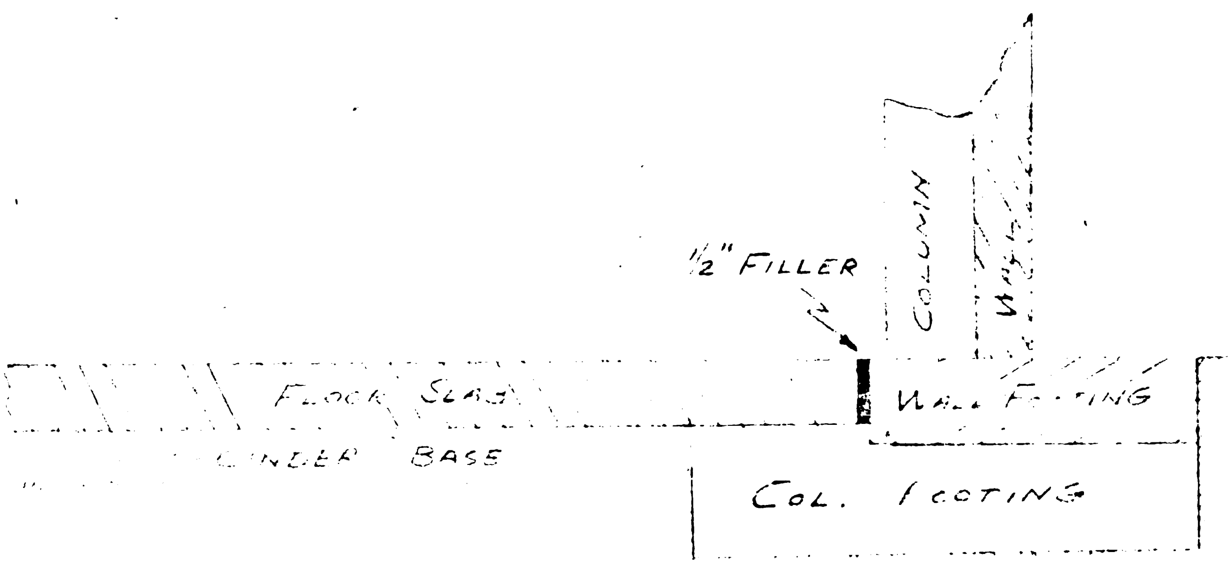
A ½" filler will be used at the junction of the floor slab and all other members such as columns, walls, etc.

DESIGN OF RETAINING WALL -

As this retaining wall is to serve as the back wall of the first floor, it must be at least 12 feet in height. As a further precaution against frost action the wall arm will be made 14 feet with 2 feet of the wall above the footing below grade.

In preparation for computing the total earth pressure





SECT. A-A

SKETCH # 31

against the wall, the equivalent surcharge of earth must be computed for all loads expected to be impressed on the drive passing by the west wall and at the loading platform.

The largest vehicle expected to park next to the loading platform is a semi-trailer as illustrated in Sketch <u>32</u>. Its wheel load of 10,000# is considered to act uniformly on an area 6' x 34', the dimensions of the tractor and trailer.

Area = 204 sq. ft.

L.L. = 4,800 + 2 x 20,000 = 44,800#

$$\text{Unit L.L.} = \frac{44{,}800}{204} = 220 \text{ p.s.f.}$$

$$\frac{220}{120} = 1.83 \text{ feet equivalent earth surcharge}$$

Total Platform Load - 1175# per Lin. Ft. of support.

Total L.L. = 1175 (18 x 2) = 42,300#

This load, too, is considered distributed over an area 6' x 18', the dimensions of the platform.

$$\therefore \text{Unit L.L.} = \frac{42{,}300}{6 \times 18} = 392 \text{ p.s.f.}$$

$$\frac{392}{120} = 3.27' \text{ Equivalent earth surcharge}$$

Total surcharge = 3.27' + 1.83 = 5.10' <u>Say 5'</u>

It is considered that an economy in the wall design out weighs the added cost of modified wall beam and column design if the loads imposed on WB2 and WB3 were transferred directly to the retaining wall as vertical loads.

This therefore will alter the design of Wall Beams 2 and 3 and also Column 2.

The thickness of the wall at the top will be 14", and there will be no batter.

By use of RANKINE'S Theory of Earth Pressure the magnitude and point of application of the earth pressure may now be calculated.

$$P = 15H(2h + H) = 6250\#$$

$$H = 16'$$

$$h = 5'$$

$$y = \frac{H(3h + H)}{3(2h + H)} = 6.35' \quad \text{distance from base}$$

$$\text{B.M. at base} = 6250 \times (6.35 - 2.00) = 27{,}200'\#$$

$$V = P = 6250\#$$

To determine "d" required at base section x - x,

(1) $M_c = kbd^2$

from which

$$d = \sqrt{\frac{27{,}200 \times 12}{236 \times 12}} = \underline{10.7"} \quad \text{Reqd.}$$

Say 11"

(2) $v = \frac{V}{bjd}$

from which

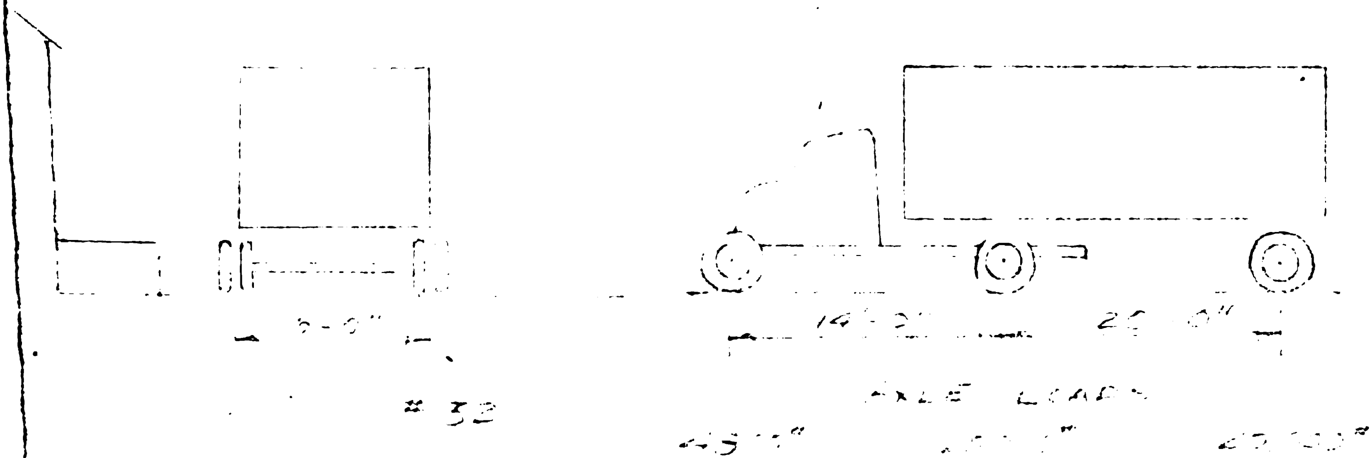
$$d = \frac{6250}{12 \times .866 \times 60} = \underline{10.0"} \quad \text{Reqd.}$$

The selected thickness of 14" provides ample cover of the maximum required "d".

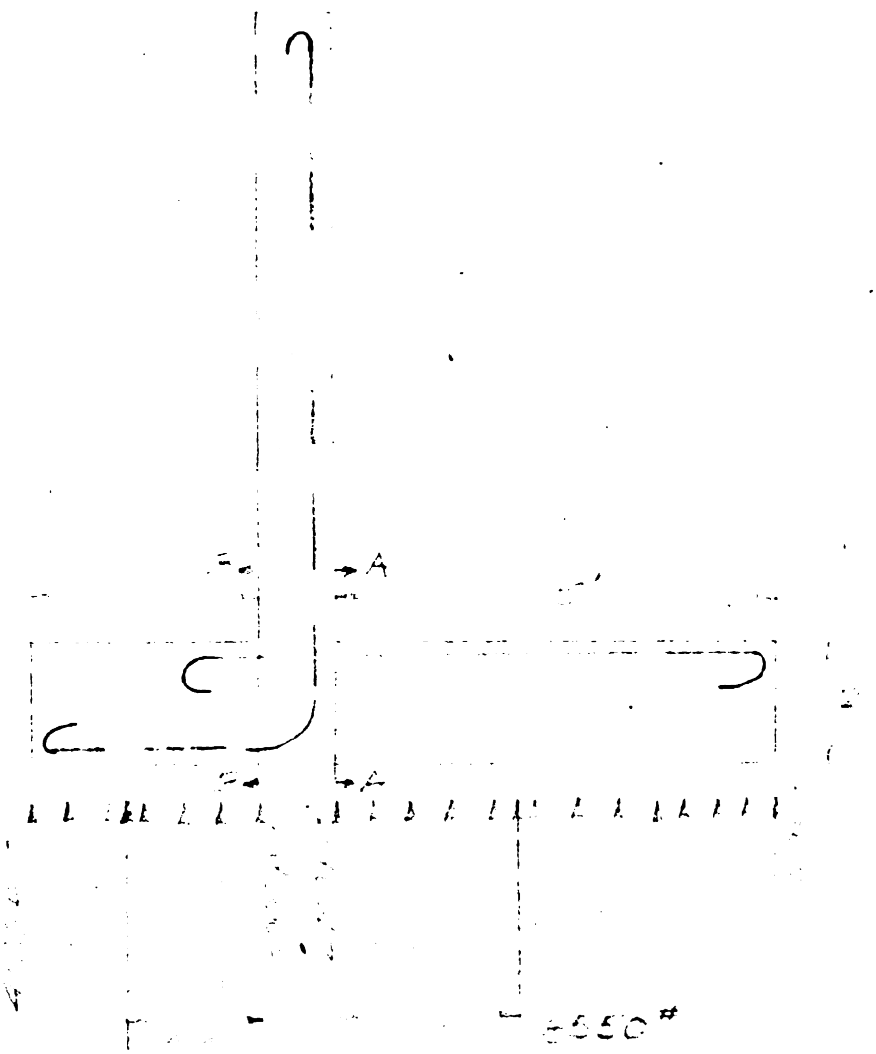
$$A_s = \frac{M}{f_s jd} = \frac{27{,}200 \times 12}{20{,}000 \times .866 \times 11} = 1.71 \text{ sq. in. Reqd.}$$

From Table 3 RCDH

Use $\frac{3}{4}$" ø Spacing 3" c.c. A = 1.76 sq. in.



32



33

12-33

Bond

$$u = \frac{6250}{9.4 \times .866 \times 11} = 70 \text{ p.s.i.}$$ OK 160 p.s.i. All.

Length of embedment required.

$$l = \frac{f_s}{4u} D = \frac{20{,}000 \times .75}{4 \times 160} = 24'' \text{ Reqd.}$$

Since there is but 24" in the base slab, this vertical tensile steel can be bent and extended into the base toward the toe in order to gain the required length. See Sketch 33

Temperature steel will be used in both the front and back faces of the wall as illustrated in DRAWING NO. 4 .

DESIGN OF BASE SLAB - HEEL

With the base slab dimensions selected as follows:

Thickness - 2'

Toe - 3' Sketch 33a

Heel - 5'

The resultant passes through the base at a point 3.5 feet from the toe, within the middle third.

Factor of Safety against overturning - $n = \frac{1}{1-Za} = \frac{9.16}{9.16-7} = 4.25$

P - earth pressure = 6250#

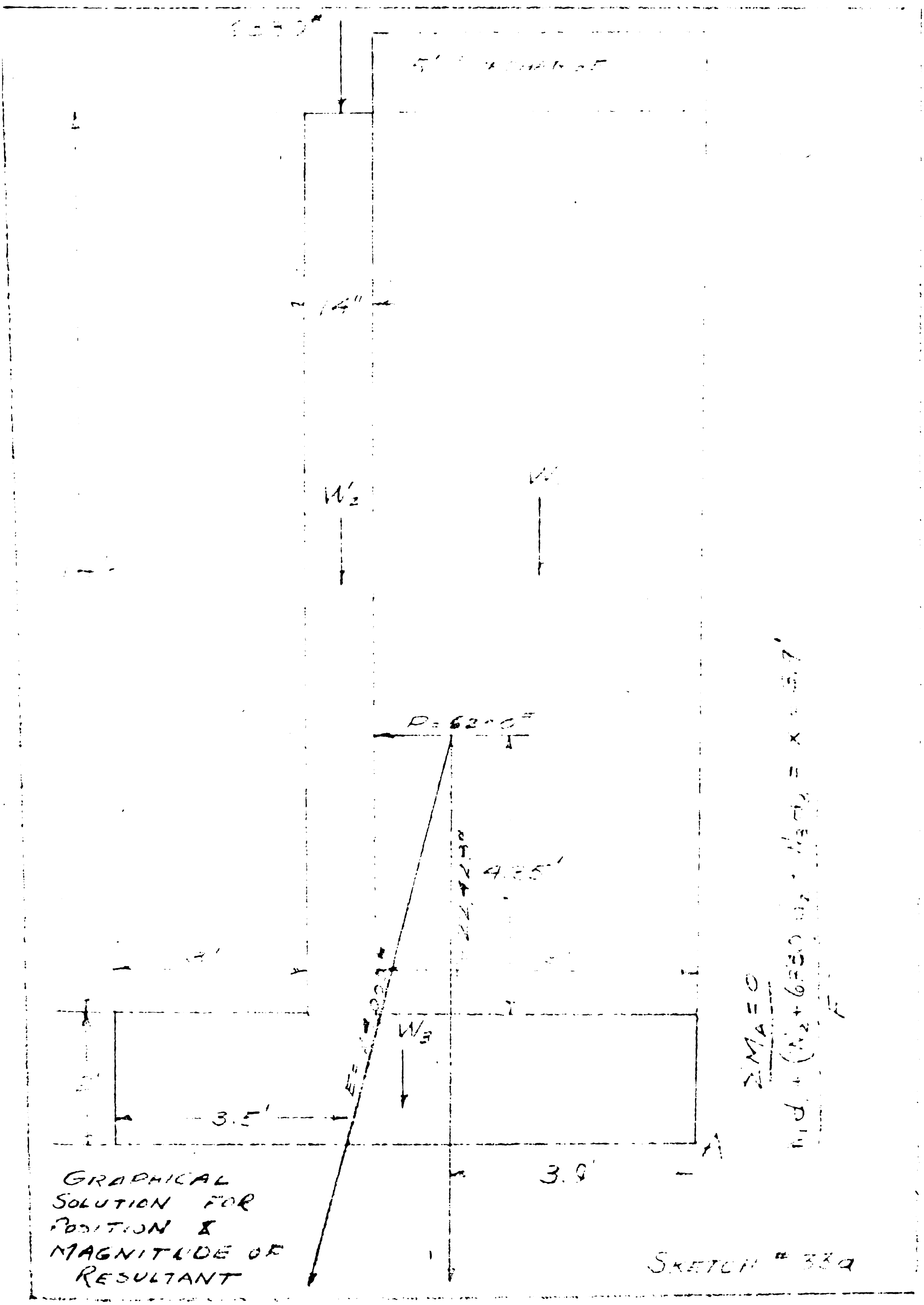
F - vertical component of Resultant = 22,428#

E - Resultant = 24,000# See Sketch 33

To solve for foundation pressures Sketch 33

$$P \text{ max.} = (4l - 6a) \frac{F}{l^2} = 4170 \text{ p.s.f.}$$ OK 6000 p.s.f. All.

$$P \text{ min.} = (6a - 2l) \frac{F}{l^2} = 720 \text{ p.s.f.}$$



14"
W'_2
W_3
3.5'
A
3.0'
$\Sigma M_A = 0$
GRAPHICAL SOLUTION FOR POSITION & MAGNITUDE OF RESULTANT

SKETCH # 33a

$$M_{AA} = 11{,}400 \times 2.5 + 1500 \times 2.5 - \left(\frac{2700 + 720}{2}\right) \times 5 \times 2.9$$

$$= 10{,}850'\#$$

$$V_{AA} = 11{,}400 + 1500 - 1710 = 11{,}190\#$$

$$d \text{ (for moment)} = \sqrt{\frac{10{,}850 \times 12}{236 \times 12}} = \underline{6.8'' \quad \text{Reqd.}}$$

$$d \text{ (for shear)} = \frac{11{,}190}{60 \times .866 \times 12} = \underline{18'' \text{ Reqd.}}$$

A slab thickness of 24 inches was tentatively used and thus it is evident that it will more than suffice.

$$A_s = \frac{10{,}850 \times 12}{20{,}000 \times .866 \times 18} = .42 \text{ sq. in. Reqd.}$$

From Table 3 RCDH

Use 3/8" ∅ Bars Spacing 3" c.c.

A = .44 sq. in.

<u>BOND STRESS</u>

$$u = \frac{11{,}190}{4.7 \times .866 \times 18} = 153 \text{ p.s.i.}$$

For sufficient embedment length:

$$l = \frac{20{,}000}{4 \times 160} \times 3/8 = 11.7''$$

- TOE DESIGN -

$$M_{BB} = \left(\frac{4170 + 3000}{2}\right) 2.2 - (3 \times 2 \times 150)\, 1.5 = 6537'\#$$

$$V_{BB} = 3585 - 1350 = 2235\#$$

$$d \text{ (for moment)} = \sqrt{\frac{6537 \times 12}{236 \times 12}} = 5.75'' \quad \underline{\text{Reqd.}}$$

$$d \text{ (for shear)} = \frac{2235}{60 \times .866 \times 12} = 3.58'' \ \underline{\text{Reqd.}}$$

A slab thickness of 24" was used for the heel, therefore the toe will be the same.

A d = 18" was selected.

$$A_s = \frac{6537 \times 12}{20{,}000 \times .866 \times 18} = .253 \text{ sq. in. Reqd.}$$

No tension steel will be designed for the toe section as the vertical steel of the stem will be bent in the slab toward the toe in order to obtain sufficient embedment length. This steel, $\frac{3}{4}$" ø 3" c.c. will more than provide the required steel area in the toe.

- FACTORS OF SAFETY -

Overturning

F.S. = 4.25

Sliding

$$F.S. = \frac{22{,}428 \times \tan 25^{\circ}}{6250} = 1.68$$

Crushing

$$F.S. = \frac{6000}{4170} = 1.44$$

- POINTS OF CUT-OFF FOR STEM VERTICAL STEEL -

Bending moments will be computed for 1' increments of height of the stem. Area of steel required for each bending moment will be computed and it will then be a simple matter to determine points at which steel may be cut-off.

Height from top	B.M.	A_s Reqd.
1	80 Ft. Lbs.	.005 Sq. In.
2	340	0.022
3	810	0.051
4	1520	0.096
5	2500	0.157
6	3780	0.238
7	5390	0.339
8	7360	0.463
9	9720	0.610
10	12,500	0.785
11	15,730	0.990
12	19,440	1.220
13	23,660	1.490
14	27,200	1.710

$$A_s = \frac{M \times 12}{f_s\ jd} = \frac{M}{159{,}000}$$

f_s = 20,000 p.s.i.

j = .866

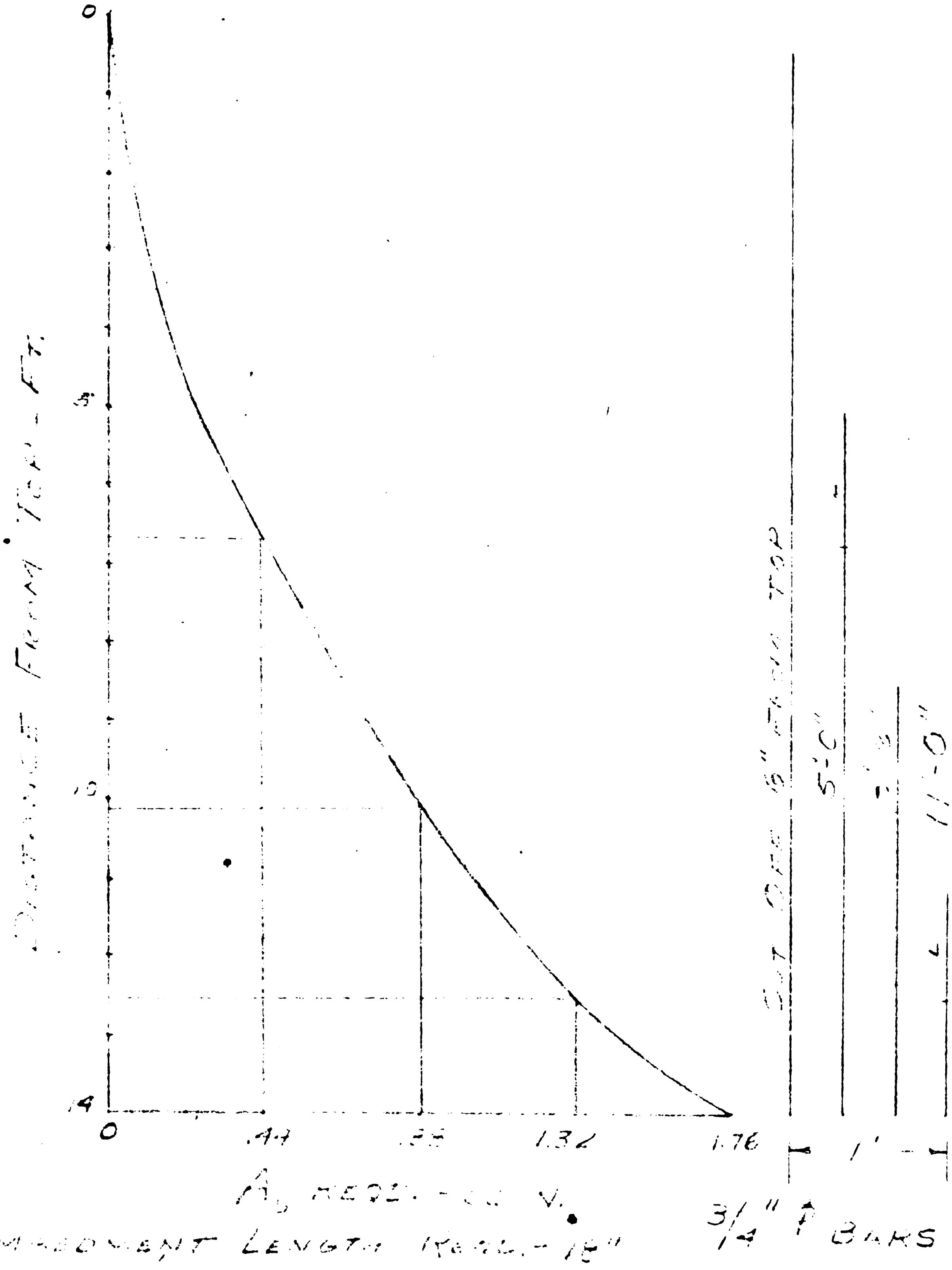
d = 11"

4 - ¾" ø Bars   3" Spacing

Add Embedment Length - L

$L = \frac{f_c}{4_u} \times D = \underline{18"}$

For points of cut-off see Sketch <u>34</u>.



SKETCH #34

WALL POUR SCHEDULE

All column and wall footings will be completed in one pour. Dowels will be placed in these footings of the same diameter as the vertical steel in these respective joining members. Dowels will project 25 bar diameters beyond the footings. L = 18"

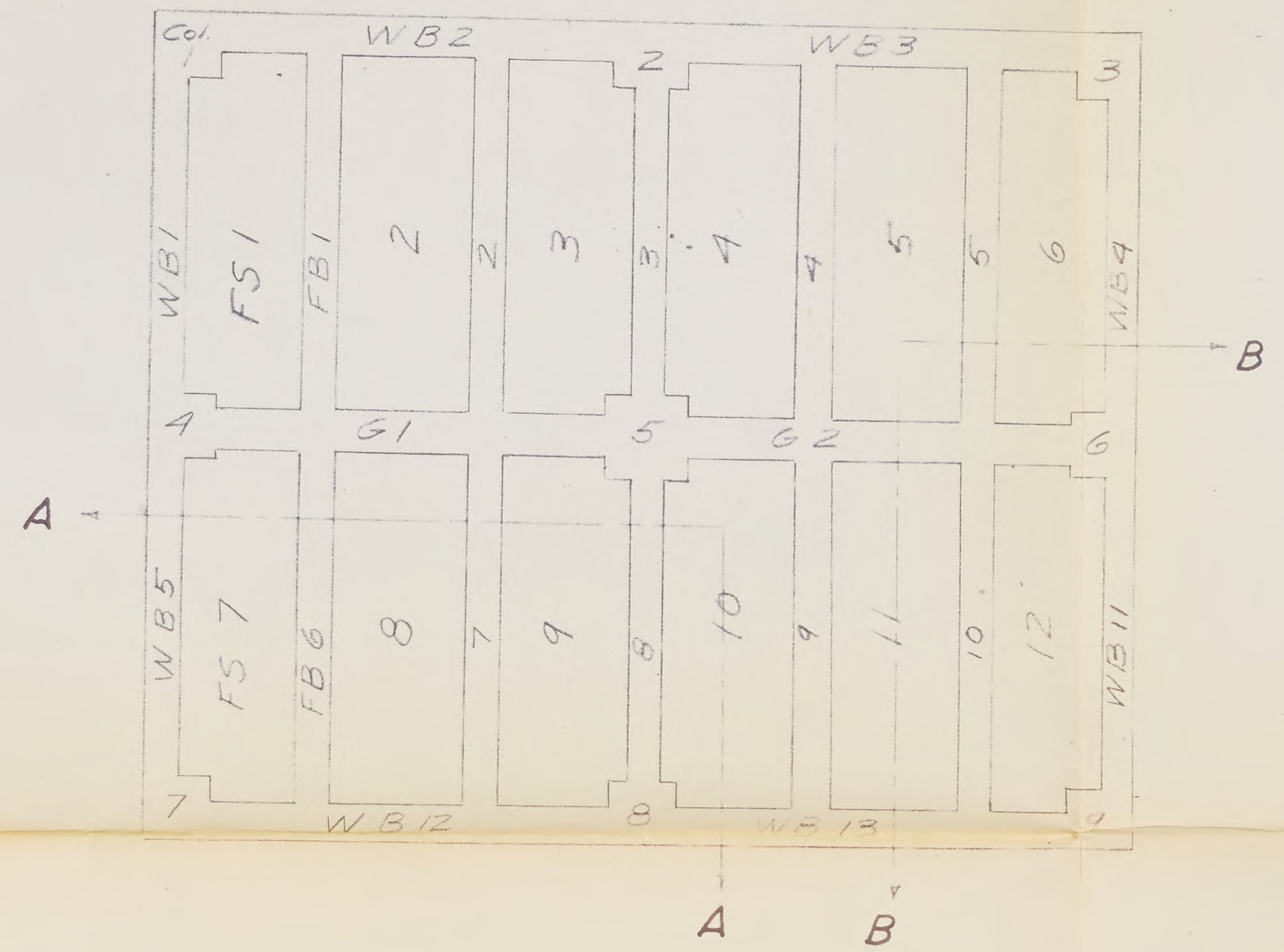
A key way will be formed in the top surface of the wall by placement of a 2 x 6 board on the centerline while the concrete is still in the plastic state. Before pouring the stem, the footing surfaces will be thoroughly cleaned, scoured, and a $\frac{1}{2}$ inch layer of neat cement paste applied before the wall pour begins.

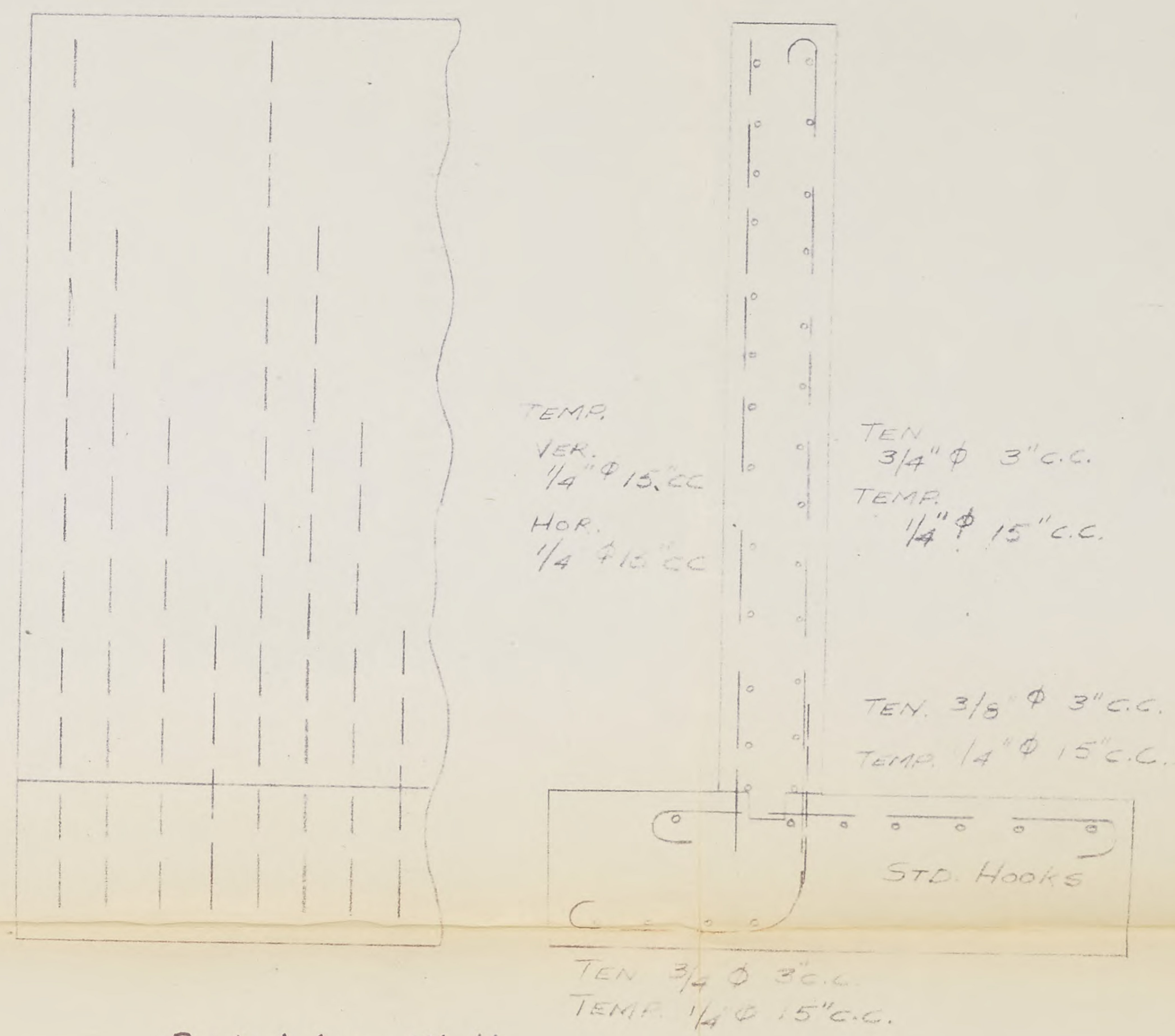
The wall proper, excluding the columns will be poured in two lifts. This is deemed necessary for proper puddling. Form work will be placed for the front or inside face of the wall for the full height - 14'. Form work for the outside face will first be placed up to 7' for the first lift. Here again, a keyway will be placed on the top surface of the first lift. The second lift will complete the wall to grade, the bottom of the floor slab.

To provide anchorage for these two members, dowels bent 90° will be placed so that 25 Dia. extend into both the wall and floor slab. Use $\frac{3}{4}$" ø bars. L = 4'.

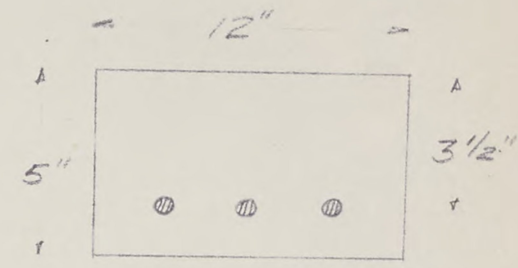
To provide anchorage of the wall and the columns, the longitudinal temperature steel of the wall will be extended 25 Dia. into these columns. L = 16".

SECTIONS



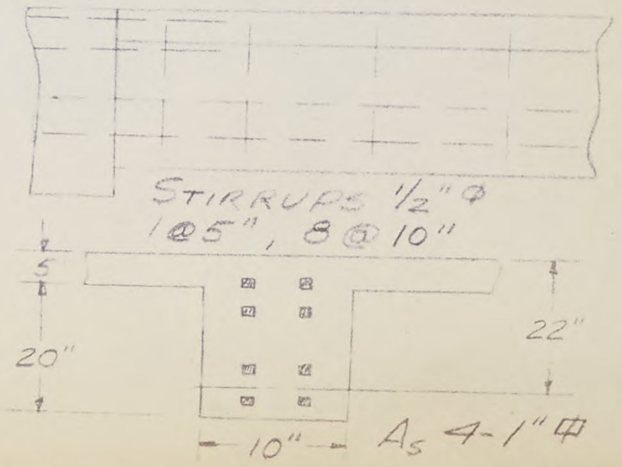


Retaining Wall



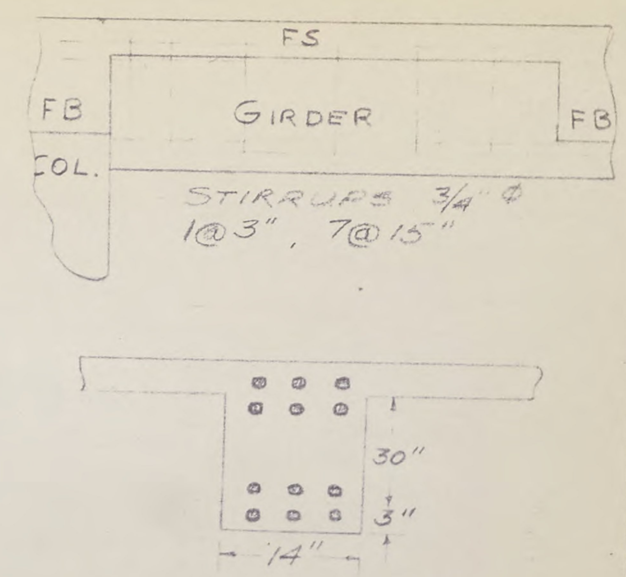
A_s 3 - 1/2" Φ 4" c.c.

Floor Slab



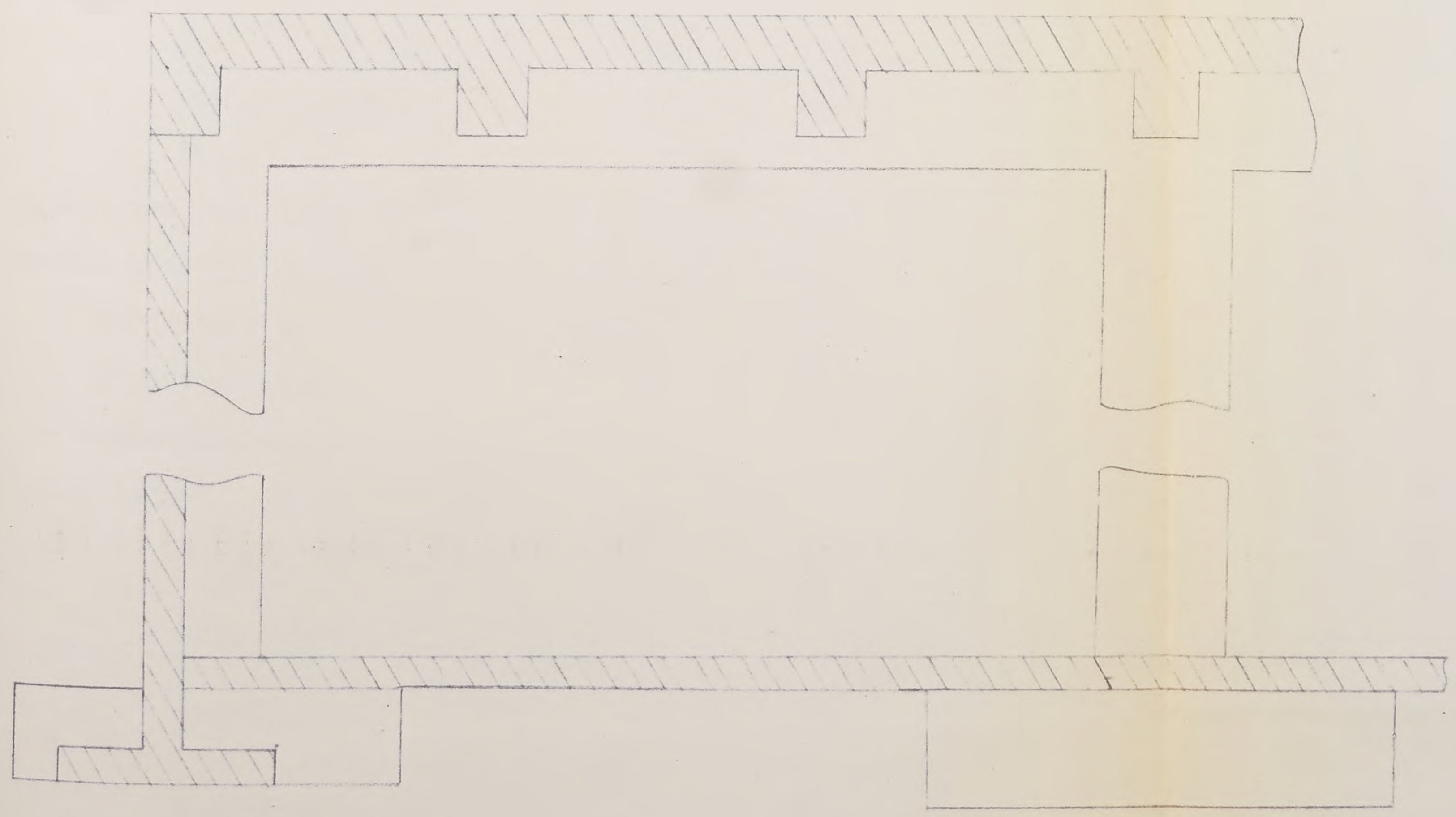
A_s 4 - 1" Φ

Floor Beam

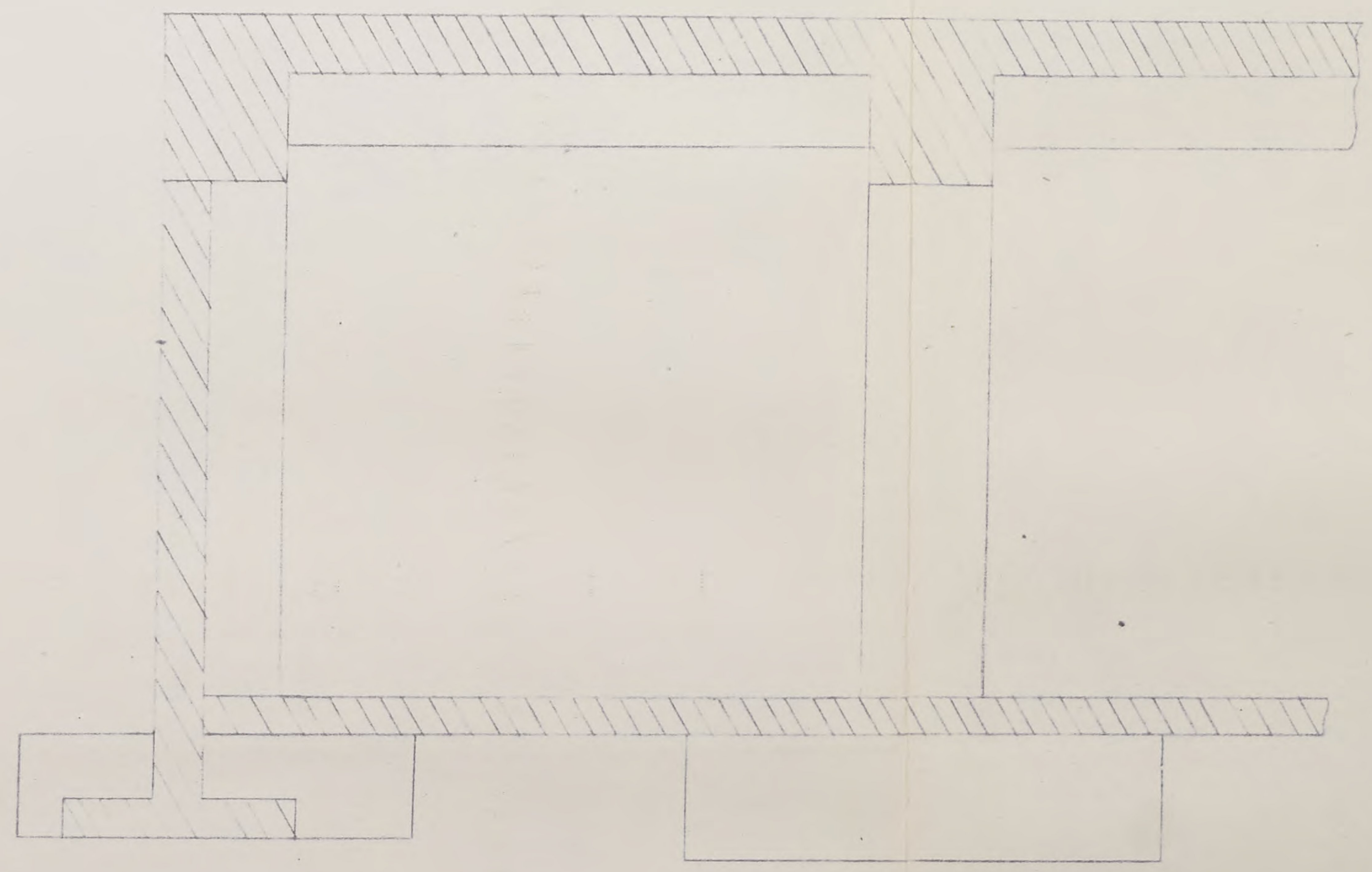


A_s 6 - 1 1/4" Φ, 3 1/2" c.c.

Girder



Sect. A-A



Sect. B-B

FRAMING PLAN

Dwg. No. 4

Upon reference to "Reinforced Concrete Design" by Sutherland and Reese, it is decided to build this wall in one section. The author prescribes a maximum wall section length of 60 ft. without putting in expansion joints.

- STAIR DESIGN -

Stairs located and dimensioned as on Sketch 35

L.L. - 100 p.s.f.   Use 6 inch Slab

D.L. - 75 p.s.f.

T.L. - 175 p.s.f.

Total rise from top of floor to first landing - 6 feet.

Stairs with 10" run, 6" rise and 1" nosing will be used.

Horizontal projection of stairway = 9.16'

Platform slab 4' length, 8' width

The stairway slab and platform slab will be designed as one slab, length = 13.16'.

$$V = \frac{wl}{2} = \frac{175 \times 13.16}{2} = 1150\#$$

$$B.M. = \frac{wl^2}{10} = \frac{175 \times 13.16^2}{10} = 3030'\#$$

$$d \text{ (for shear)} = \frac{1150}{12 \times .866 \times 60} = 1.85''$$

$$d \text{ (for moment)} = \sqrt{\frac{3030 \times 12}{12 \times 236}} = 3.6''$$

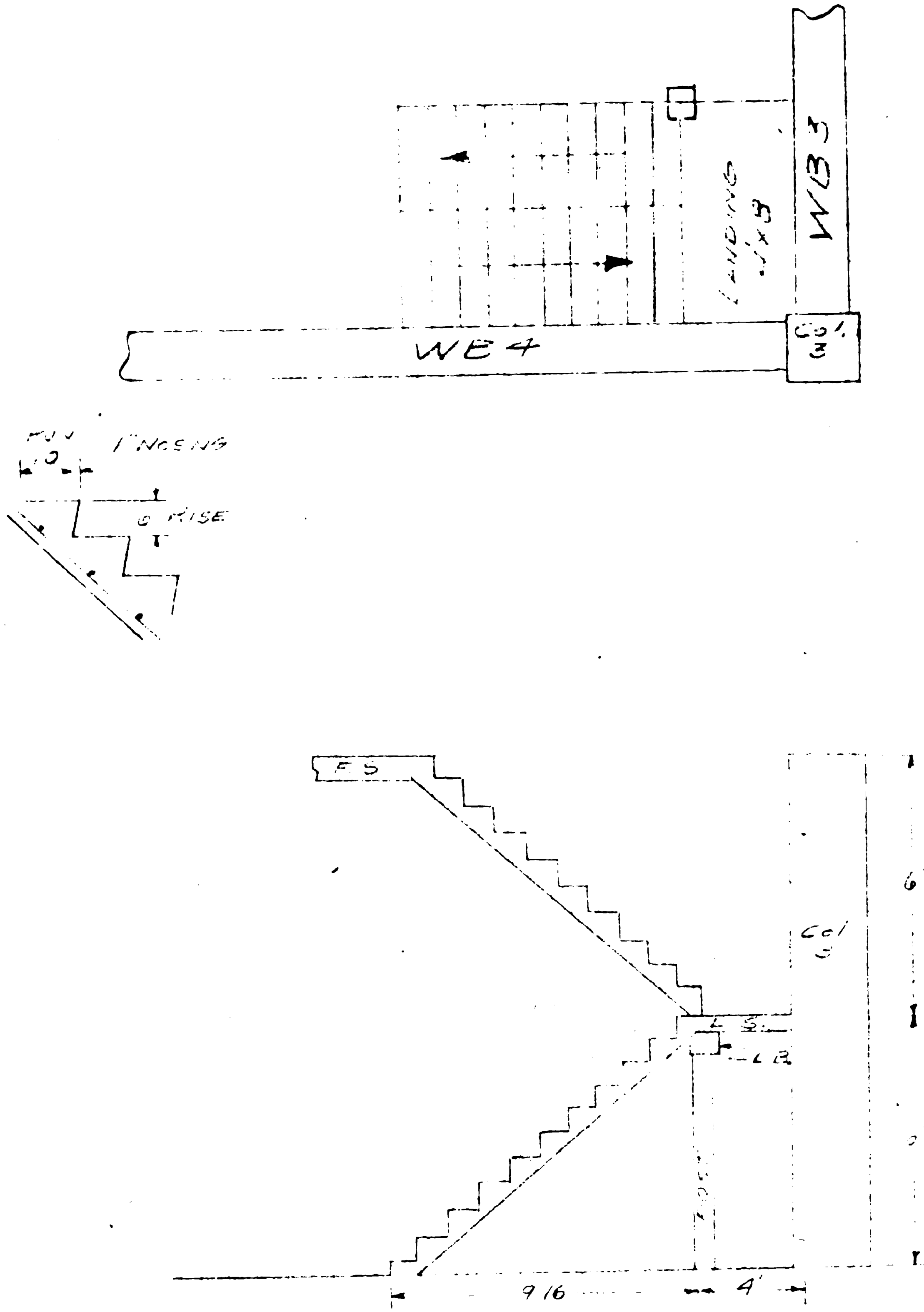
Use d = 4"

$$A_s = \frac{3030 \times 12}{20{,}000 \times .866 \times 4.0} = .525 \text{ sq. in. per ft.}$$

Use 3/8" ø bars   Spacing $2\frac{1}{2}$" c.c.

BOND - $u = \frac{1150}{5.7 \times .866 \times 4} = 58.2$ p.s.i.

The lower and upper stairway slabs are fixed to the landing



SKETCH #35

slab along the inside edge. The landing slab, 4 ft. long is fixed in the west wall. The stairway slab reinforcement is continued through the landing slab and anchored into the wall a length:

$$l = \frac{20000}{4 \times 120} \times 3/8 = 16"$$

This same reinforcement will be run along the 8 ft. length of the landing slab.

A beam will be designed to support the east or inside edge of the landing slab. The north end of this beam will be fixed in the north wall and the south end of this 8 ft. beam will be supported by a short column.

<u>Design of landing slab beam</u> -

Stairway slab reaction = 1150#/ft.

6 inch landing slab = 75 #/ft.

$$R = \frac{4 \times 75}{2} = 150\#/ft.$$

Landing slab reaction = 150#/ft.

Total load on landing beam = 1300#/ft.

Assume weight of stem = 20#/ft.

T.L. = 1320#/ft.

$$V = \frac{1320 \times 8}{2} = 5280\#$$

$$v = \frac{V}{b'dj} \quad , \quad b'd = \frac{5280}{180 \times 7/8} = 33.6 \text{ sq. in. reqd.}$$

Web reinforcement needed - stirrups

Let b' = 4" then d = 9"

d ≁ 2" = 11"

≁ = 6"

$11 - 6 = 5''$

$\frac{5 \times 4}{144} \times 150 = 20.4\#/'$ Weight OK

$B.M. = \frac{1320 \times 64}{8} = 10{,}560'\#$

$j = .92$

$B.M. = Tjd$

$T = \frac{10{,}560 \times 12}{.92 \times 9} = 15{,}300\#$

$A_s = \frac{15{,}300}{20{,}000} = .765$ sq. in. reqd.

Use 2 - $\frac{3}{4}''$ ø bars $A_s = .88$ sq. in.

BOND

$u = \frac{5280}{4.7 \times .92 \times 9} = 136$ p.s.i. Use deformed bars.

Maximum shear stress at ends:

$v_\perp = \frac{5280}{4 \times .92 \times 9} = 160$ p.s.i.

Max. shear stress at half span = $v_1 \times 25\% = 40$ p.s.i.

Total shear taken by stirrups:

$V = \frac{100 \times 3.33' \times 4 \times 12}{2} = 8000\#$

Using $\frac{1}{4}''$ ø stirrups, A = .05 sq. in. 2A = .1 sq. in.

.1 x 16,000 p.s.i. = 1600# taken by each stirrup

$\frac{8000}{1600} = 5$ stirrups Reqd.

Minimum stirrup spacing = $\frac{d}{2} = 4.5''$

$\frac{3.33' \times 12''}{4.5} = 9$ stirrups @ 4.5" c.c. at each end.

Landing Post Design -

Let A = 4 x 12 = 48 sq. in. Gross area.

$p = .02$

$A_s = 48 \times .02 = .96$ sq. in.

Use 4 - $\frac{1}{2}$" ϕ bars $A = 1.00$ sq. in.

$$Wt. = \frac{48}{144} \times 515 \times 150 = 275\#$$

Beam reaction = 5280#

Total axial load at base = 5555#

$\frac{5555}{48} = 1155$ p.s.i. Compression stress upon floor.

Allowable = 1350 p.s.i., therefore no footing is required for this post.

PARTITION WALLS

The North, South and East lower floor walls will be concrete partition walls.

A tentative thickness of 6" was arrived at by the use of

$$t = \frac{10' \times 12}{30} = 4''$$

Assuming a wind load of 30 p.s.f. as before in the roof truss loading, this thickness is investigated.

This wall will be independent of the inclosing members such as the upper wall beams and wall columns. Therefore it will be assumed that there will be no transfer of loads from these members to the wall.

The maximum wall area under action of wind loads will be taken as $\frac{1}{2}$ of the East wall minus the door area, the column area, and the wall beam area.

$$A = \tfrac{1}{2}(12 \times 48) - \left(\frac{14}{12} \times 12 + \frac{33}{12} \times 24 + 12 \times 10\right)$$

$A = 90$ sq. ft.

Total wind load = $30 \times 90 = 2700\#$

Unit wind load = $\frac{2700\#}{13'} = 208\#$/Lin. Ft.

This force may be considered to act at the third point from the base. Height = 12 feet - 33 inches = 111"

$$B.M. = \frac{208 \times 111}{3} = 770 \text{ in. lbs.}$$

$$A_s = \frac{770}{20{,}000 \times .866 \times 4} = .011 \text{ sq. in. per ft.}$$

Use $\frac{1}{4}$" ø $5\frac{1}{2}$" c.c. spacing

$$\text{Shear} = v = \frac{208}{12 \times .866 \times 4} = 5 \text{ p.s.i.} \quad OK$$

BOND

$$u = \frac{208}{2 \times .79 \times .866 \times 4} = 38 \text{ p.s.i.} \quad OK$$

Temperature Steel $.002\ A_c$

$$A_s = \frac{111 \times 6}{144} \times .002 = .0093 \text{ sq. in. Reqd.}$$

Use $\frac{1}{4}$" ø 12" c.c. Spacing

Door Lintel

There is no appreciable load of partition wall above the 10' doors, therefore lintel is deemed unnecessary at these locations.

Special reinforcement will however be placed at the top corners of the door openings in the wall.

PARTITION WALL FOOTINGS

For a 1 ft. section of wall, 6" thickness, height 117"

$$W = .5 \times 1 \times \frac{117}{12} \times 150 = 730\#/\text{lin. ft.}$$

Assume wt. of footing = 150#/lin. ft.

T.L.= 880#/lin. ft.

$\frac{880}{6000}$ = .147 sq. ft. bearing area Reqd.

Use a 2' width 6" thickness

No reinforcement needed but temperature steel will be used.

Use $\frac{1}{4}$" ø bars 15" c.c. Spacing

For Drawings -

Girder Stirrups -

$\frac{3}{4}$" ø 1 @ 3" 7 @ 15"

WATER DEMANDS

(1) Irrigation

3 acres @ $\frac{1}{4}$ in. - acre/week

Each acre to be irrigated once every 6 days

$43{,}560 \times \frac{.25}{12} = 967$ cu. ft./acre/2 days

$Q = 967 \times 7.48 = 7240$ gal./2 days

This quanity of water to be pumped in about 10 - 15 hours and stored for 2 days.

Tank must have min. capacity of 7240 gal.

Pump and well must be able to supply - $\frac{7240}{10} = 724 \frac{\text{Gal.}}{\text{Hr.}}$

or

$\frac{7240}{15} = 485 \frac{\text{Gal.}}{\text{Hr.}}$

(2) Livestock and Poultry Reqm.

Cattle	- 20 Hd.	@ 10 g.p.d.	-	200 g.p.d.
Pigs	- 10	@ 2	-	20
Poultry	200	@ 5g/100	-	10
		Total	-	230 g.p.d.

(3) Spraying Reqm.

The spraying equipment, capacity 400 gal., is used in the spring, summer and fall. Spraying services are required mostly in the spring and fall, therefore any demand for this purpose will not be added to the other demand for the total figure.

The max. spraying accomplished in one 24 hr. period is 5 loads - $5 \times 400 = 2000$ gals.

It may be stated that max. demand conditions will occur in the summer when irrigation will be carried on at almost a constant rate. To this irrigation demand will be added the constant poultry and livestock demand.

$$Qt = 7240 + (2 \times 230) = 7700 \text{ gal.} \quad \text{(Every 2 days)}$$

$$\frac{7700}{10} = 770 \text{ gal./hr.} \quad \text{Rate for a 10 hr. pumping period.}$$

Assuming the water table at 75' below Gd. Surface.

A 4" well casing used

Using these conditions, different type pumps will be selected from "Deming Pump Catalog" and a final choice made.

1 pipe jet	2 pipe jet	Force Pump
Unit No. S-660	Unit No. C-660	10150
$1\frac{1}{2}$ HP	$1\frac{1}{2}$ HP	$1\frac{1}{2}$ HP
Capa. - 845 g/hr.	845 g/hr.	855 g/hr.
$\frac{7700}{845} = 9.1$ hr. of pumping	$\frac{7700}{845} = 9.1$ hr.	$\frac{7700}{855} = 9$ hr.
4" Casing	But, a 5" casing must be used.	$3\frac{1}{4}$" Casing
Unit Cost $266.00	$262.00	$383.00

The 1 pipe jet pump will be installed because of its low initial cost and size of well casing required.

- STORAGE TANK DESIGN -

The pump unit selected was capable of pumping against a 20 p.s.i. tank pressure. Since a pressure tank is not being used , the maximum allowable lift from the pump to the storage tank is 20 x 2.31 = 46.2 ft.

The minimum tank capacity was computed as 7700 gal.

In anticipation of future expansion and peak demands, a required tank capacity of 8500 gal.

$$\frac{8500}{7.48} = 1136 \text{ cu. ft.}$$

Assuming a tank 10' in height.

$$A = 113.6 \text{ sq. ft.}$$

$$\text{Dia.} = \sqrt{\frac{113.6 \times 4}{\pi}} = 12.1 \text{ ft. Say 12 ft.}$$

The flow of 100 gallons will cause a decrease of $\frac{100}{7.48} = 13.35$ cu.'

$$\frac{13.35}{A} = \frac{13.35}{113} = .118 \text{ ft.}$$

$$1 \text{ ft.} = 847.5 \text{ gal.}$$

The discharge of each 100 gallons from this tank will lower the water level only .12 ft. This of course is true when the sides of the tank are vertical or if all horizontal sections are identical. This feature is favorable, that is, a constant static head under discharge provides uniform flow.

Rather than use pressure switches to govern the action of the pump - a float switch arrangement will be installed. This arrangement will operate so that pumping will begin when the tank water level is lower than 5.00 feet and shut off when the water level reaches 9.75 feet.

$$9.75 - 5.00 = 4.75 \text{ foot range}$$

$$4.75 \times 847.5 = 4030 \text{ gal.}$$

$$\frac{4030}{845} = 4.75 \text{ hour pumping period.}$$

The outlet of the tank will be installed at least ½ ft. from the bottom of the tank in order to prevent flow of settled material in the line.

The advantage of static head gained in elevating this storage tank is deemed insufficient as compared to the cost of constructing a suitable support structure. Therefore the tank will simply rest upon a spread footing. Therefore the gross static head available at the tank site is 9.75 feet.

The well will be located adjacent to the tank and the pump will be mounted in the building, in the compartment on the first floor under the stairway landing. The distance from the pump to the well and tank is 20 feet.

Ground elevation at tank - 130 ft.

Length of pipe line to the point of irrigation distribution = 163 feet.

Elevation - 108 ft.

Difference of elevation - 22 ft.

Total gross static head - 9.75 + 22 = 31.75 ft.

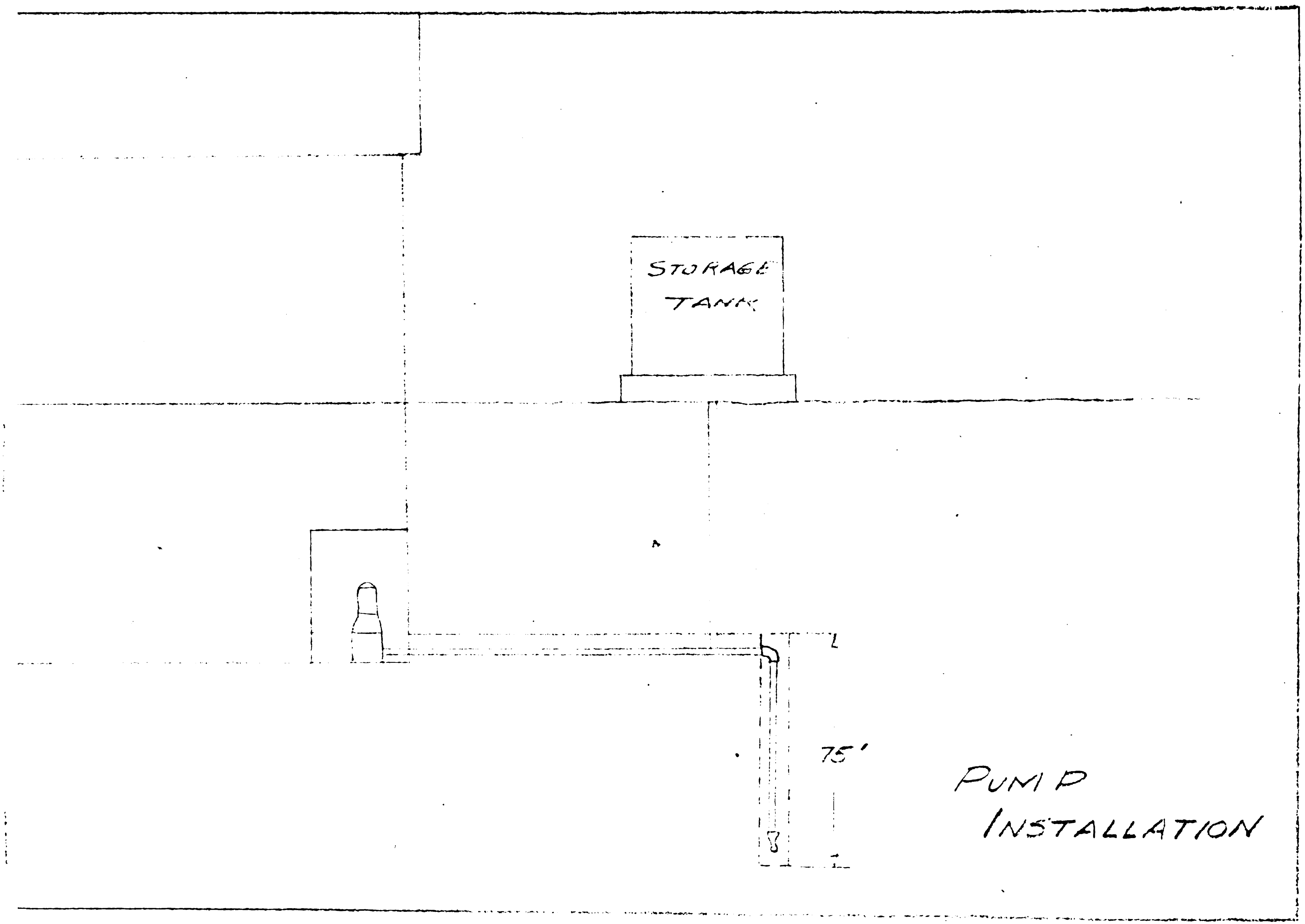
<u>Ground slope (along pipe line)</u>

L_1 - Water tank + 43' - (130 - 116') - 14 ft. Diff. Elev.

L_2 - Next 51' - (116 - 110) - 6 ft. Diff. Elev.

L_3 - Next 69' - (110 - 108) - 2 ft. Diff. Elev.

The pipe line will be laid to a minimum depth of 2 feet. No account of frost protection will be taken as the water facilities will not be used in freezing weather. Provisions


STORAGE
TANK
75'
PUMP
INSTALLATION

will be made to drain the line.

$$\text{Slope } L_1 = \frac{14}{43} = 32.6\%$$

$$L_2 = \frac{6}{51} = 11.75\%$$

$$L_3 = \frac{2}{69} = 2.9\ \%$$

- Pipe Size Selection and Friction Losses -

By the use of a diagram based on Hazen - Williams Formula with C = 100 for C.I. pipe after 20 years service.

$$V = Cr^{0.63}S^{0.54}{0.001}^{-0.04}$$

Selecting a 4" diameter pipe, and a desired Q = 100 gal./min.

Head loss for L_1 = 1.25 x .43 = 0.54'

Head loss for L_2 = 1.25 x .51 = 0.64'

Head loss for L_3 = 1.25 x .69 = 0.86'

Total Head Loss = 2.04'

V = 2.6 f.p.s.

A total head loss of 1 velocity head will be assumed sufficient to cover bend and nozzle losses:

$$h_L = \frac{2.6^2}{2 \times 32.2} = .105 \text{ ft.}$$

Total Head loss to this point:

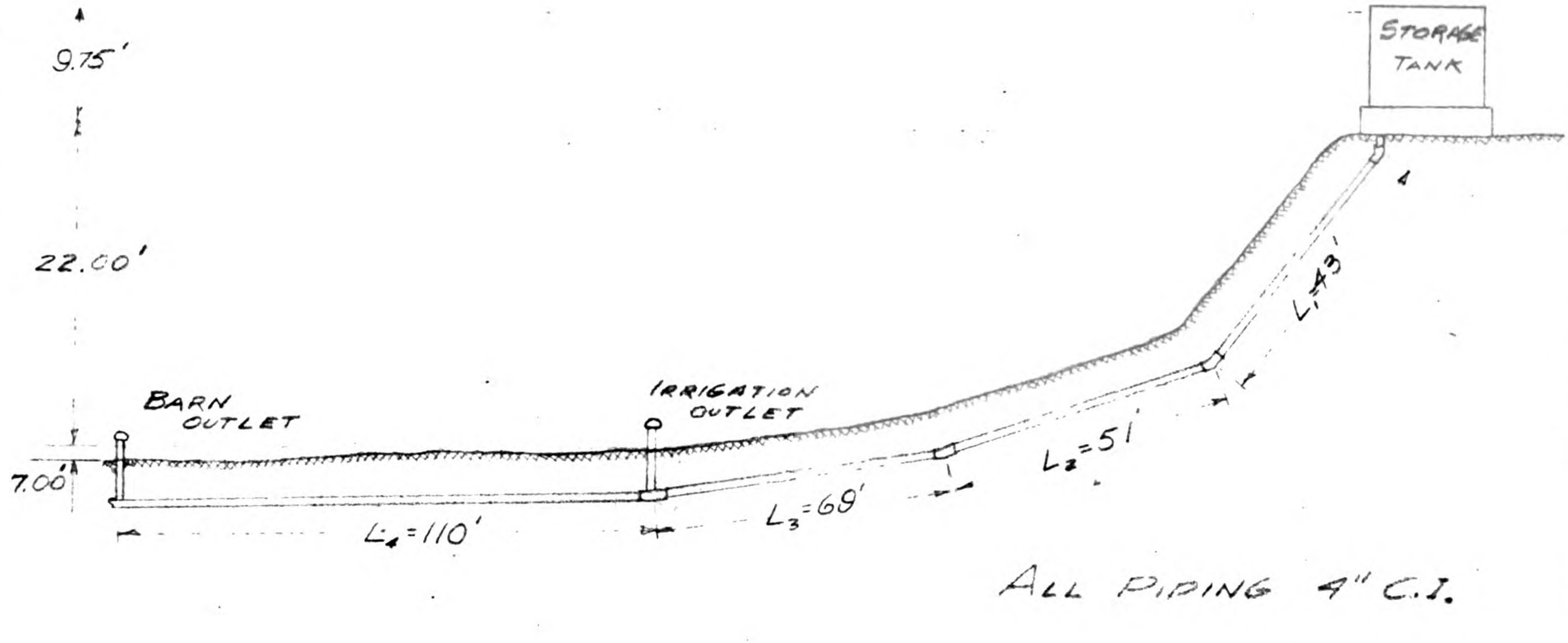
2.04 + 0.105 = 2.145 ft.

The next point of distribution and the end of the line for the present period of design is centrally located in the farm yard 110 ft. from the first point.

Elevation - 101'

Difference in elev. - (108-101) = 7'

Head loss in L_4 = 1.25 x 1.1 = 1.375'



STORAGE TANK
9.75'
22.00'
7.00'
BARN OUTLET
IRRIGATION OUTLET
L1=43'
L2=51'
L3=69'
L4=110'
ALL PIPING 4" C.I.

WATER LINE

Head loss for entire length:

$$2.145 + 1.375 = 3.320'$$

Net Available head = (130 - 101) + 9.75 - 3.32

= 35.43'

Pressure = 35.43' x .433 = 15.3 p.s.i.

This pressure is available for fire protection for the barns and residence.

FARM ROAD DESIGN

Road width - 12'

The material for this road will be taken from the gravel out croppings on the farm. Maximum aggregate size will be 1½".

Those sections of the road with the crown on the centerline will have:

Thickness at crown - 6"

Thickness at edge - 3"

This type of road is an adaptation of the feather edge type. 12 ft. was deemed an adequate width and since most of the travel will occur along the edges, these were made 3" thick instead of 0".

The surface of the ground along the road line will be cleared of trees, stumps, rocky and other objectional material before placing the pit run gravel.

After placing the gravel and compacting, a binder of native clay will be spread on the surface and allowed to weather and drop among the aggregate. The proportion of clay

ROAD DESIGN

A - PARKING & TURNAROUND - 48
30' WIDE

<u>LOCATION</u>

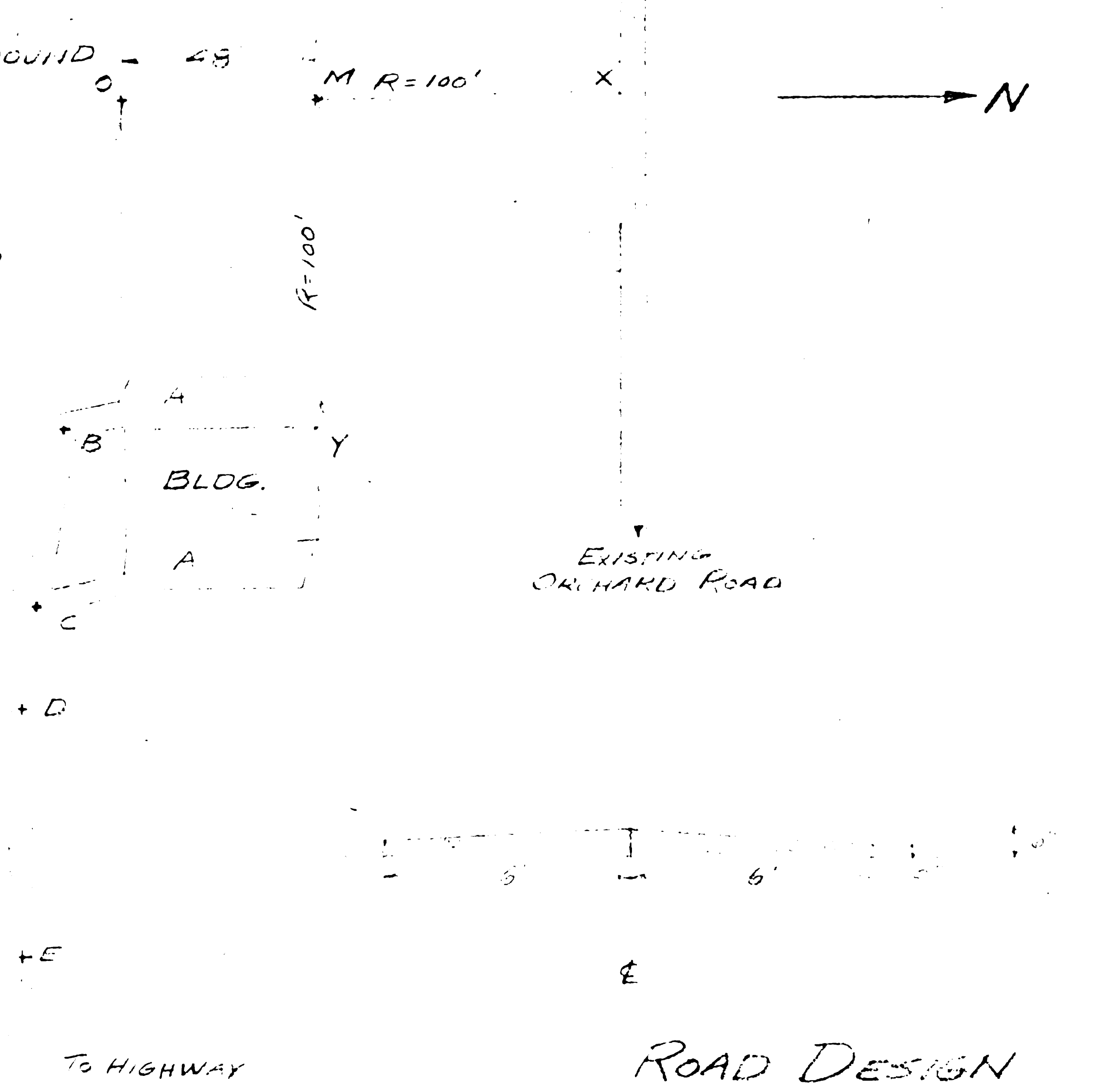
XY - CIRCULAR CURVE
M - CENTER
MX, MY - RADII - 100' 112'
∠XMY = 90°

∠XOB = 101°
OB = 110'

∠XOC = 100°05'
OC = 165'

∠XOD = 99°58'
OD = 200'

∠XOE = 97°20'
OE = 279'



to be thus applied will be 20% or 5 cu. ft. per 100 cu. ft. of gravel.

- FINIS -