A Study of the Sufficiency of Materials
In a One Story Residence
Built to Withstand Hurricane Winds

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

By

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Bachelor of Science

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THESIS

C. 1

Acknowledgments

The writer wishes to acknowledge his appreciation to members of the Civil Engineering Department, particularly Professor C. A. Miller, for their kind assistance in this study.

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Dedication

To my wife.

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Symbols and Notation

a : coefficient used in $A_s = \frac{M}{ad}$

A : cross sectional area

A.C.I. : American Concrete Institute

A_s : area of tensile reinforcement or of column bars

A'_s : ares of compressive reinforcement in flexual members

A_v : area of web reinforcement

b : width of rectangular beam

B.M. : bending moment

c : coefficient used in $A'_s = \frac{M-KF}{cd}$; also distance from neutral axis to extreme fiber of a section

C : resultant of compressive stresses

d : effective depth of flexual members

d' : distance from extreme fiber to compressive reinforcement

δ : declination (degrees)

f_c : compressive stress in extreme fiber

f_s : stress in tensile reinforcement

ft. : foot

f_v : stress in web reinforcement

f_w : allowable working stress

F : $\frac{bd}{12,000}$; used in determination of resisting moment of concrete sections

H.B.C. : Building Code, City of Houston, Texas

I : moment of inertia

in. : inch

j : ratio of distance (jd) between resultants of compressive and tensile stresses to effective depth

k : ratio of distance (kt) between extreme fiber and neutral axis to effective depth

K : $\frac{1}{2}f_c jk$

l : length of span

lb. : pound

M : external moment (ft. kips)

m : $np+(n-1)p'$; used in determination of k

n : ratio of modulus of elasticity of steel (E_s) to that of concrete (E_c)

N.C.M.A. : National Concrete Masonry Association

N : number of stirrups

ϕ : latitude (degrees)

p' : ratio of compressive reinforcement in beams

psf : pounds per square foot

q : $np+(n-1)p'\frac{d'}{d}$; used in determination of k

Σ_o : sum of perimeters of bars

s : spacing of stirrups (in.)

S : base length of shear diagram (ft.)

T : resultant of tensile stresses

u : bond stress

v : shearing stress

v' : shearing stress taken by web reinforcement

V : total shear

V' : the total external vertical shear in excess of that allocated to the unreinforced web w : unit load per lineal ft.

z : ratio of distance (zkd) between extreme fiber and resultant of compressive stresses to distance kd

z : meridian zenith distance--angular distance from a point directly overhead (zenith) to the sun and measured along a meridian.

Introduction

In these days of the ever present housing shortage everyone has a dream house. One day that dream will come true.

But long before this, the wise builder did a lot of thinking and planning. It is with this idea in mind that the writer approaches the subject.

The first consideration is a floor plan which will give the physical measurements necessary to carry on an investigation. Features that were to be included were an entry, breakfast alcove, laundry room, sewing room, heater room, cold storage room, dressing room off master bedroom, and all rooms opening on a gallery. Since no such plan was available it was necessary to act as my own architect. The reader is reminded at this point that it is bad business to try to make your own working drawings in case you are planning to build your own home. All construction should be supervised by a competent builder. For instance, to leave out one small detail such as roof flashings around a chimney would be a very costly detail indeed.

To satisfy the requirement that all rooms open on a gallery suggested a U-shaped floor plan with the gallery opening on an interior courtyard.

Start with the kitchen since most of the fixtures are standardized as to size and deciding on an arrangement of the fixtures will determine the size of the room. It was decided to include a combination sink and dishwasher, gas stove, gas refrigerator separated by a base cabinet 18 inches wide, desk to hold a telephone and file recipes, corner base cabinets, and other base cabinets such as are necessary to fill out the circumference of the room. Base cabinets are 19 inches deep and 30 inches high. Wall cabinets extend along the outside wall over the sink-dishwasher combination and along one interior wall over the stove and refrigerator. The north wall is open over the base cabinets. The base cabinets act as a serving counter for the breakfast alcove in the next room. A door on the east wall opens off the gallery.

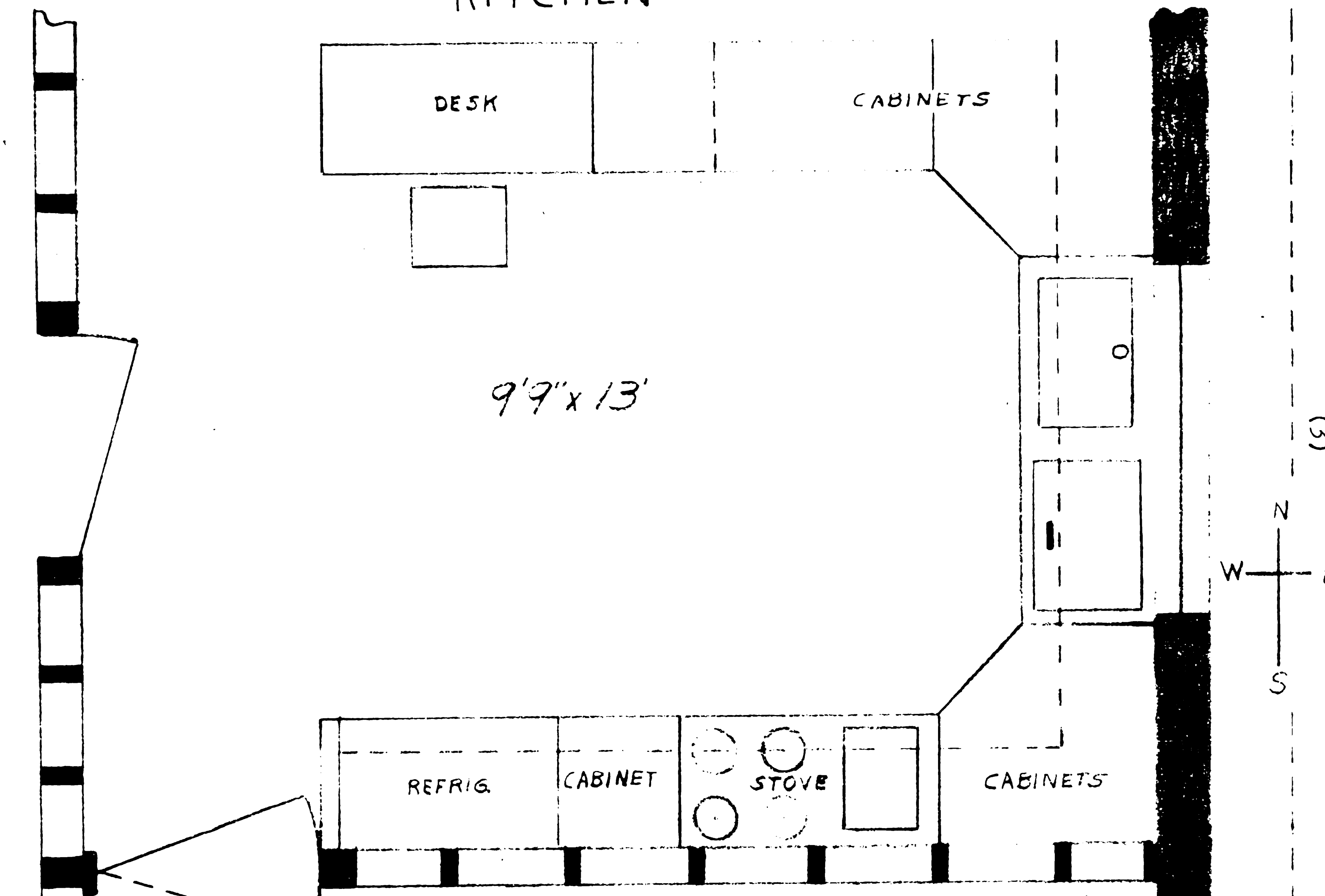
The breakfast alcove has built-in seats along the outside wall and along the back of the kitchen base cabinets. There are three chairs on the north side of the table.

The laundry and sewing room has a Bendix washer and dryer, clothes hampers for the three classifications of soiled clothing, a desk to hold the sewing machine and store buttons, paterns, etc., wall cabinet storage space for other sewing aids and necessities, laundry sink, rotary ironer, ironing board that folds up to fit into a wall cabinet. There is also storage space in wall cabinets for pottery and this room can also act as flower arranging center. A door opens onto the gallery.

The utility room contains the hot water boiler and automatic hot water heater. Both use natural gas. The valves for the adjustment and control of the radiant heating panels are also in this room. The east wall is taken up by a refrigerated cold storage room. It is not intended to heat the utility room. Wall cabinets could be built to provide additional

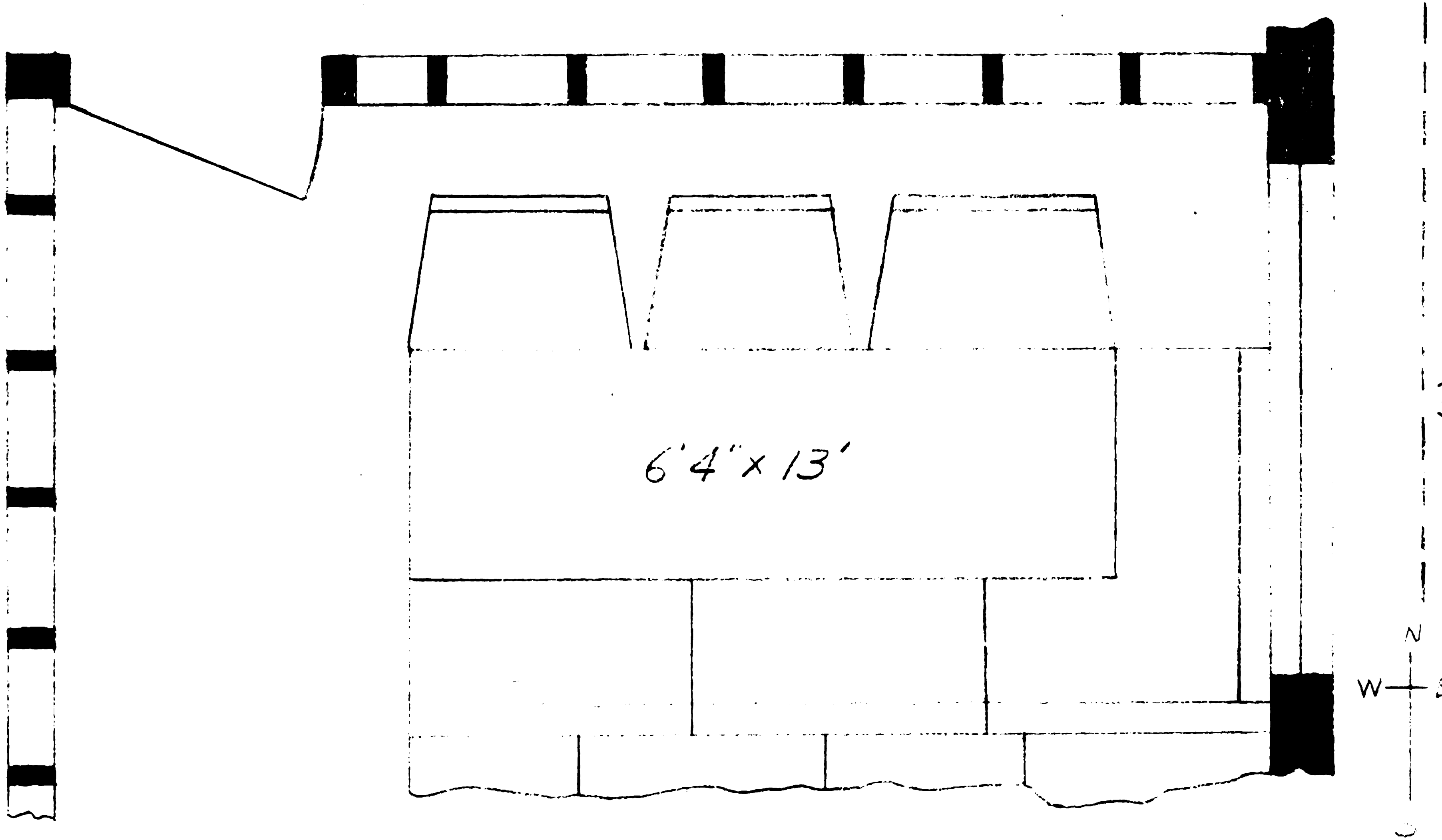
KITCHEN

KITCHEN



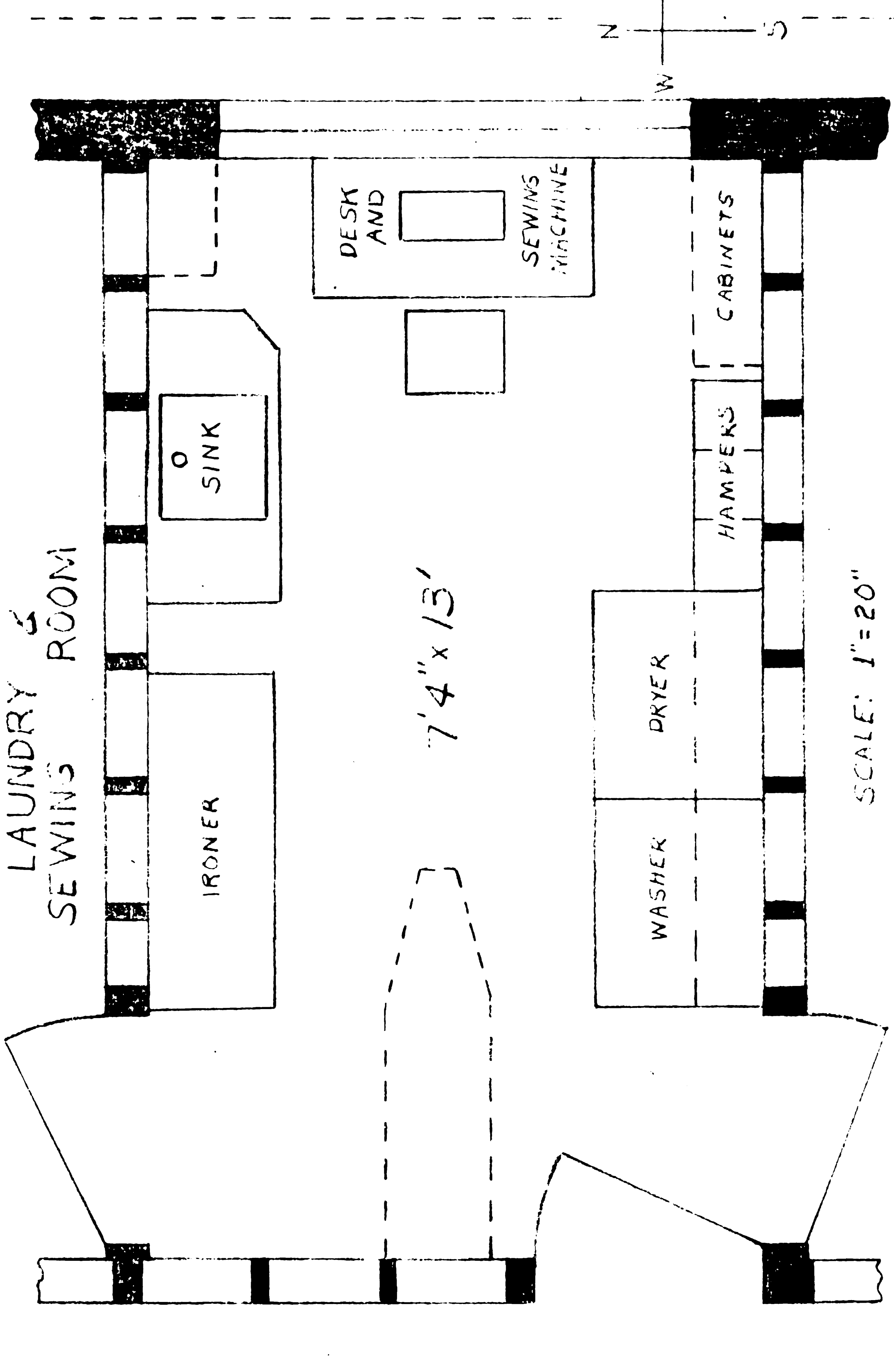
SCALE. 1" = 20"
Fig. 1

BREAKFAST ALCOVE



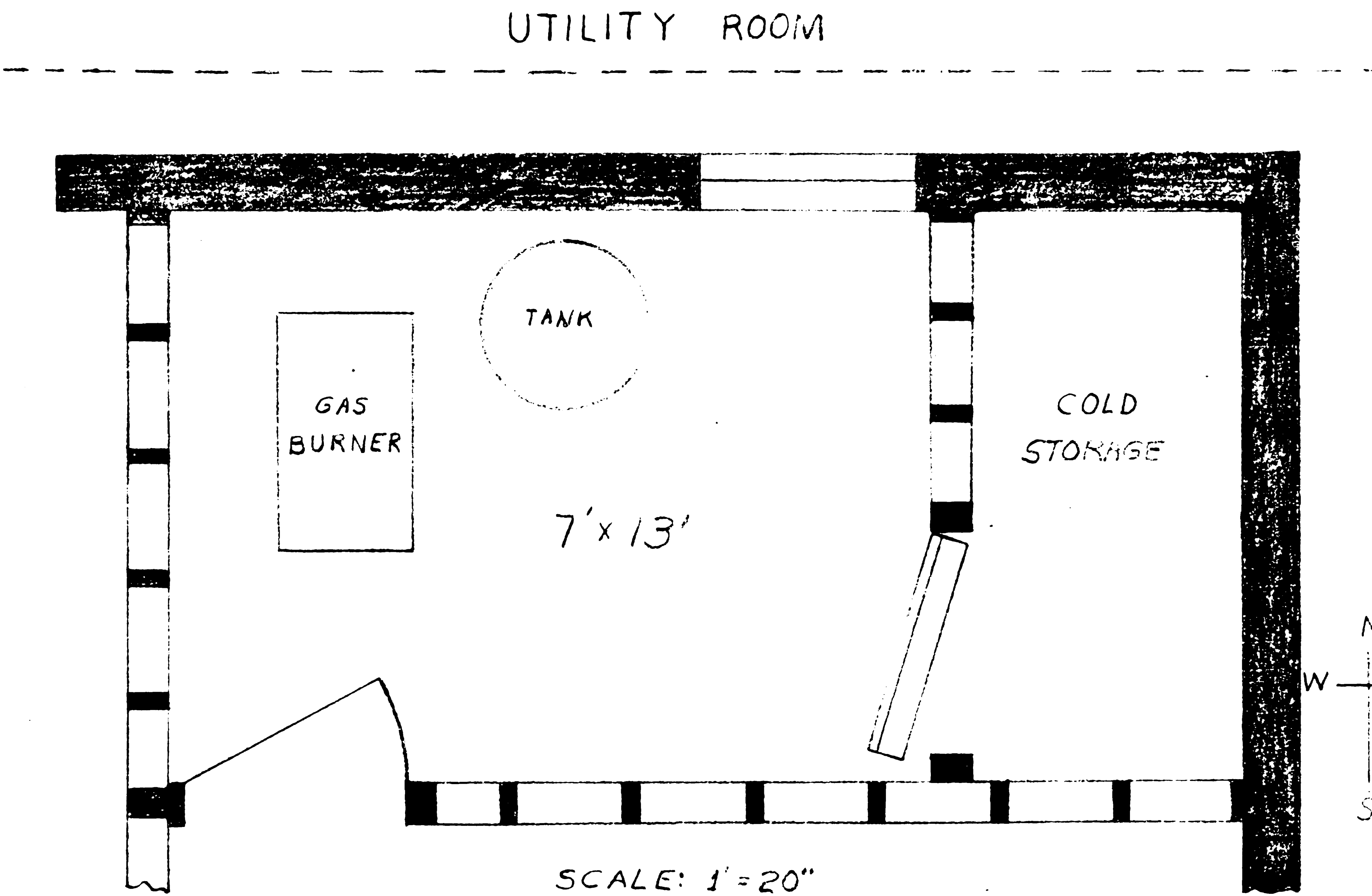
SCALE: 1" = 20"

FIG 2



SCALE: 1"=20"

FIG. 3



SCALE: 1' = 20"

FIG. 4

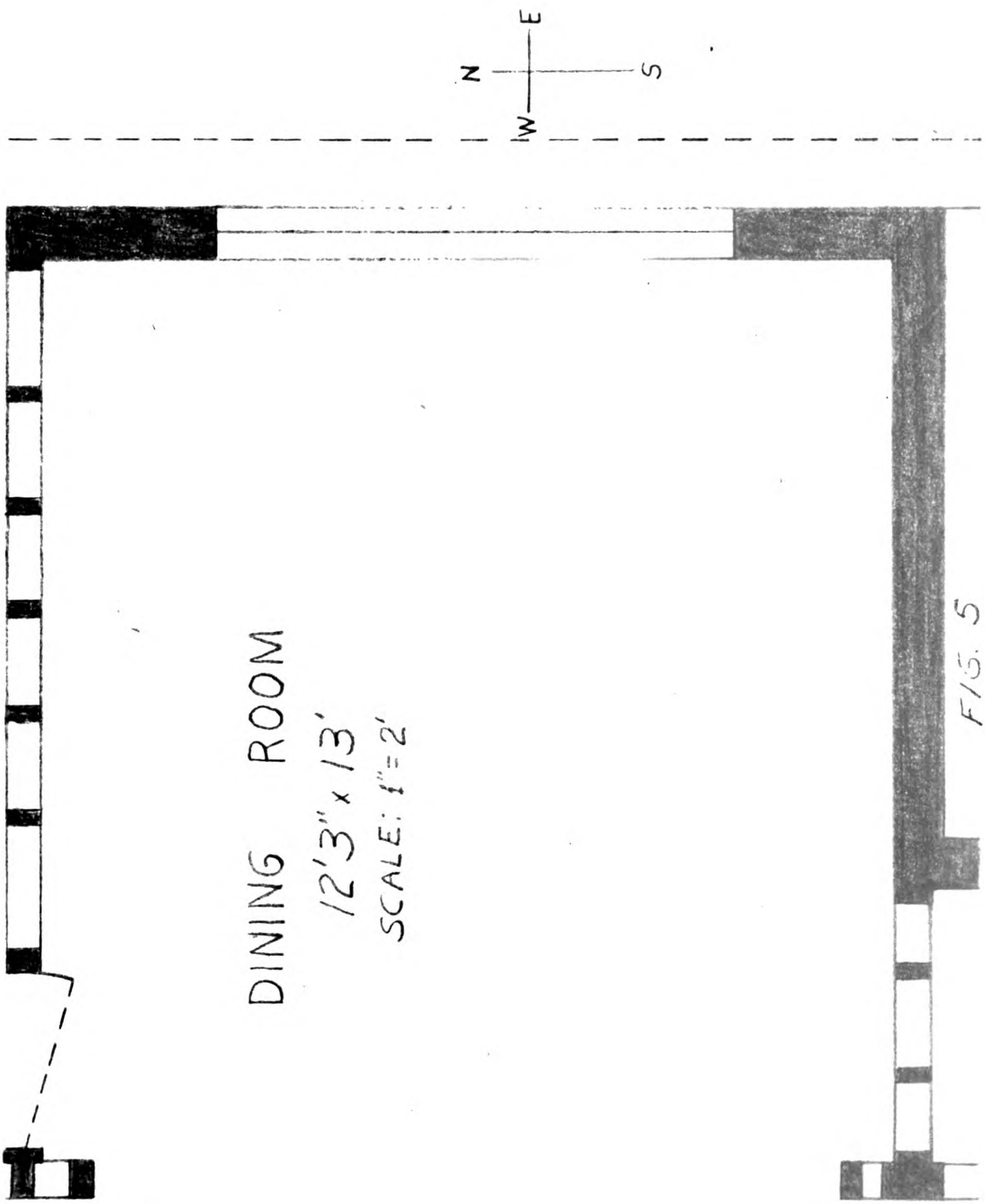


FIG. 5

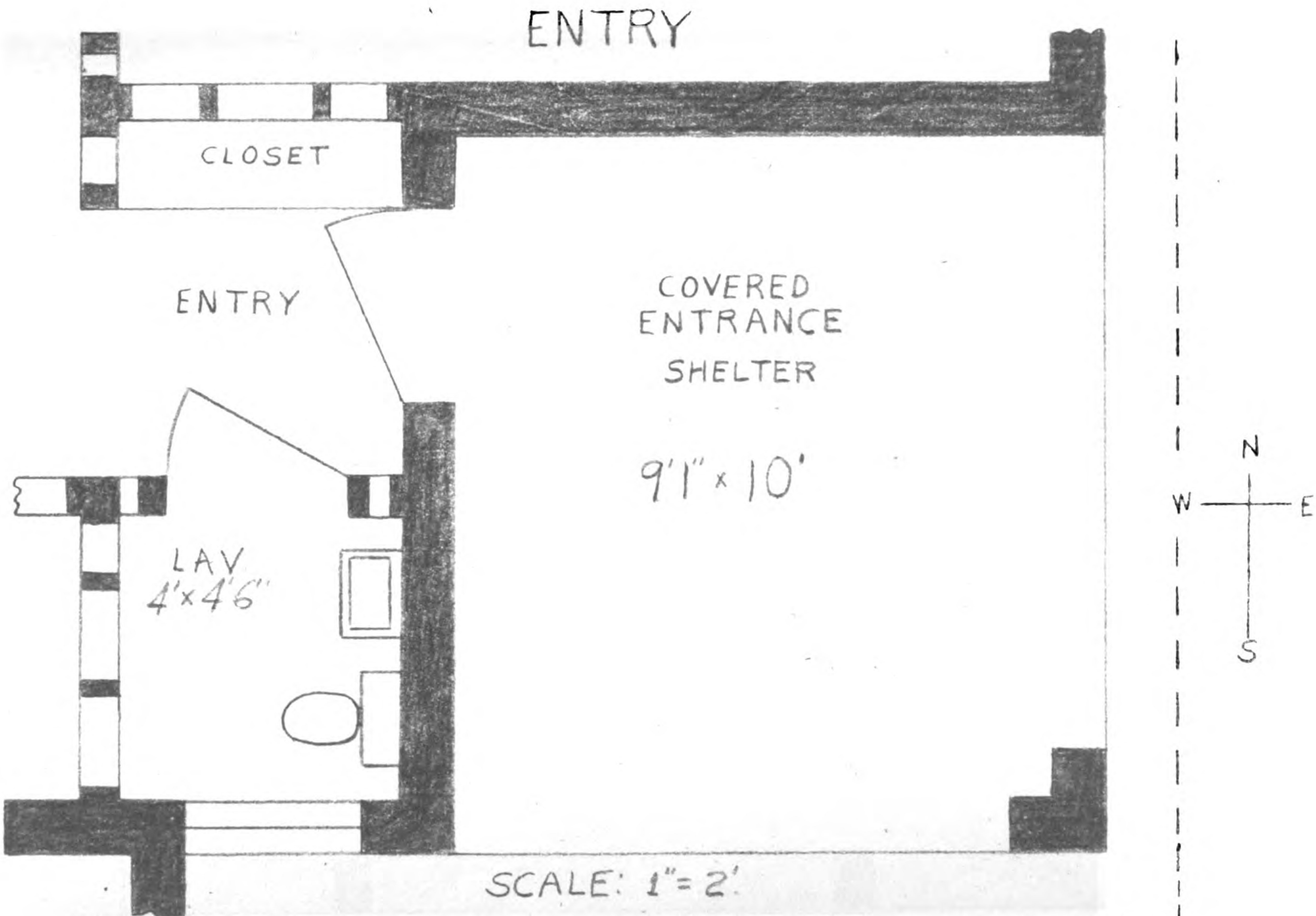
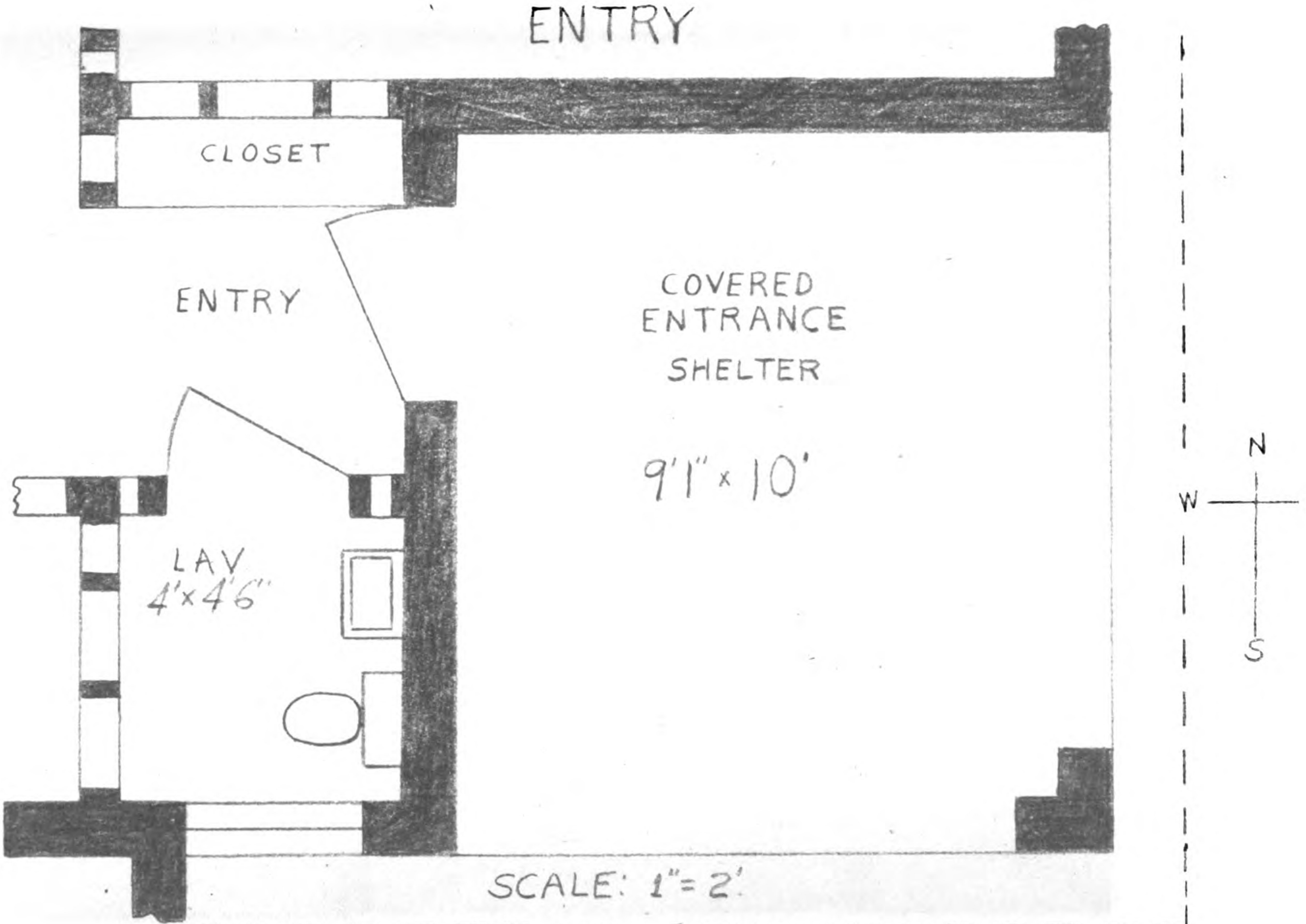

ENTRY
CLOSET
ENTRY
COVERED
ENTRANCE
SHELTER
9'1" x 10'
LAV
4'x4'6"
N
W
E
S
SCALE: 1" = 2'

FIG. 6


ENTRY
CLOSET
ENTRY
COVERED
ENTRANCE
SHELTER
9'1" x 10'
LAV
4'x4'6"
N
W
E
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SCALE: 1"= 2'

FIG. 6

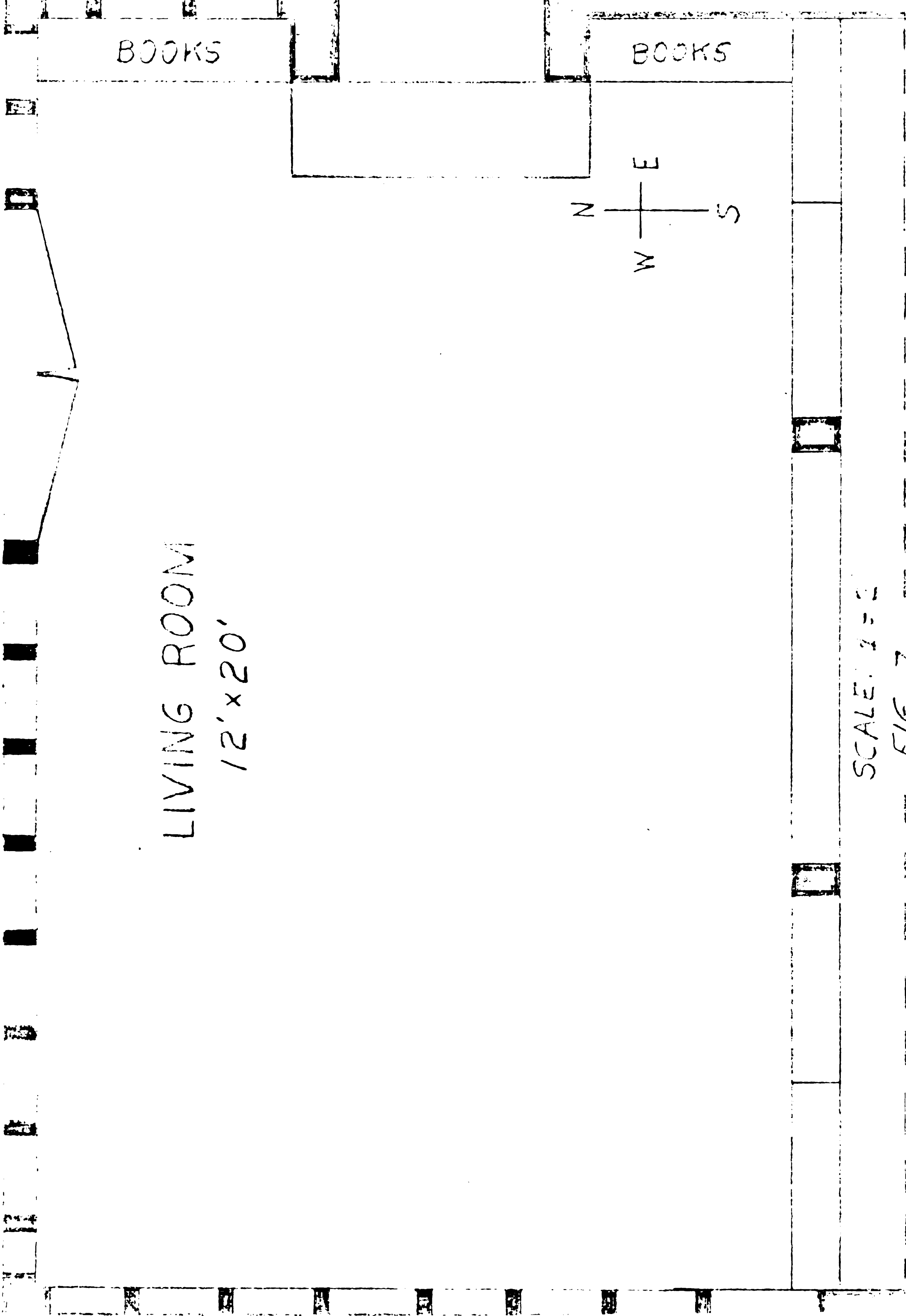


FIG. 7

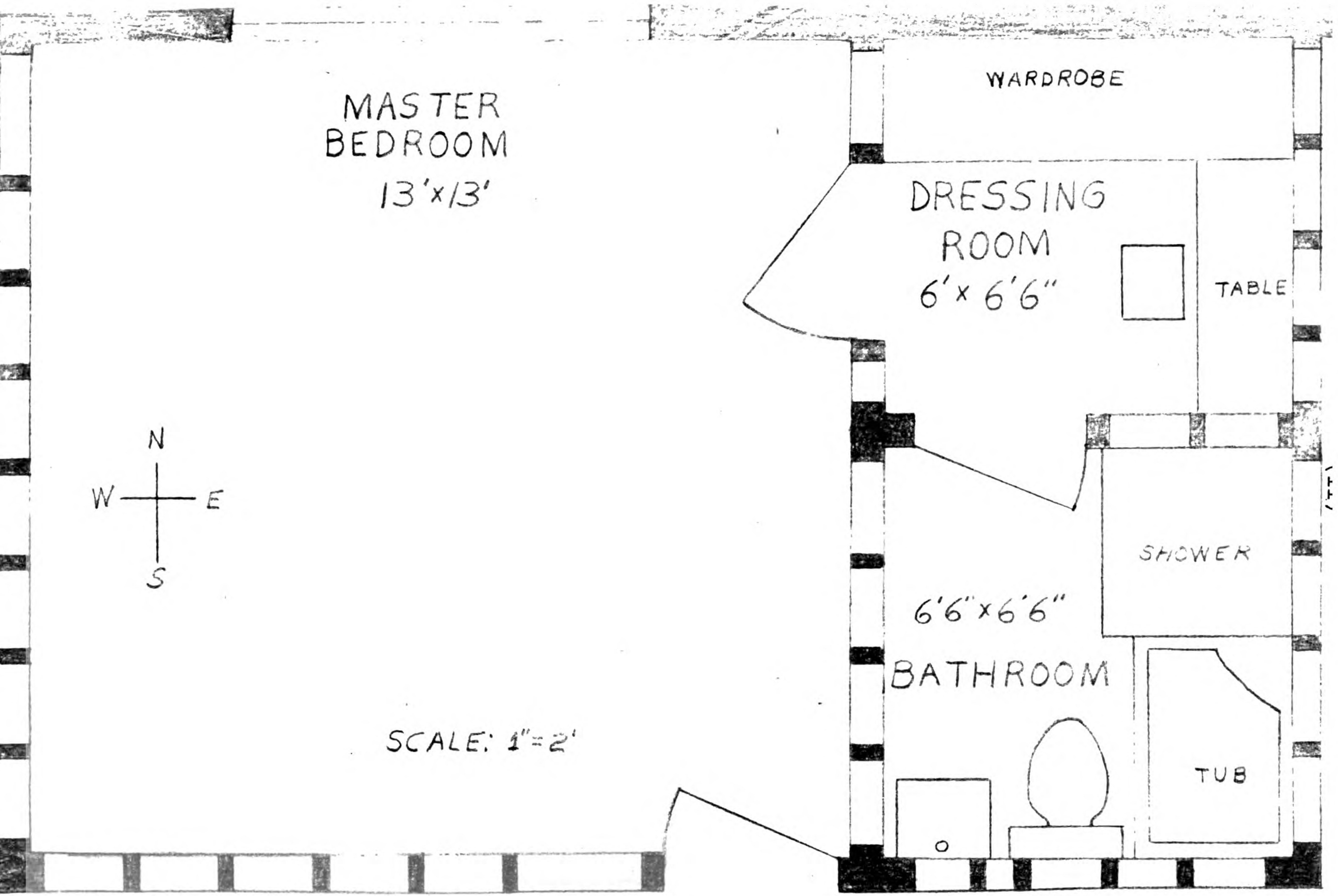
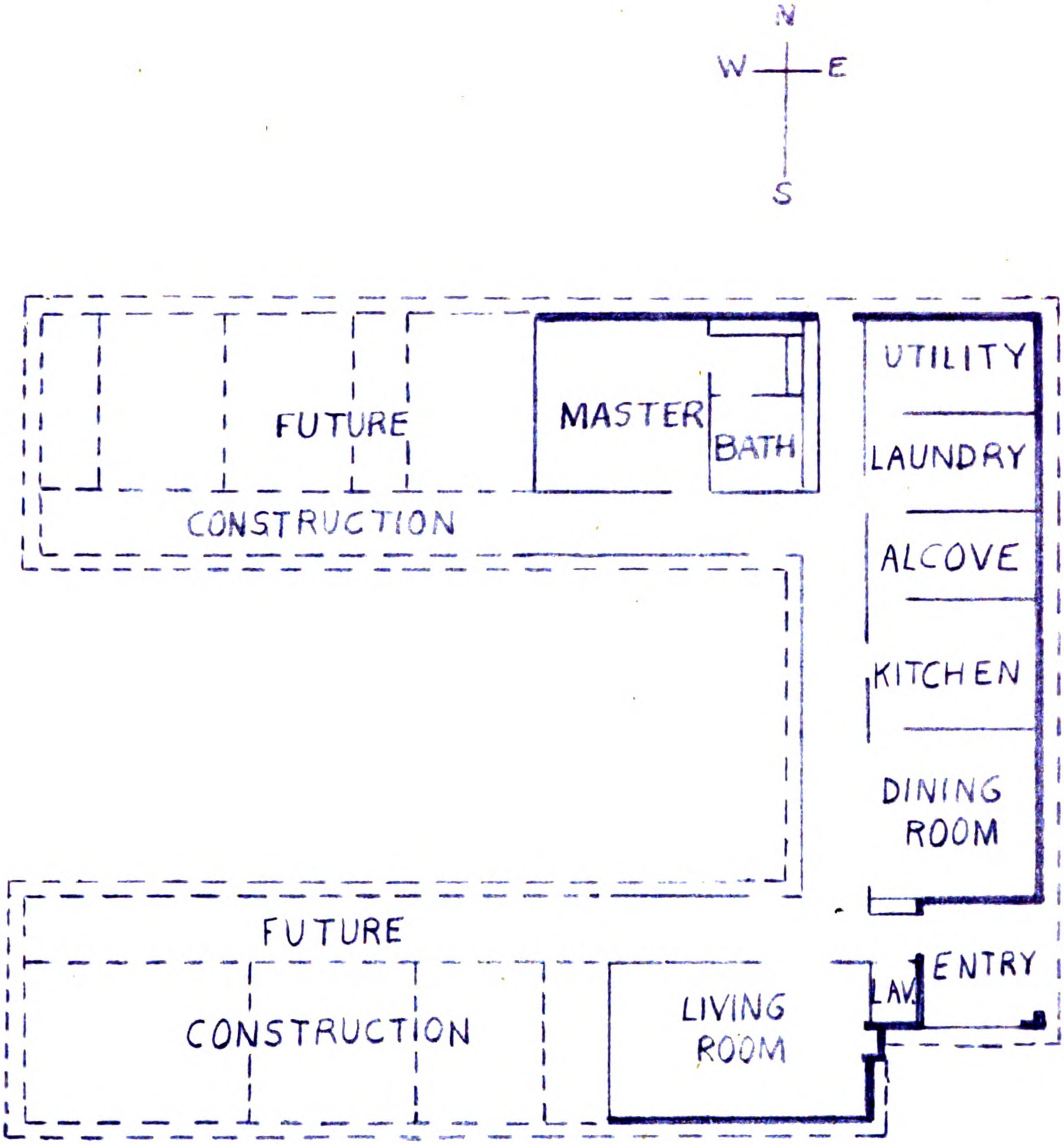

MASTER
BEDROOM
13'x13'
N
W
E
S
SCALE: 1"=2'
WARDROBE
DRESSING
ROOM
6'x 6'6"
TABLE
SHOWER
6'6"x6'6"
BATHROOM
TUB

FIG. 8



FLOOR PLAN

SCALE: 1 CM. = 5'

FIG. 9

until it reaches its lower culmination on December 21. We know that the slant rays of a December sun have very little warmth and so we want all of the sun to enter the windows. These sun angles are best shown by the diagram of Fig. 14.

The clear distance from floor to ceiling is 7 ft. 6 in. Figure the overhang by using the tan of the angle of the suns rays. This angle varies with different localities. Since Houston, Texas is in the hurricane area, calculations will be for that locality. Houston is located at 29 north latitude. From Astronomy is given the formula

$z_m = \phi - \delta$ for a heavenly body south of the zenith

$\delta = 23°26.5'$ (1946 Ephemeris)

Now

$z_m = \phi - \delta = 29° - 23°26.5' = 5°33.5'$

Overhang $= 7.5 \tan 5°33.5' = 7.5 \times 0.09731 = 0.73$ ft.

Use a one foot overhang as was used on all other sides of the house.

Floor Thickness

Next check to see if the floor is thick enough. Standard thickness for floors with pipes embeded for radiant heating is six inches. Assuming one inch of cover coat over the tension reinforcement would give an effective depth of 5 inches. Table 2 of the American Concrete Institute Reinforced Concrete Design Handbook gives a resisting moment of a rectangular section one foot wide with an effective depth of 5 inches as 6.2 ft. kips.

storage space.

The master bedroom has a dressing room with built-in wardrobe closet. Future proposed additions to this wing are three smaller bedrooms for the children. A bathroom would serve two bedrooms.

Access to the dining room is by means of a swing door off the kitchen. The west side of the room was left entirely open.

The living room has a mirror that runs the full width of the room above the fireplace mantel. Below the mirror and flanking the fireplace are two bookcases.

Future proposed additions to this wing are a study room, a large recreation room, and a guest room with bath.

Both the front and the side entrances have storage space for clothing as you come from the outside. The two halls running east and west will have doors at their far ends when the proposed additions are completed.

It will only be necessary to investigate the original construction since conditions are representative in this part of the house.

Roof Overhang

The first step is to calculate the roof overhang on the south side of the house. This overhang acts as a sun shade for the windows on this side of the house. This sun shade is needed when the sun reaches its upper culmination or its high point in the sky. Upper culmination occurs on June 21. Then the sun gradually travels southward a small amount each day

Sec. 2612, H.B.C. gives other values as

f'_c = 3,000 psi.

f_s = 20,000 psi. for billet-steel bars

f_c = 0.40 f'_c = 1,200 psi.

n = 10

The bending moment is calculated to see that it does not exceed the resisting moment.

Span= 18 ft. 11 in. use 19 ft.

Live load= 40 psf. Sec. 2304, H.B.C.

Dead load= $\frac{6}{12}$ x 150= 75 psf.

B.M.= $\frac{1}{8}wl^2$ where w= 115 lb. per lineal ft.

B.M.= $\frac{1}{8}wl^2 = \frac{1}{8}(115)(19)^2 = 5{,}200$ ft. lb.

5,200 < 6,200, therefore the floor meets the code specifications. Figure the tension steel required for the floor slab.

$M = Tjd = A_s f_s jd$

From Table 1, A.C.I. Design Handbook

j= 0.875

$A = \frac{M}{f_s jd} = \frac{6{,}200 \times 12}{20{,}000 \times 0.875 \times 5} = 0.851$ sq. in.

Table 3, A.C.I. Design Handbook gives 0.88 sq. in. for $\frac{3}{4}$ in. round bars spaced 6 in. center to center. Use ½ in. round bars spaced 1 ft. center to center as spacers.

Wind Velocity and Pressure

Before designing for stresses set up in the building an equivalent wind loading must be decided upon.

Hurricane winds frequently attain velocities of 100 miles per hour. Records indicate that these strongest winds often reach and, for 5 minutes or more, maintain velocities of 100 miles per hour, and occasionally even 125 to 150 miles per hour with gusts of still greater velocity. No state along the South Atlantic and Gulf Coasts has an average of one tropical storm of full hurricane force in a year.

According to engineers, Coral Gables, Florida was centrally located in the path of the hurricane of September 18, 1926. This storm, in the opinion of R. W. Gray, weather observer at Miami, exceeded all prior hurricanes in the United States from the standpoint of severity. The wind velocity was 125 miles per hour. A deluge of rain amounting to between 8 and 15 inches in 16 hours fell during the storm.

At the time of the hurricane Coral Gables contained some 2,500 residences, apartments and other buildings, all of which had concrete masonry walls. These buildings erected in accordance with provisions of the local building code, came thru the storm without a single case of destruction and with but slight damage.

George E. Merrick, president, Coral Gables Corporation, in commenting on the storm, wrote: "Coral Gables, wisely restricted to concrete construction, withstood the force of the gale.......The total damage to the entire city of Coral Gables will not exceed $1,500,000 or about 1 per cent of the amount of money that has been spent in Coral Gables in construction and development."

Mr. Merrick was speaking of concrete masonry when he said concrete construction because at the time concrete masonry had been used in 93% of the buildings in Coral Gables.

The building code that was adopted was in practical conformity with the recommendations of the Building Code Committee, U.S. Department of Commerce. The quality of mortar was definitely regulated, the specification for portland cement mortar requiring 1 part portland cement and not to exceed 3 parts sand, by volume; for portland cement-lime mortar, 1 part portland cement, 1 part slaked lime, and not more than 6 parts sand, by volume. Under the provisions of the code, anchoring of roofs and tying-in of floors are required.

Pressure exerted on a building by wind is governed by many factors, such as wind velocity, atmospheric pressure and temperature, height above ground, shape of building, and its orrientation towards the air stream.

In these investigations use velocity pressure P, in pounds per square foot equals 0.0033 V , where V= velocity of wind in miles per hour. Based on the foregoing formula with V= 125, the approximate velocity pressure is 50 pounds per square foot.

H. L. Whittemore, according to unpublished data on strength of low-cost houses of the National Bureau of Standards, worked out velocity pressures that could be anticipated at 30 feet above ground in any locality in the United States. Pressures at 20 feet would be approximately 10 per cent less. His pressure curves are based on the maximum average wind velocity for a 5-minute interval as reported by the U. S. Weather Bureau plus 50 per cent gust. The formula

(P-0.0033V) used above was the Weather Bureau formula.

On the strength of Whittemore's data, it seems reasonable to assume for design purposes a velocity pressure of 45 lb. per sq. ft. after a 10 per cent reduction.

Distribution of Wind Forces

Table 1 shows the average distribution in terms of velocity pressure P on walls, roof, and eaves.

Table 1

Average Distribution in Terms of

Velocity Pressure P, on Walls, Roofs, and Eaves

Building Element	Orientation	Velocity Pressure
Walls	Windward	0.80 P pressure
Walls	Leeward	0.50 P suction
Walls	Parallel	0.58 P suction
Roof	Windward (0 to 20 deg. slope)	0.77 P suction
Roof	Leeward, flat	0.77 P suction
Roof	Parallel, any slope	0.77 P suction
Eaves	Maximum pressure up	1.57 P
Eaves	Maximum pressure down	1.16 P

The wind forces shown above are based on a windtight building. Loads from internal wind forces must be added for buildings with openings or potential openings such as windows or doors. For buildings having 30 per cent or more wall openings in windward side add 0.77 P pressure outward on roof and walls, and for openings in leeward or parallel sides add 0.58 P suction in. For openings between 0 and 30 per cent of wall area, assume proportional additional loads.

The percentage of openings on each side are as follows:

East side -

$$\frac{18.8 + 48.2 + 10.6 + 20.9 + 20.9}{62 \times 7.5} = \frac{119.4}{465} = 25.7\%$$

South side

$$\frac{2.6 + 116.6}{33 \times 7.5} = \frac{119.2}{248} = 48\%$$

West side

$$\frac{42.0}{7.5 \times 62} = \frac{42.0}{465} = 9\%$$

North side

$$\frac{13.9 + 18.8 + 31.4}{7.5 \times 38.5} = \frac{64.1}{289} = 22.1\%$$

Wind from east

For openings in windward side add

$\frac{25.7}{30} \times 0.77 \times 45 = 29.7$ lb. pressure out on roof & walls

For openings in leeward side and parallel walls add

$\frac{119 + 42 + 64}{248 + 465 + 289} \times \frac{1}{30} \times 0.58 \times 45 = 19.6$ lb. suction in on roof and walls

This gives a net result due to wall openings of 10.1 lb. pressure out on walls and roof.

If the roof slab were poured using 2 x 6 in. edge forms its thickness would be $5\frac{5}{8}$ in. and would weigh

$\frac{5.625}{12} \times 1 \times 1 \times 150 = 70.3$ lb. per sq. ft.

Pressure on east wall

$0.8 \times 45 - 10.1 = 25.9$ lb. pressure in

Pressure on west wall

$0.5 \times 45 \times 1 - 10.1 = 32.6$ lb. suction out

Pressure on north and south walls

$0.58 \times 45 + 10.1 = 32.6$ lb. outward suction

Pressure on roof

$70.3 - 0.77 \times 45 - 10.1 = 25.5$ lb. pressure down

Max. pressure up on eaves $= 1.57 \times 45 - 70.3 = 0.3$ lb. pressure up

Max. pressure down on eaves $= 1.16 \times 45 + 70.3 = 122.5$ lb. pressure down

Wind from west

For openings in windward side add

$\frac{9}{30} \times 0.77 \times 45 = 10.4$ lb. pressure out on roof and walls

For openings in leeward side and parallel walls add

$\frac{119 + 119 + 64}{465 + 248 + 289} \times \frac{1}{30} \times 0.58 \times 45 = 26.1$ lb. suction in on walls and roof

This gives a net result due to wall openings of 15.7 lb pressure in on walls and roof

Pressure on west wall

$0.8 \times 45 + 26.1 = 62.1$ lb. pressure in

Pressure on east wall

$26.1 - 0.5 \times 45 = 3.6$ lb. pressure in

Pressure on north and south walls

$26.1 - 0.58 \times 45 = 0$ lb. pressure

Pressure on roof

$70.3 + 15.7 - 0.77 \times 45 = 51.3$ lb pressure down

Wind from north

For openings in windward side add

$22.1 \times 0.77 \times 45 = 25.5$ lb. pressure out on roof & walls

For openings in leeward side and parallel walls add

$\frac{119 + 119 + 42}{248 + 465 + 465} \times \frac{1}{30} \times 0.58 \times 45 = 10.2$ lb. suction in on walls and roof

This gives a net result due to wall openings of 15.3 lb. pressure out on roof and walls.

Pressure on north wall

$0.8 \times 45 - 15.3 = 20.7$ lb. pressure in

Pressure on south wall

$0.5 \times 45 + 15.3 = 37.8$ lb. suction out

Pressure on east and west walls

$0.58 \times 45 + 15.3 = 41.4$ lb. suction out

Pressure on roof

$70.3 - 15.3 - 0.77 \times 45 = 20.4$ lb. pressure down

Wind from south

For openings in windward side add

$0.77 \times 45 = 34.7$ lb. pressure out on walls and roof

For openings in leeward side and parallel walls add

$\frac{64 + 119 + 42}{289 + 465 + 465} \times \frac{1}{30} \times 0.58 \times 45 = 16$ lb. suction in on roof and walls

This gives a net result due to wall openings of 18.7 lb. pressure out on walls and roof

Pressure on south wall

$0.8 \times 45 - 18.7 = 17.3$ lb. pressure in

Pressure on north wall -

0.5 x 45 + 18.7 = 41.2 lb. suction out

Pressure on east and west walls

0.58 x 45 + 18.7 = 44.8 lb. suction out

Pressure on roof

70.3 - 18.7 - 0.77 x 45 = 16.9 lb. pressure down

Wind loadings are sumarized in Table 2.

From Table 2 the greatest velocity pressure exerted was 62.1 psf. pressure on the west wall with wind from the west. Modulus of rupture as given by the Portland Cement Association is 130 psi. for course construction of dry tamped concrete as in a double hollow wall. The Underwriters' Laboratories' Standard for Concrete Masonry Units defines hollow units as having average core areas in excess of 25 per cent of gross volume. The unit used in these walls was a 3-oval core 8 x 8 x 16 in. cinder block. This unit falls within the Underwriters' Standard for a hollow unit. The units alone will not rupture.

Transverse Strength of Walls

The University of Illinois conducted tests for determining the modulus of rupture of walls. Results in psi. for 3-oval core 8 x 8 x 16 in. cinder blocks were 34, 42, 18, 22, and 23.

The flexure test was made on large walls. These walls, which failed by cracking along a horizontal joint near mid-height, were later tested in compression , and developed as good compressive strength as the uncracked walls.

Table 2

Velocity Pressures on Building Elements According

To Orrientation With Respect to Wind

Wind from	Building Element	Velocity Pressure
East	E. wall	25.9 psf. pressure
	W. wall	32.6 psf. suction
	N. & S. walls	32.6 psf. suction
	Roof	25.5 psf. pressure
	Eaves	0.3 psf. pressure up
	"	122.5 psf. pressure down
West	W. wall	62.1 psf. pressure
	E. wall	3.6 psf. pressure
	N. & S. walls	0.0 psf.
	Roof	51.3 psf. pressure
	Eaves	0.3 psf. pressure up
	"	122.5 psf. pressure down
North	N. wall	20.7 psf. pressure
	S. wall	37.8 psf. suction
	E. & W. walls	41.4 psf. suction
	Roof	20.4 psf. pressure
	Eaves	0.3 psf. pressure up
	"	122.5 psf. pressure down
South	S. wall	17.3 psf. pressure
	N. wall	41.2 psf. suction
	E. & W. wall	44.8 psf. suction
	Roof	16.9 psf. pressure
	Eaves	0.3 psf. pressure up
	"	122.5 psf. pressure down

The flexure test was made by placing the wall against a vertical structural steel framework and applying a lateral concentrated load along the horizontal center line with the slab supported along the top and bottom edges. The span was 9 feet. The load was applied by means of a 10-ton screw jack, a spring dynamometer and a steel I-beam.

Values of modulus of rupture were computed on the basis of a beam of solid rectangular section, with a width equal to the length of the wall and a depth equal to the wall thickness. This assumption is used to facillitate computation and gives values that err on the side of conservatism. The failure of the walls was in all cases a failure in adhesion of mortar to unit. In all of the oval core units, molded on a metal pallet, the failure was between the smooth, molded lower surface of the block and the mortar. The walls with face-shell bedding gave about as good results, considering the mortar strengths, as do corresponding walls with full bedding. This is logical, since the area occupied by cross webs should have little effect on flexual resistance. The low values for modulus of rupture were for low strength mortars. Therefore we are safe in throwing out the low value which gives an average for the modulus of rupture as 30 pounds per square inch.

The Portland Cement Association, in their publication, Reinforced Concrete Houses, calculate the resistance of a dwelling wall to wind pressure by assuming the minimum horizontal section with openings deducted and the wind load carried on the vertical span.

From Table 2 the wind load on the west wall is 62.1 psf. The clear span between the roof and floor is 7.5 ft.

$$M = \frac{1}{8} wl^2 = \frac{62.1}{8} \times 7.5^2 = 436 \text{ ft. lb.}$$

For combined bending with axial load

$$f = \frac{P}{A} \pm \frac{Mc}{I} \quad \text{where } c = 4 \text{ in.}$$

$$P = 51.3 \times \frac{21.583}{2} \times \frac{16}{12} + 32.5 \times 6 = 934 \text{ lb.}$$

$$A = 0.60 \times 15.75 \times 8 = 75.7 \text{ sq in.}$$

$$I = \frac{1}{12} \times 15.75 \times 8^3 - 4 \times 0.0491 \times 3.21 \times 5^3 = 592.4 \text{ in.}$$

$$f = \frac{934}{75.7} \pm \frac{436 \times 12 \times 4}{592.4} = 12.3 \pm 35.3 = 23 \text{ psi. tension}$$

Even though the tests indicate that a wall could be expected to rupture at an average of 30 psi., the wall will be desigmed so there is no resulting tension in the mortar joint. If the cinder units above the midpoint of the vertical span are filled with concrete, it will give additional compressive forces. All cores above the midpoint are filled in sections of the west wall that are unbroken by openings. The cores of the first course can be filled on the ground and allowed to set before the blocks are placed in the wall. Cores of the other four courses above this one can be filled in place in the wall.

Core volumes in each block when filled with concrete give additional weight as figured below

$$4 \times \frac{0.7854 \times 3.21 \times 5 \times 7.75}{1728} \times 150 = 34 \text{ lb.}$$

This added weight gives stresses in the wall due to combined compressive and bending forces as follows:

$$f = \frac{1104}{126} + \frac{436 \times 12 \times 4}{671} = 8.8 + 31.2 = 22.4 \text{ psi. tension}$$

It is apparant that there is no advantage in filling the cores with concrete. In fact it is a definite disadvantage as far as the insulating properties of the wall are concerned and makes a damp wall.

The wall can be designed with tension steel spaced 6 ft. center to center and anchored in mortar placed in the cores. Use an 8 x 8 in. bond beam along the top of the wall and use hooked bars. Embed hooked dowels in the footing and cut them off two feet above the top of the footing. These dowels are of the same size as the vertical steel. The vertical reinforcement is placed $1\frac{1}{2}$ in. in from both the outside and the inside wall surfaces on 6 ft. centers.

With a stress intensity of 23 psi. at the extreme fiber, the stress intensity $1\frac{1}{2}$ in. in (thickness of face shell), by similar triangles is

$$\frac{2.5}{4} = \frac{x}{23} \qquad x = 14.4 \text{ psi.}$$

Then the tension in the mortar joint along the length of one block is

$$\frac{14.4 + 23}{2} \times 1.5 \times 15.75 = 442 \text{ lb.}$$

Tension over 4.5 blocks or 6 ft. is

$$442 \times 4.5 = 1{,}985 \text{ lb. tension}$$

$$A_s = \frac{T}{f_s} = \frac{1{,}985}{20{,}000} = 0.0994 \text{ sq. in.}$$

A for $\frac{1}{2}$ in. round rods is 0.20 sq. in.

This method of reinforcing is used in the west wall only.

Strength of Mortar Joints

No wall is stronger than its mortar joints. The amount of bedding area is important because the greater the area provided for mortar bedding, the stronger the wall. Full mortar bedding results in a stronger wall than face shell bedding, as the load is distributed over a greater area. The design of the unit also is an important factor, as it governs the bedding area, the shell and web thickness of the units, and their alignment in the wall.

With face shell bedding of the 3 - oval core, 8 x 8 x 16 in. unit, the mortar bedding area is 45 per cent of the gross area as given in Table 8, Facts About Concrete Masonry.

The unit stress under a loading of 80 psi. on the wall is 178 psi. To provide a factor of safety of 4, the mortar should have a strength of 4 x 178 or 712 psi. A 1:1:6 portland cement-lime or stronger mortar is recommended for all masonry wall construction. The 1:1:6 portland cement-lime mortar has a strength of 1,000 psi.

The National Concrete Masonry Association figures that for face shell bedding of mortar the wall strength is 42 per cent of the strength of the units.

The Houston Building Code requires a hollow concrete unit to test in compression to 1,000 psi. over the gross area. This would give a wall strength of 420 psi. gross area and a factor of safety of 5.25 for allowable loads of 80 psi. A factor of safety of 4 is usually required.

Roof Strength

The greatest velocity pressure exerted on the roof during the hurricane was 51.3 psf. pressure. This pressure was a combination of the live and dead loads. The dead load alone was 70.3 psf. Part of this weight was canceled by suction from the wind. Therefore the design will be for the deal load and a live load of 25 psf. as required by the Houston Building Code.

Effective depth is 4.5 in. with a 1 in. cover coat over the tension reinforcement. Table 2 of the A.C.I. Design Handbook gives a resisting moment of a rectangular section 1 ft. wide with an effective depth of 4.5 in. as 5 ft. kips. Other data used was

f_s = 20,000 psi. for reinforcement bars of structural grade

f'_c = 3,000 psi.

$f_c = 0.40\ f'_c = 1,200$ psi.

$n = 10$

The bending moment is now calculated

Span = 19 ft. 7 in. = 19.58 ft.

w = 95.3 psf.

$$\text{B.M.} = \frac{1}{8} wl^2 = \frac{1}{8} \times 95.3 \times 19.58^2 = 4,570 \text{ ft. lb.}$$

This moment neglects the overhang and gives results that are conservative.

$4,570 < 5,000$, therefore the roof meets the code specifications.

Tension steel required for the roof slab is now figured

$$M = Tjd = A_s f_s jd$$

From Table 1, A.C.I. Design Handbook

$j = 0.875$

$$A_s = \frac{M}{f_s jd} = \frac{5,000 \times 12}{20,000 \times 0.875 \times 4.5} = 0.761 \text{ sq. in.}$$

From Table 3, A.C.I. Design Handbook, 0.75 sq. in. call for $\frac{3}{4}$ in. round bars spaced 7 inches center to center. Use $\frac{1}{2}$ in. round bars spaced 1 ft. center to center as spacers.

Steel is also required for stresses developed due to negative moment where the roof slab passes over the outside wall.

$M = Tjd = A_s f_s jd$

$$A_s = \frac{M}{f_s jd} = \frac{84.3 \times 12}{20,000 \times 0.875 \times 4.5} = 0.0129 \text{ sq. in.}$$

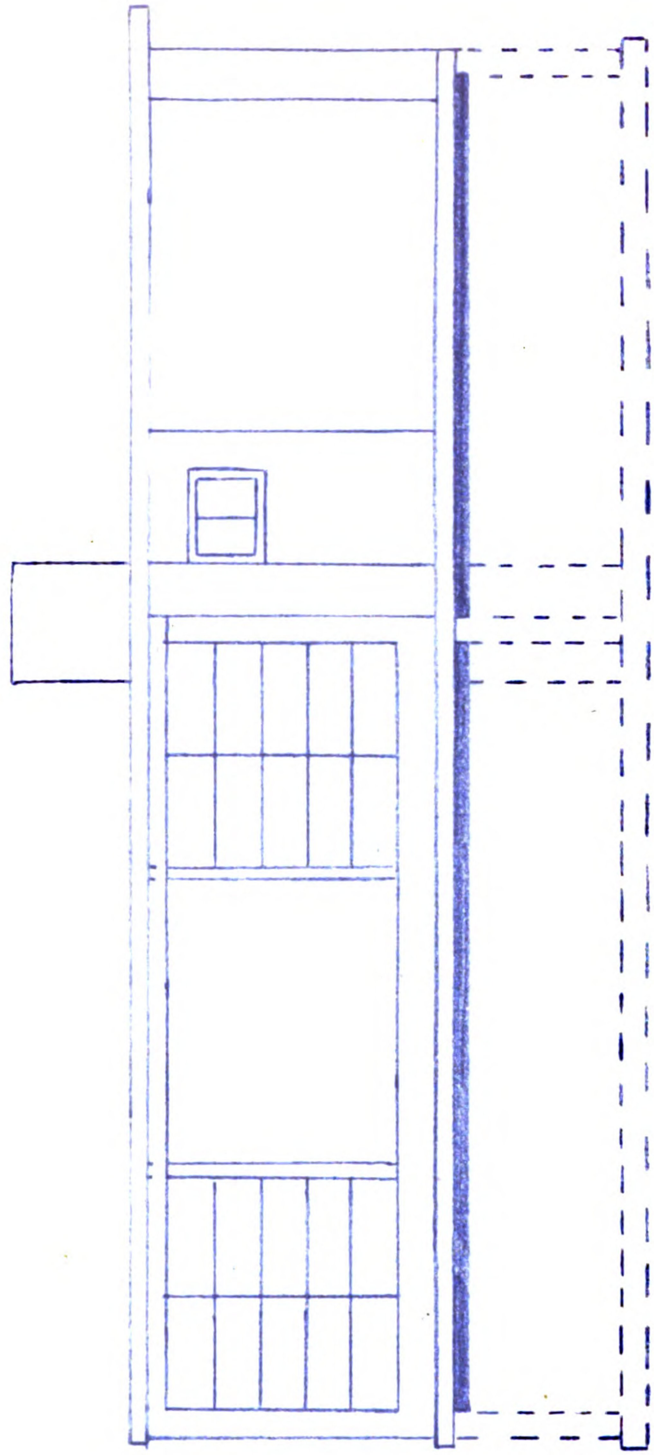
Use a $\frac{3}{8}$ in. round bar (A_s = 0.11 sq. in.) two feet long and spaced 21 in. center to center around the outside walls.

When the soffit forms for pouring the roof slab are placed, quarter round moulding is placed around the circumference of the roof and three inches in from the edge, to be later removed and form a drip. The roof does not need anchoring as it has sufficient weight.

Design of Lintels

Door lintel

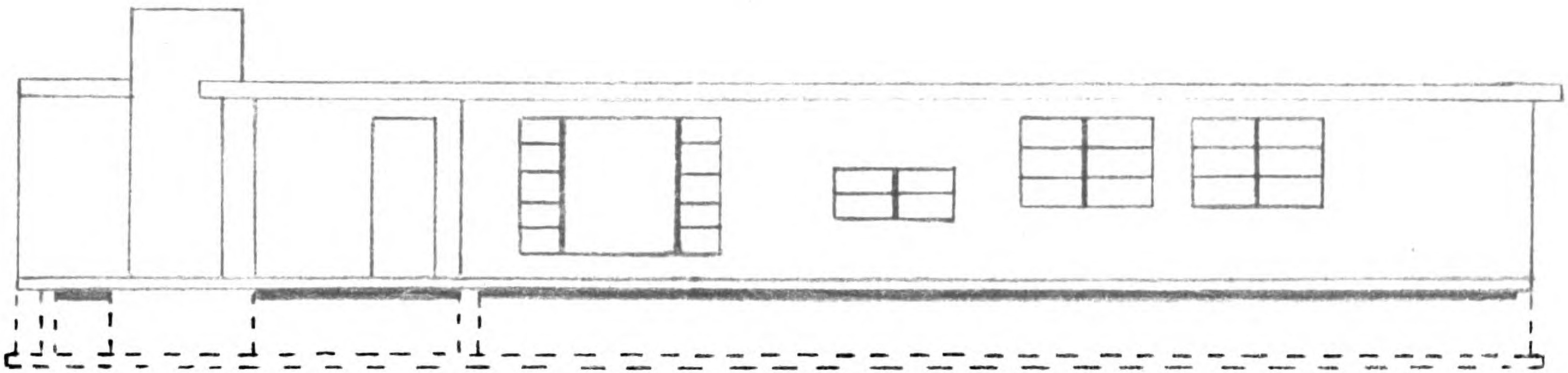
The National Concrete Masonry Association calls for a lintel 5 $\frac{3}{4}$ in. high and eight inches wide for clear spans up to 7 ft. It uses two $\frac{3}{8}$ in. round deformed bars placed $1\frac{1}{2}$ in. above the bottom and 2 in. in from the sides.



SCALE: 1"=5'

SOUTH ELEVATION

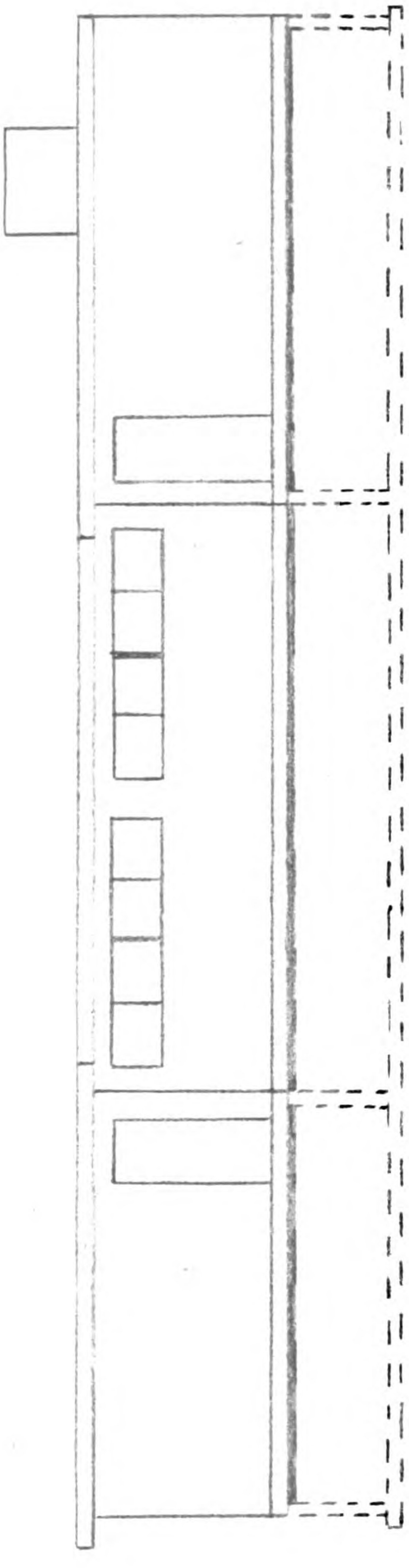
FIG. 10



EAST ELEVATION

SCALE: 1 CM. = 3'

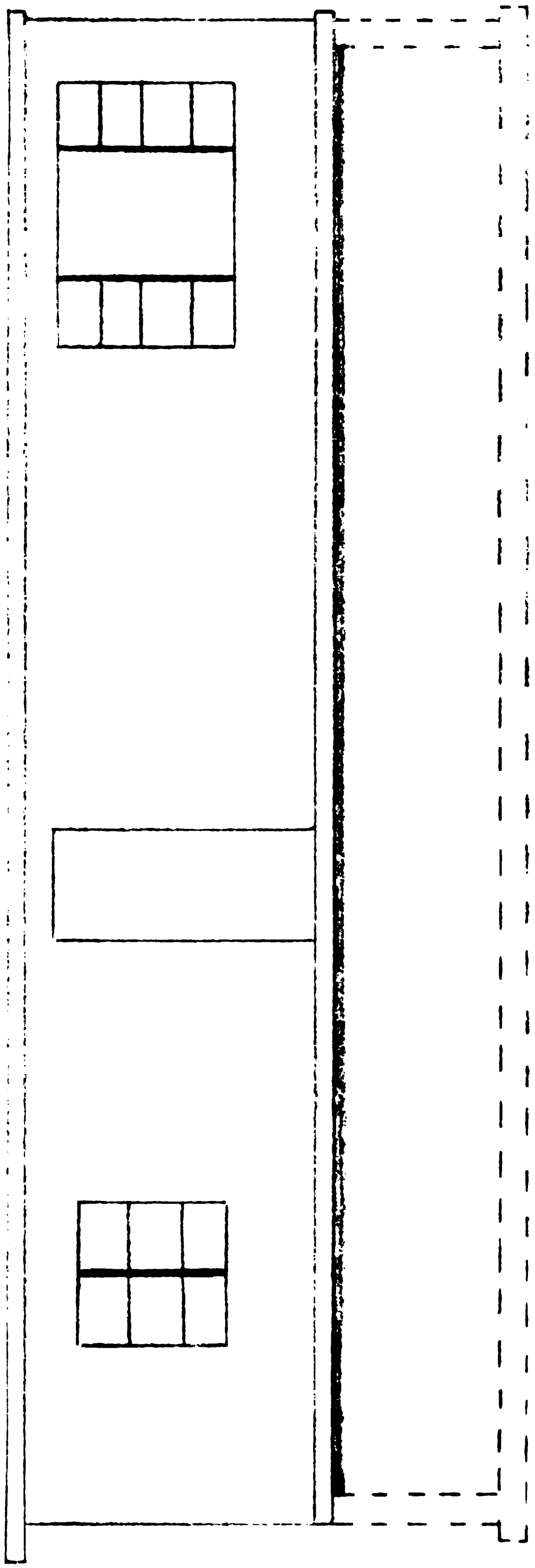
FIG. 11



WEST ELEVATION

SCALE: 1 CM = 3'

FIG. 12



NORTH ELEVATION

SCALE. 1"=5'

FIG. 13

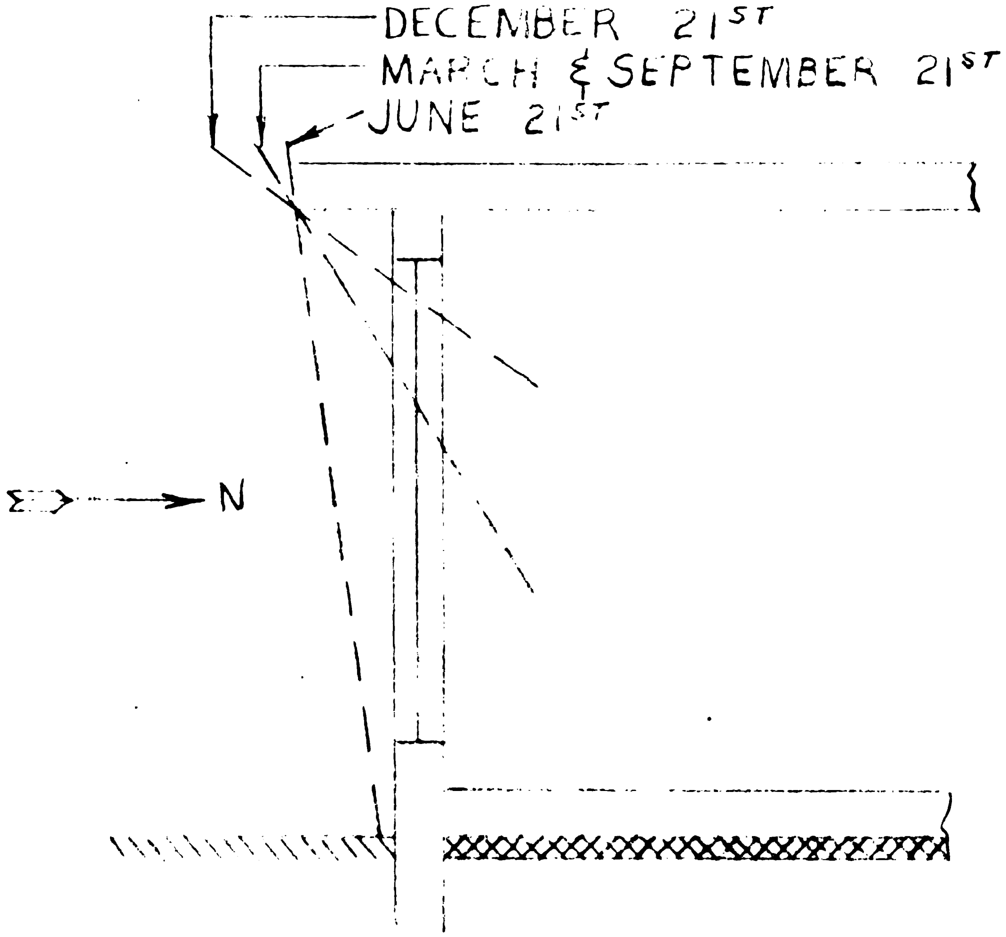
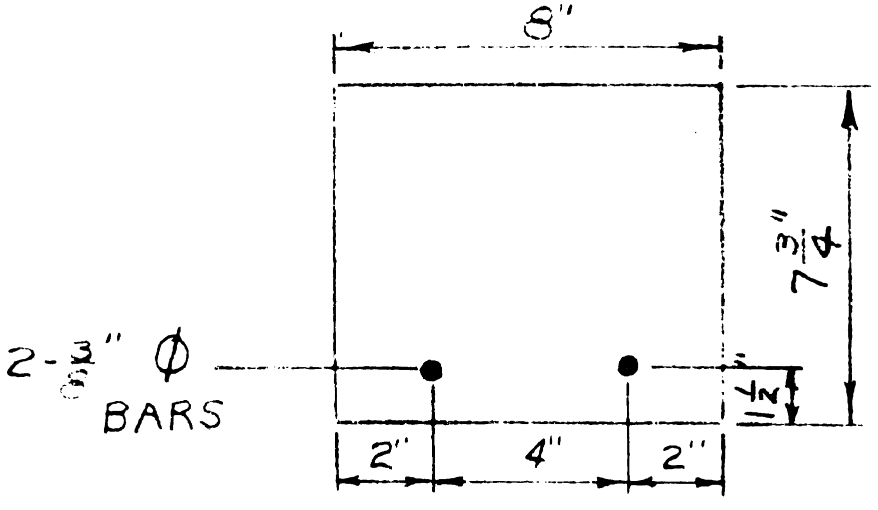


DIAGRAM OF SUN ANGLES
FIG. 14



DOOR LINTEL

NO SCALE

FIG. 15

With a clear opening of 33 in. and a bearing area extending 4 in. on each side use a design span of 37 in. and design as a simple beam. Roof load is 70.3 psf.

Suggested design by N.C.M.A. is shown in Fig. 15. The design will be investigated as shown on page 17, A.C.I. Design Handbook.

Unit load per running foot of lintel is

$$70.3 \times \frac{21.583}{2} + \frac{8}{12} \times \frac{8}{12} \times 1 \times 150 = 827 \text{ lb. per ft.}$$

$$M = \frac{1}{8} wl^2 = \frac{827}{8} \times 3.08^2 = 983 \text{ ft. lb.}$$

Given:

$b = 8$ in.; $d = 6\frac{1}{4}$ in.; $A_s = 0.22$ sq. in.; $n = 10$

$M = 983$ ft. lb.

$$m = q = \frac{nA_s}{bd} = \frac{10 \times 0.22}{8 \times 6.25} = 0.044$$

From Table 11

$$k = \sqrt{m + 2q} - m = \sqrt{0.044 + 0.088} - 0.088 = 0.212$$

From Table 13

for $z = 0.33$ and $k = 0.212$: $j = 0.93$

$$f_s = \frac{12M}{jdA} = \frac{12 \times 983}{0.93 \times 6.25 \times 0.22} = 9{,}240 \text{ psi.}$$

Sec. 2613, H.B.C.: allowable $f_s = 20{,}000$ psi.

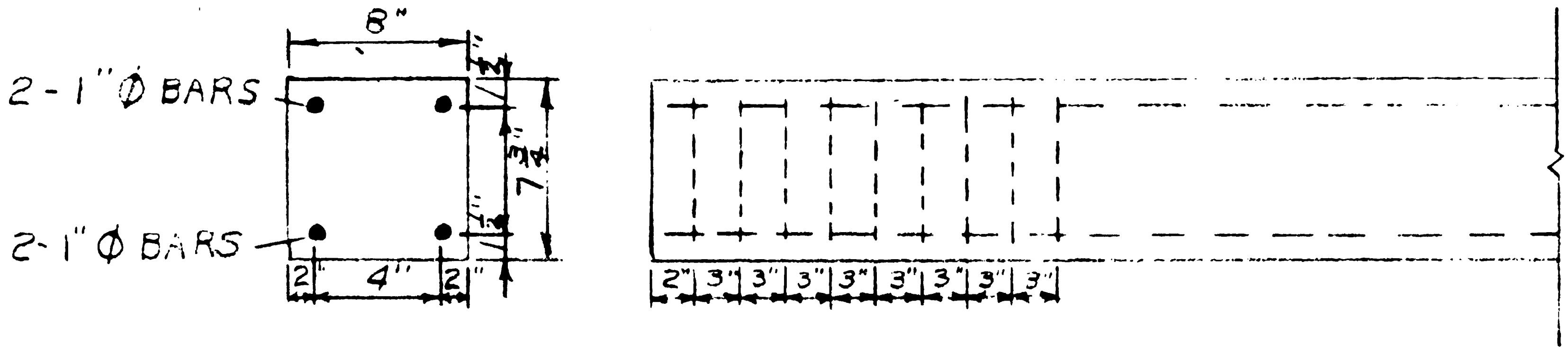
$$f_c - \frac{f_s}{n} \times \frac{k}{1-k} = \frac{9{,}240}{10} \times \frac{0.212}{1 - 0.212} = 249 \text{ psi.}$$

Sec. 2613, H.B.C.: allowable $f_c = 1{,}200$ psi.

The design is acceptable.

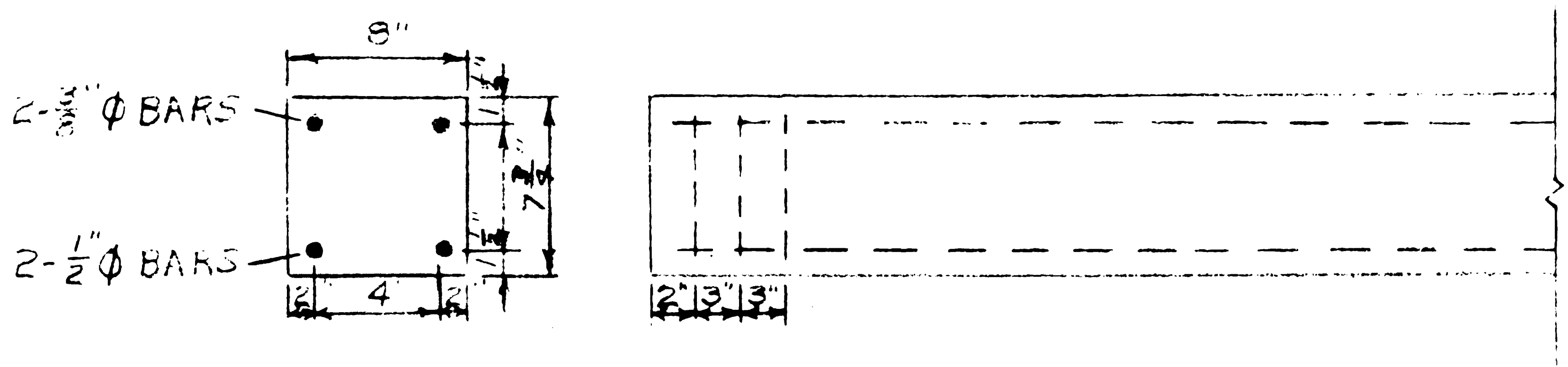
Master Bedroom Window Lintel

Suggested design by N.C.M.A. is shown in Fig. 16. The design will be investigated as shown on page 17, A.C.I. Design



MASTER BEDROOM WINDOW LINTEL

FIG. 16



UTILITY ROOM LINTEL

FIG. 17

NO SCALE

Handbook. The opening is 6 ft. 10 in. and the design span is 7 ft. 6 in.

Given:

$b = 8$ in.; $d = 6\frac{1}{4}$ in.; $d' = 1\frac{1}{2}$ in.; $A_s = 1$ sq. in.

$A_s' = 1$ sq. in.; $n = 10$; $w = 827$ lb. per ft.

$$M = \frac{1}{8} wl^2 = \frac{827}{8} \times 7.5^2 = 5{,}820 \text{ ft. lb.}$$

$$m = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} = \frac{10 \times 1}{8 \times 6.25} + \frac{9 \times 1}{8 \times 6.25} = 0.20 + 0.18$$

$$= 0.38$$

$$q = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} \times \frac{d'}{d} = \frac{10 \times 1}{8 \times 6.5} + \frac{9 \times 1}{8 \times 6.5} \times \frac{1.5}{6.25} = 0.20 +$$

$$0.043 = 0.243$$

From Table 11, for $m = 0.38$ and $q = 0.243$: $k = 0.414$

For entering Table 12 determine

$$\frac{1}{k} \times \frac{(n-1)A_s'}{bd} = \frac{1}{0.414} \times \frac{9 \times 1}{8 \times 6.25} = 0.434$$

$$\frac{1}{k} \times \frac{d'}{d} = \frac{1}{0.414} \times \frac{1.5}{6.25} = 0.579$$

From Table 12:

$$z = \frac{\frac{1}{6} + \frac{(n-1)A_s'}{kbd} \times \frac{d'}{kd} \times \left(1 - \frac{d'}{kd}\right)}{\frac{1}{2} + \frac{(n-1)A_s'}{kbd} \times \left(1 - \frac{d'}{kd}\right)} = \frac{0.167 + 0.434 \times 0.579}{0.5 + 0.434 \times 0.421}$$

$$\times 0.421 = \frac{0.167 + 0.0768}{0.5 + 0.18} = 0.359$$

From Table 13, for $z = 0.359$ and $k = 0.414$: $j = 0.853$

$$f_s = \frac{12M}{jdA_s} = \frac{12 \times 5{,}820}{0.853 \times 6.25 \times 1} = 13{,}100 \text{ psi.}$$

Sec. 2613, H.B.C.: allowable $f_s = 20{,}000$ psi.

f_s' always less than nf_c or less than 9,280 psi. when

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k} = \frac{13{,}100}{10} \times \frac{0.414}{1-0.414} = 928 \text{ psi.}$$

Shear $= \frac{wl}{2} = \frac{827}{2} \times 7.5 = 3{,}100$ lb.

Sec. 2618, H.B.C.

$v = \frac{V}{bjd} = \frac{3{,}100}{8 \times 0.853 \times 6.25} = 72.7$ psi.

Sec. 2613, H.B.C. gives allowable $v = 0.02\, f_c' = 0.02 \times 3{,}000 =$ 60 psi. with no web reinforcement. Requires web reinforcement. Allowable shear with web reinforcement $= 0.06 \times 3{,}000 = 180$ psi.

Stirrups are made of No. 6 gage cold drawn steel wire with a tensile strength of 16,000 psi. Sec. 3613, H.B.C. Formerly stirrups were allowed to carry 20,000 psi. Design will be by the method on page 30, A.C.I. Design Handbook. The shear diagram is triangular.

Given:

$v' = 12.7$ psi.; $b = 8$ in.; $d = 6.25$ in.; $f_v = 16{,}000$ psi.

$f_c' = 3{,}000$ psi.; No. 6 wire stirrups; $A_v = 0.0648$ sq. in.

$\frac{S}{3.75} = \frac{12.7}{72.7}$ $\quad S = 0.656$ ft.

$A_v f_v = 16{,}000 \times 0.0648 = 1{,}040$

$\frac{1}{s} = \frac{v'b}{A_v f_v} = \frac{12.7 \times 8}{1{,}040} = 0.098$

$N = 6S(\frac{1}{s}) = 6 \times 0.656 \times 0.098 = 1$ stirrup

Index $= \frac{1.5\, S}{\frac{1}{s}} = \frac{1.5 \times 0.656}{0.098} = 10$

From diagram 17, 1 stirrup is required.

Stirrups can be securely fastened to compression steel and hence will not require any minimum depth of embedment. The suggested design by the N.C.M.A. is shown in Fig. 16 and will be adopted.

Utility Room Lintel

Clear opening is 3 ft. 8 in. and the design span is 4 ft. 4 in. Suggested design by the N.C.M.A. is shown in Fig. 17. The design will be investigated by the method shown on page 17, A.C.I. Design Handbook.

Given:

$b = 8$ in.; $d = 6.25$ in. ; $d' = 1.5$ in.; $A_s = 0.40$ sq. in.

$A_s' = 0.22$ sq. in.; $n = 10$; $w = 827$ lb. per ft.

$M = \frac{1}{8} \times 827 \times 4.25^2 = 1{,}865$ ft. lb.

$$m = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} = \frac{10 \times 0.40}{8 \times 6.25} + \frac{9 \times 0.22}{8 \times 6.25} = 0.08 + 0.0397$$

$$= 0.120$$

$$z = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} \times \frac{d'}{d} = 0.08 + 0.0379 \times \frac{1.5}{6.25} = 0.08 + 0.0091$$

$$= 0.0891$$

From Table 11, for $m = 0.12$ and $q = 0.0891$: $k = 0.318$

For entering Table 12 determine

$$\frac{1}{k} \times \frac{(n-1)A_s'}{bd} = \frac{1}{0.318} \times \frac{9 \times 0.22}{8 \times 6.25} = 0.125$$

$$\frac{1}{k} \times \frac{d'}{d} = \frac{1}{0.318} \times \frac{1.5}{6.25} = 0.756$$

From Table 12: $z = 0.377$ determined as follows

$$z = \frac{\frac{1}{6} + \frac{(n-1)A_s'}{kbd} \times \frac{d'}{kd} \times (1 - \frac{d'}{kd})}{\frac{1}{2} + \frac{(n-1)\ A_s'}{kbd} \times (1 - \frac{d'}{kd})} = \frac{0.167 + 0.0125 \times 0.756 \times 0.244}{0.5 + 0.0125 \times 0.244}$$

$$= \frac{0.167 + 0.023}{0.5 + 0.003} = \frac{0.190}{0.503} = 0.377$$

From Table 13, for $z = 0.377$ and $k = 0.318$: $j = 0.87$

$$f_s = \frac{12M}{jdA_s} = \frac{12 \times 1{,}865}{0.87 \times 6.25 \times 0.40} = 10{,}300 \text{ psi.}$$

$f_c = \frac{f_s}{10} \times \frac{k}{1-k} = \frac{10,300}{10} \times \frac{0.318}{0.682} = 482$ psi.

f_s' is always less than nf_c or less than 4,820 psi.

Sec. 2613, H.B.C. : allowable $f_s = 20,000$ psi.

allowable $f_c = 1,200$ psi.

Shear $= \frac{wl}{2} = \frac{827}{2} \times 4.25 = 1,760$ lb.

Sec. 2618, H.B.C.

$v = \frac{V}{bjd} = \frac{1,760}{8 \times 0.87 \times 6.25} = 40.4$ psi.

Sec. 2613, H.B.C. gives allowable $v = 0.02 f_c' = 60$ psi. without web reinforcement. Stirrups shown in Fig. 17 are not necessary.

Laundry and Breakfast Alcove Lintels

Clear opening is 5 ft. 7 in. Design span is 6 ft. 3 in. Suggested design by N.C.M.A. is shown in Fig. 18. The design will be investigated by the method shown on page 17, A.C.I. Design Handbook.

Given:

$b = 8$ in.; $d = 6.25$ in.; $d' = 1.5$ in.; $A_s = 0.88$ sq. in.

$A_s' = 0.88$ sq. in.; $n = 10$; $w = 827$ lb. per ft.

$M = \frac{827}{8} \times 6.25^2 = 4,030$ ft. lb.

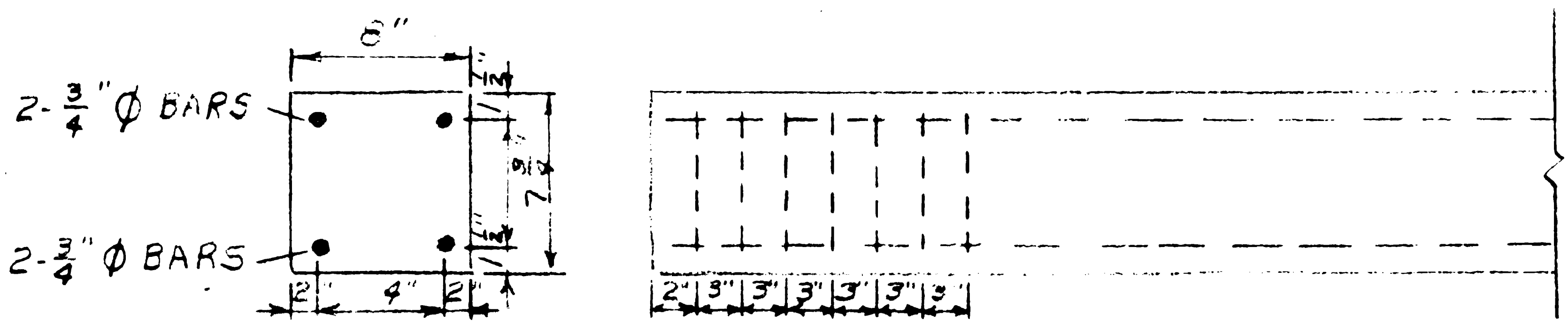
$m = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} = \frac{10 \times 0.88}{8 \times 6.25} + \frac{9 \times 0.88}{8 \times 6.25} = 0.176 + 0.159$

$= 0.335$

$q = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} \times \frac{d'}{d} = 0.176 + 0.159 \times 0.24 = 0.214$

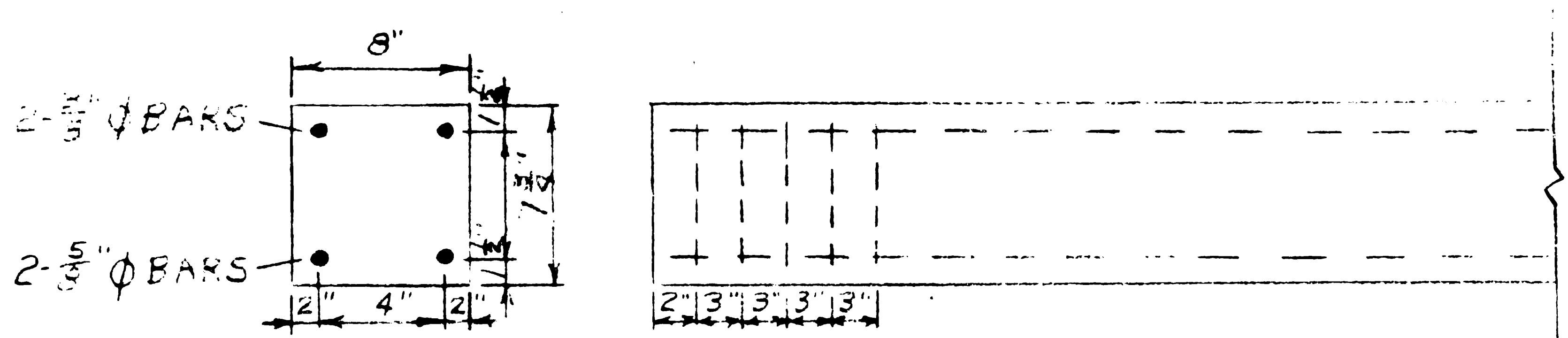
From Table 11, for $m = 0.335$ and $q = 0.214$: $k = 0.40$

For entering Table 12 determine



LAUNDRY & BREAKFAST ALCOVE WINDOW LINTELS

FIG. 18



KITCHEN WINDOW LINTEL

FIG. 19

NO SCALE

$$\frac{1}{k} \times \frac{(n-1)A_s'}{bd} = \frac{1}{0.4} \times \frac{9 \times 0.88}{8 \times 6.25} = 0.396$$

$$\frac{1}{k}\frac{d'}{d} = \frac{1}{0.4} \times 0.24 = 0.60$$

$$z = \frac{\frac{1}{6} + \frac{(n-1)A_s'}{kbd} \times \frac{d'}{kd} \times (1-\frac{d'}{kd})}{\frac{1}{2} + \frac{(n-1)A_s'}{kbd} \times (1-\frac{d'}{kd})} = \frac{0.167 + 0.396 \times 0.60 \times 0.40}{0.5 + 0.396 \times 0.40}$$

$$= \frac{0.167 + 0.095}{0.5 + 0.159} = 0.397$$

From Table 13, for $z = 0.397$ and $k = 0.40$: $j = 1-zk =$ $1-0.40 \times 0.397 = 0.841$

$$f_s - \frac{12M}{jdA_s} = \frac{12 \times 4,030}{0.84 \times 6.25 \times 0.88} = 10,500 \text{ psi.}$$

$$f_c - \frac{f_s}{n} \times \frac{k}{1-k} = \frac{10,500}{10} \times \frac{0.4}{1-0.4} = 701 \text{ psi.}$$

f_s' is always less than nf_c or less than 7,010 psi.

Sec. 2613, H.B.C.: allowable $f_s = 20,000$ psi

allowable $f_c = 1,200$ psi.

$$\text{Shear} = \frac{wl}{2} = \frac{827}{2} \times 6.25 = 2,580 \text{ lb.}$$

Sec. 2618, H.B.C.

$$v = \frac{V}{bjd} = \frac{2,580}{8 \times 0.841 \times 6.25} = 61.4 \text{ psi.}$$

Sec. 2613, H.B.C. gives allowable $v = 60$ psi. without web reinforcement. Since our span is 5 in. less than the span of the suggested design the stirrups will be considered as unnecessary.

Kitchen Lintel

Clear opening is 5 ft. Design span is 5 ft. 8 in. Suggested design by N.C.M.A. is shown in Fig 19. The design will be investigated by the method shown on page 17, A.C.I.

Design Handbook.

Given:

$b = 8$ in.; $d = 6.25$ in.; $d' = 1.5$ in.; $A_s = 0.62$ sq. in.

$A_s' = 0.22$ sq. in.; $n = 10$

$$M = \frac{827}{8} \times 5.67^2 = 3,320 \text{ ft. lb.}$$

$$m = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} = \frac{10 \times 0.62}{8 \times 6.25} + \frac{9 \times 0.22}{8 \times 6.25} = 0.124 + 0.0198$$

$$= 0.144$$

$$q = \frac{nA_s}{bd} + \frac{(n-1)A_s'}{bd} \times \frac{d'}{d} = 0.124 + 0.0198 \times 0.24 =$$

$$= 0.124 + 0.005 = 0.129$$

From Table 11, for $m = 0.144$ and $q = 0.129$: $k = 0.384$

For entering Table 12 determine

$$\frac{1}{k} \times \frac{(n-1)A_s'}{bd} = \frac{1}{0.384} \times \frac{9 \times 0.22}{50} = 0.103$$

$$\frac{1}{k} \times \frac{d'}{d} = \frac{1}{0.384} \times 0.24 = 0.625$$

From Table 12

$$z = \frac{\frac{1}{6} + \frac{(n-1)A_s'}{kbd} \times \frac{d'}{kd} \times (1-\frac{d'}{kd})}{\frac{1}{2} + \frac{(n-1)A_s'}{kbd} \times (1-\frac{d'}{kd})} = \frac{0.167 + 0.103 \times 0.625 \times 0.375}{0.5 + 0.103 \times 0.375}$$

$$= \frac{0.167 + 0.024}{0.5 + 0.039} = 0.354$$

From Table 13, for $z = 0.354$ and $k = 0.384$: $j = 0.868$

$$f_s = \frac{12M}{jdA_s} = \frac{12 \times 3,320}{0.868 \times 6.25 \times 0.62} = 11,860 \text{ psi.}$$

$$f_c - \frac{f_s}{n} \times \frac{k}{1-k} = \frac{11,860}{10} \times \frac{0.384}{0.616} = 7,410 \text{ psi.}$$

Sec. 2613, H.B.C.: allowable $f_s = 20,000$ psi.

allowable $f_c = 1,200$ psi.

$$\text{Shear} = \frac{wl}{2} = \frac{827}{2} \times 5.67 = 2,340 \text{ lb.}$$

Sec. 2618, H.B.C.

$$v = \frac{V}{bjd} = \frac{2,340}{50 \times 0.868} = 53.8 \text{ psi.}$$

Sec. 2613, H.B.C. gives allowable $v = 0.02\, f'_c = 60$ psi. without web reinforcement. Stirrups shown in Fig. 19 are not necessary.

Dining Room Lintel

Clear opening is 8 ft. 6 in. Design span is 9 ft. 2 in. The N.C.M.A. gives no suggested designs for spans over 7 ft. Design will be by the method on page 9, A.C.I. Design Handbook.

Given:

$f_s = 20,000$ psi.; $n = 10$; $f_c = 1,200$ psi.; $b = 8$ in.

$d = 6.25$ in.; $d' = 1.5$ in.

$$M = \frac{827}{8} \times 9.167^2 = 8,680 \text{ ft. lb.}$$

From Table 1, for 20,000/10/1,200: $k = 197$

From Table 4, for $b \times d = 8 \times 6.25$: $F = 0.026$

then

$$\begin{aligned} M &= & 8.68 \\ KF &= 197 \times 0.026 = & \underline{5.12} \\ M - KF &= & 3.56 \end{aligned}$$

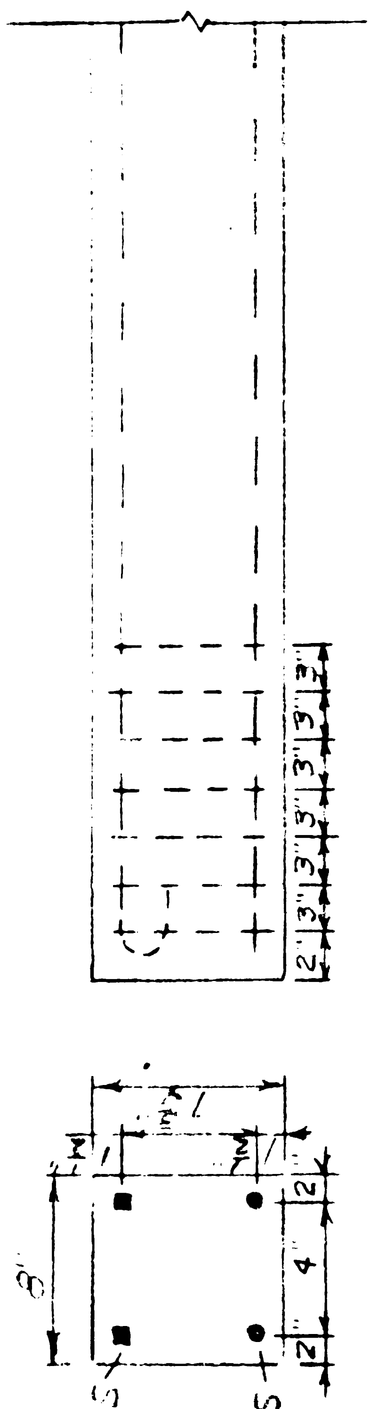
Compressive reinforcement is required when $(M - KF)$ is positive, since this is the residual moment not taken by the concrete.

From Table 7, for 20,000/10/1,200 and $\frac{d'}{d} = 0.24$

$$c = \frac{f_s\,(n-1)(1-\frac{d'}{d})(k-\frac{d'}{d})}{12,000\, n\,(1-k)} = \frac{20,000 \times 9(1-0.24)(0.375-0.24)}{12,000 \times 10(1-0.375)}$$

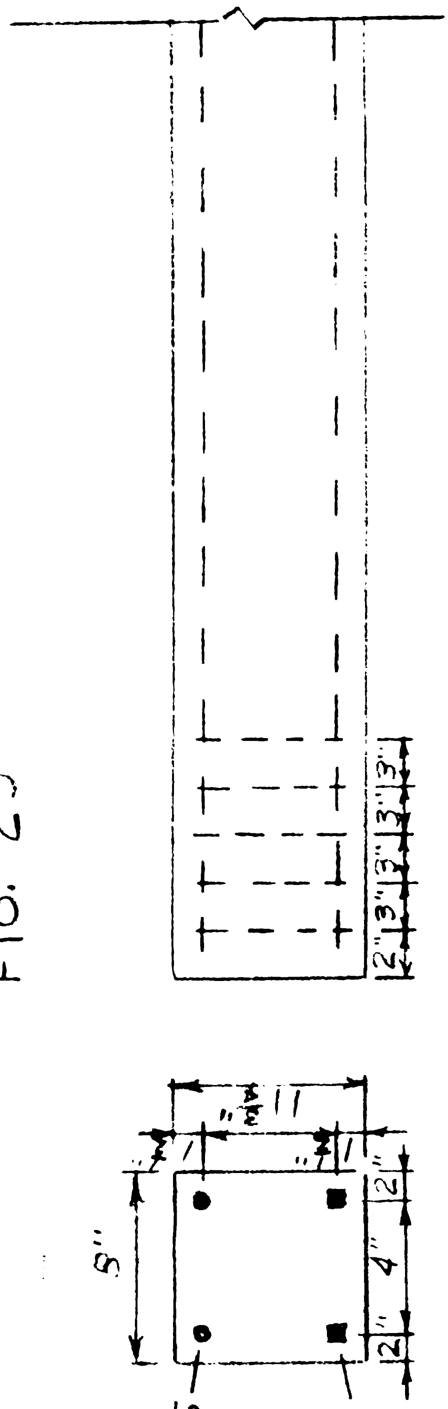
$$= \frac{180,000 \times 0.76 \times 0.135}{120,000 \times 0.625} = 0.246$$

NO SCALE



DINING ROOM WINDOW LINTEL

FIG. 20



GALLERY WINDOW LINTELS

FIG. 21

$A_s' = \frac{M-KF}{cd} = \frac{3.56 \quad -}{0.246 \times 6.25} = 2.32$ sq. in.

From Table 1, for $f_s = 20,000$: $a = 1.44$, therefore

$A_s = \frac{A}{ad} = \frac{8.68}{1.44 \times 6.25} = 0.967$ sq. in.

With u = allowable bond stress, compute $\Sigma_o = \frac{8,000\ V}{7ud}$ select bars from Table 5, and check width of web required to accommodate bars from Table 6.

Sec. 2613, H.B.C., $u = 0.04\ f_c' = 0.04 \times 3,000 = 120$ psi.

$V = \frac{wl}{2} = \frac{827}{2} \times 9.167 = 3,780$ lb.

$= \frac{8,000 \times 3.78}{7 \times 120 \times 6.25} = 5.78$ in.

From Table 5, use two 1 $\frac{1}{8}$ in. square bars at the top of the lintel and two $\frac{7}{8}$ in. round bars at the bottom of the lintel to give 3.74 sq. in. and $\Sigma_o = 14.5$ in. From Table 6, minimum web widths for these two combinations are 8.5 in. and 8 in. respectively. With special anchorage minimum web width is 8 in. In accordance with Sec. 2619, H.B.C., the two 1 $\frac{1}{8}$ in. bars will be bent down to form a semi-circular hook of minimum radius of 4 bar diameters.

Sec. 2618, H.B.C.

$v = \frac{V}{bjd} = \frac{3,780}{50 \times 0.875} = 86.5$ psi.

Sec. 2613, H.B.C. gives allowable $v = 0.02\ f_c' = 60$ psi. without web reinforcement.

Stirrup design will be as shown on page 30, A.C.I. Design Handbook.

Given:

$v' = 26.5$ psi.; $b = 8$ in.; $d = 6.25$ in.; $f_v = 16{,}000$ psi.

$f'_c = 3{,}000$ psi.; No. 6 wire U-stirrups; $A_v = 0.0648$ sq. in.

$$\frac{S}{4.583} = \frac{26.5}{86.5} \qquad S = 1.41$$

From diagram 17, for $f_v = 16{,}000$ and No. 6 wire U-stirrups:

$A_v f_v = 1{,}040$

$$\frac{1}{s} = \frac{v'b}{A_v f_v} = \frac{26.5 \times 6.25}{1{,}040} = 0.16$$

$$N = 6S\frac{1}{s} = 6 \times 1.41 \times 0.16 = 2 \text{ stirrups}$$

$$\text{Index} = \frac{1.5\ S}{\frac{1}{s}} = \frac{1.5 \times 1.41}{0.16} = 13.2$$

From diagram 17, 2 stirrups are needed. The report of the A.C.I. Joint Committee on Standard Specifications for Concrete and Reinforced Concrete gives the spacing of vertical stirrups as

$$s = \frac{A_v f_v jd}{V'}, \quad \text{where } V' = v'bjd = 26.5 \times 0.875 \times 50 = 1{,}160 \text{ lb.}$$

$$s = \frac{1{,}040 \times 0.875 \times 6.25}{1{,}160} = 4.9 \text{ in.}$$

$S = 1.41 \times 12 = 16.9$ in.

Use 7 stirrups, spaced 2,3,3,3,3,3,3 in. as shown in Fig. 20. Stirrups are to be securely fastened to compression steel.

Gallery Window Lintels

Lintels extend over two double window units and have a 8 in. bearing width at each end. Clear opening of each double unit is 10 ft. 8 in. Two identical designs will be used to eliminate the necessity of additional steel at the top of the lintel to take care of the negative moment which would result with a lintel continuous over two spans. Design span is 11 ft. 4 in. Design will be by the method on page 9, A.C.I. Design Handbook.

Given:

$f_s = 20{,}000$ psi.; $n = 10$; $f'_c = 3{,}000$ psi.; $b = 8$ in.

$f_c = 1{,}200$ psi.;

$M = \frac{827}{8} \times 11.33^2 = 13{,}260$ ft. lb.

From Table 1, for 20,000/10/1,200; $k = 197$

From Table 4, for $b \times d = 8 \times 6.25$: $F = 0.026$

$M =$ 13.26

$KF = 197 \times 0.026 = \underline{5.12}$

$M\text{-}KF =$ 8.14

Compressive reinforcement is required since (M-KF) is positive.

From Table 7, for 20,000/10/1,200 and $\frac{d'}{d} = 0.24$: $c = 0.246$

$A'_s = \frac{M-KF}{cd} = \frac{8.14}{0.246 \times 6.25} = 5.3$ sq. in.

This would require 4-1¼ in. square bars and a minimum web width of 9 in. without special anchorage or 8.5 in. with special anchorage. Redesign with an 8 in. width and 11 $\frac{3}{4}$ in. depth.

$w = 827 + \frac{8}{12} \times \frac{4}{12} \times 1 \times 150 = 860$ lb. per ft.

From Table 4, for $b \times d = 8 \times 10.25$: $F = 0.0705$

$M = \frac{860}{8} \times 11.332^2 = 13.80$

$KF = 197 \times 0.0705 = \underline{13.90}$

$M\text{-}KF =$ $-$ 0.10

Compressive reinforcement is not required since (M-KF) is negative.

From Table 1, for $f_s = 20{,}000$: $a = 1.44$, therefore

$$A_s = \frac{M}{ad} = \frac{13.8}{1.44 \times 10.25} = 0.935 \text{ sq. in.}$$

Sec. 2613, H.B.C., $u = 0.04\, f'_c = 120$ psi.

$$V = \frac{wl}{2} = \frac{860}{2} \times 11.33 = 4{,}870 \text{ lb.}$$

$$\Sigma_o = \frac{8{,}000\, V}{7ud} = \frac{8{,}000 \times 4.87}{7 \times 120 \times 10.25} = 4.51 \text{ in.}$$

From Table 5, for $\Sigma_o = 4.51$ in. and $A_s = 0.935$ sq. in. :

Two $\frac{1}{2}$ in. square bars give $A_s = 0.94$ sq. in. and $\Sigma_o = 6.4$ in.

From Table 6, minimum web width is 6.5 in.

Sec. 2618, H.B.C.

$$v = \frac{V}{bjd} = \frac{4{,}870}{8 \times 0.875 \times 10.25} = 68 \text{ psi.}$$

Sec. 2613, H.B.C. gives allowable $v = 0.02\, f'_c = 60$ psi. without web reinforcement. Stirrups are needed.

Stirrup design will be as shown on page 30, A.C.I. Design Handbook.

Given:

$v' = 8$ psi.; $b = 8$ in.; $f_v = 16{,}000$ psi.; $f'_c = 3{,}000$ psi.

No. 6 wire U-stirrups; $A_v = 0.0648$ sq. in.

$$\frac{S}{5.66} = \frac{8}{68} \qquad S = 0.666 \text{ ft.}$$

$$A_v f_v = 16{,}000 \times 0.0648 = 1{,}040$$

$$\frac{1}{s} = \frac{v'b}{A_v f_v} = \frac{8 \times 8}{1{,}040} = 0.0615$$

$$N = 6S\left(\frac{1}{s}\right) = 6 \times 0.666 \times 0.0615 = 1 \text{ stirrup}$$

$$\text{Index} = \frac{1.5\, S}{\frac{1}{s}} = \frac{1.5 \times 0.666}{0.0615} = 16.3$$

From diagram 17, 1 stirrup is required.

The A.C.I. Joint Committee gives the spacing of vertical sitrrups as

$s = \frac{A_v f_v jd}{V'}$ where $V' = v'bjd = 8 \times 0.875 \times 8 \times 10.25 = 573$ lb.

$$s = \frac{1,040 \times 0.875 \times 10.25}{8 \times 0.875 \times 8 \times 10.25} = 16.3 \text{ in.}$$

$S = 0.66 \times 12 = 8$ in.

Use 5 U-stirrups spaced 2,3,3,3,3 in. It will be necessary to use two $\frac{3}{8}$ in. round bars at the top of the lintel to anchor the stirrups. Design is shown in Fig. 21.

Living Room Lintel

Clear opening is 20 ft. Design span is 20 ft. 8 in. Design will be by methods of pages 8 and 9, A.C.I. Design Handbook.

Given:

$f_s = 20,000$ psi.; $n = 10$; $f_c = 1,200$ psi.; $b = 8$ in.

$f_c' = 3,000$ psi.; $w = 827$ lb. per ft.

The value of w is only approximate since it is the loading due to combined dead and live loads on a lintel 7 $\frac{3}{4}$ in. deep.

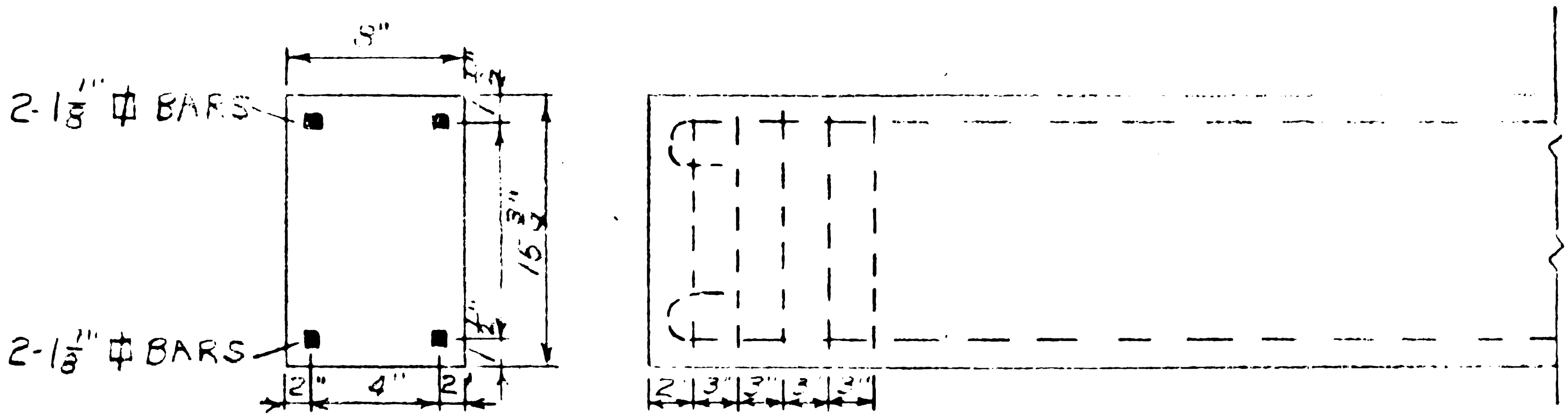
$$M = \frac{827}{8} \times 20.667^2 = 44,100 \text{ ft. lb.}$$

From Table 1, for 20,000/10/1,200, $K = 197$

$$\frac{M}{K} = \frac{44.1}{197} = 0.224 = F$$

From Table 4, select $b \times d = 8 \times 18.5$ $(F = 0.228)$

An overall depth of 20 in. would be required with no compressive reinforcement.



LIVING ROOM LINTEL

FIG. 22

NO SCALE

Redesign with a depth of 15 $\frac{3}{4}$ in.

$w = 827 + \frac{8}{12} \times \frac{8}{12} \times 1 \times 150 = 894$ lb. per ft.

$M = \frac{894}{8} \times 20.667^2 = 47{,}750$ ft. lb.

$d = 14.25$ in.

From Table 4, for $b \times d = 8 \times 14.25$: $F = 0.1355$

$M = \qquad\qquad 47.75$

$KF = 197 \times 0.1355 = \underline{26.7}$

$M\text{-}KF = \qquad\qquad 21.05$

Compressive reinforcement is required since (M-KF) is positive.

From Table 7, for 20,000/10/1,200 and $\frac{d'}{d} - 0.105$: $c - 0.59$

$A' - \frac{M\text{-}KF}{cd} - \frac{21.05}{0.59 \times 14.25} - 2.5$ sq. in.

From Table 1, for $f - 20{,}000$: $a - 1.44$

$A - \frac{M}{ad} - \frac{47.75}{1.44 \times 14.25} - 2.33$ sq. in.

Sec. 2613, H.B.C., $u - 0.04\ f' - 120$ psi.

$V - \frac{wl}{2} - \frac{894}{2} \times 20.667 - 9{,}230$ lb.

$\Sigma_o - \frac{8{,}000\ V}{7ud} - \frac{8{,}000 \times 9.23}{7 \times 120 \times 14.25} - 6.18$ in.

From Table 5, use two 1 $\frac{1}{8}$ in square bars at both the top and bottom of the lintel to give $A - 5.08$ sq. in. and $\Sigma_o = 18$ in.

From Table 6, minimum web width is 8 in. with special anchorage.

Sec. 2619, H.B.C. requires that the two 1 $\frac{1}{8}$ in. top bars be bent down to form a semi-circular hook of minimum radius of 4 bar diameters. The bottom bars will be bent up to the same radius.

Sec. 2618, H.B.C.

$$v = \frac{V}{bjd} = \frac{9,230}{8 \times 0.875 \times 14.25} = 92.5 \text{ psi.}$$

Sec. 2613, H.B.C. gives allowable $v = 0.03 f'_c = 90$ psi. with no web reinforcement but with anchorage of longitudinal reinforcement.

Use 5 U-stirrups made of No. 6 wire and spaced at 2,3,3,3,3 in. as shown in Fig. 22.

Investigation of Window Areas

The next step is an investigation to see if the windows will withstand the force of the wind. Don Graf's Technical Sheets give a formula to examine a glass pane on the pressure side for strength as

$PA = 3.48 \, Mt^2F$ where

M - modulus of rupture which is taken as 6,000 pounds per sq. in. The formula now becomes approximately

$P = \dfrac{21,000 \, t^2F}{AS}$ in which

P = the pressure in pounds per sq. ft.

t = the thickness in inches

A = the area in square feet

F = the factor for ratio of width to height of the pane

S = Safety factor. Safety factor of either 5 to 10 is recommended, depending on glass application. For example, for glass subjected to pressures less than 40 pounds per square inch, a safety factor of 5 is used. When the pressure exceeds 40 psi., a safety factor of 10 arbitrarily is used.

Ratio Width-Height	Factor (F)
10:10 (square)	1.000
9:10	1.005
8:10	1.02
7:10	1.07
6:10	1.14
5:10	1.25
4:10	1.45
3:10	1.8
2:10	2.6
1:10	5.0

The Libbey-Owens-Ford Glass Also makes a tempered plate glass called Tuf-flex which has a modulus of rupture of 30,000 psi.

Master Bedroom Windows

This window is shown in Fig. 13. The center unit is a Thermopane unit 36 in. or 3 ft. wide and 48 $\frac{3}{4}$ in. or 4 ft. high. Thermopane is a factory-built transparent insulating glass unit for windows composed of two or more lights of glass separated by $\frac{1}{4}$ in. or $\frac{1}{2}$ in. of dehydrated air space and hermetically sealed around the edges at the factory with a patented metal-to-glass bond. The bond between the metal seal and the glass will withstand a shearing force greater than 1,000 psi.

From Table 2 with wind from the south there is 41.2 psf.

suction on the north side. If we assume that glazing methods will anchor the glass sufficiently to keep the panes from popping out, we can use this pressure to investigate. The thickness necessary to withstand the pressure is expressed by

$$t = \sqrt{\frac{PAS}{21{,}000\ F}} = \sqrt{\frac{41.2 \times 12 \times 10}{21.000 \times 1.07}} = 0.468 = \frac{15}{32} \text{ in.}$$

The thickest glass made for Thermopane units is $\frac{1}{4}$ in. The $\frac{7}{32}$ in. sheet glass is double strength. With double strength glass the thickness required would be $\frac{15}{64}$ in. This glass can be considered as safe since the factor of safety might have been taken as 5 with the consideration that the pressure was so near the 40 psf. recommended for a safety factor of 5.

The swinging casements on each side of the picture window have lights 16 $\frac{5}{16}$ in. or 1.36 ft. wide and 12 in. or 1 ft. high. A = 1.36 sq. ft.

$\frac{1}{1.35} = \frac{7.4}{10}$ F = 1.07 with width and height dimensions reversed

$$t = \sqrt{\frac{PAS}{21{,}000\ F}} = \sqrt{\frac{41.2 \times 1.36 \times 10}{21.000 \times 1.07}} = 0.157 = \frac{5}{32} \text{ in.}$$

$\frac{3}{16}$ in. sheet glass satisfy the requirements for area and thickness.

The swinging casements are double glazed. The double glazing is essentially an inside storm window that can be replaced by screens in the summer. This is a very desirable feature from the standpoint of reducing heat losses thru the glass areas.

Utility Room Window -

This window has 6 lights 16 $\frac{5}{16}$ in. or 1.36 ft. wide and 1 ft. high as shown in Fig. 13. Again we can use $\frac{3}{16}$ in. thick sheet glass. It is a double glazed horizontal gliding unit.

Laundry and Breakfast Alcove Windows

Fig. 11 shows these double glazed horizontal gliding units. Both windows have 6 lights 27 $\frac{5}{8}$ in. or 2.3 ft. wide and 1 ft. high. Area of 1 light is 2.3 sq. ft.

$\frac{1}{2.3} = \frac{4.34}{10}$ F = 1.45 with width and height dimensions reversed

From Table 2, with wind from the south, the maximum velocity pressure on the east wall is 44.8 psf. suction. Again assuming that the glazing methods will prevent the panes from popping out, the 44.8 psf. can be used as a design factor.

$$t = \sqrt{\frac{PAS}{21{,}000\ F}} = \sqrt{\frac{44.8 \times 2.3 \times 10}{21{,}000 \times 1.45}} = 0.184 = \frac{3}{16} \text{ in.}$$

Use $\frac{3}{16}$ in. thick sheet glass.

Kitchen Window

This double glazed horizontal gliding window is pictured in Fig. 11. There are 4 lights 24 $\frac{5}{8}$ in. or 2.05 ft. wide and 10 in. or 0.833 ft. high. The area of one light is 1.71 sq.ft.

$\frac{0.833}{2.05} = \frac{4.06}{10}$ F = 1.45 with width and height dimensions reversed

$$t = \sqrt{\frac{PAS}{21{,}000\ F}} = \sqrt{\frac{44.8 \times 1.7 \times 10}{21{,}000 \times 1.45}} = 0.1595 = \frac{11}{64} \text{ in.}$$

Use $\frac{3}{16}$ in. thick sheet glass.

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Dining Room Window

The unit as shown in Fig.11 consists of a picture window of Thermopane with swinging casement windows on each side with double glazing. The swinging casement lights are 16¼ in. or 13.5 ft. wide and 1 ft. high.

$$t = \sqrt{\frac{PAS}{21,000\ F}} = \sqrt{\frac{44.8 \times 13.5 \times 10}{21,000 \times 1.07}} = 0.164 = \frac{11}{64} \text{ in.}$$

Use $\frac{3}{16}$ in thick sheet glass

The picture window is 55 in. or 4.56 ft. wide and 60 $\frac{7}{8}$ in. or 5.07 ft. high. Area is 5.07 x 4.56 = 23.1 sq. ft.

$$\frac{4.56}{5.07} = \frac{9}{10} \qquad F = 1.005$$

$$t = \sqrt{\frac{PAS}{21,000\ F}} = \frac{44.8 \times 23.1 \times 10}{21,000 \times 1.005} = 0.686 \text{ in.} = \frac{11}{16} \text{ in.}$$

The thickest glass made for Thermopane units is ¼ in. Neither the double strength sheet glass nor the Tuf-flex heat treated glass are fabricated in this width.

The ¼ in. plate glass will stand

$$P = \frac{21,000\ t^2F}{AS} = \frac{21,000 \times 1.005}{16 \times 23.1 \times 5} = 11.44 \text{ psf.}$$

The wind velocity in miles per hour (V) which is eqivalent to given pressures in pounds per square foot may be found from the formula

$$\text{Pressure} \approx 0.004\ V^2$$

$$V = \sqrt{\frac{P}{0.004}} = \frac{11.44}{0.004} = 53.4 \text{ miles per hour}$$

This window can be expected to rupture. From Table 2, it can be seen that the window will pop out with wind from the south with 44.8 psf. suction and also pop out with wind from the north with 41.4 psf. suction. With wind from the east, the window will rupture inward under a pressure of 25.9 psf.

Living Room Window

This window is shown in Fig. 10. The center unit is $75 \frac{9}{16}$ in. or 6.29 ft. high and $48 \frac{11}{16}$ in. or 4.05 ft. wide. The unit is a Thermopane unit and has an area of 25.5 sq ft.

With the wind from the north, the maximum velocity pressure is 37.8 psf. suction. Assuming that glazing methods are adequate to keep the glass from popping out the 37.8 psf. can be used as a design criterion.

$$\frac{4.05}{6.29} = \frac{6.43}{10} \quad F = 1.14 \text{ with the width and height dimensions reversed}$$

$$t = \sqrt{\frac{PAS}{21,000\ F}} = \sqrt{\frac{37.8 \times 25.5 \times 5}{21,000 \times 1.14}} = 0.458 = \frac{15}{32} \text{ in.}$$

Again the thickest glass made is $\frac{1}{4}$ in. and double strength sheet glass and heat treated Tuf-flex are not fabricated in this width. With $\frac{1}{4}$ in. plate glass, the window will stand

$$P = \frac{21,000\ t^2 F}{AS} = \frac{21,000 \times 1.14}{16 \times 25.5 \times 5} = 11.73$$

The approximate equivalent wind velocity is

$$V = \sqrt{\frac{P}{0.004}} = \frac{11.73}{0.004} = 54.1 \text{ miles per hour}$$

From Table 2, it is apparant that with wind from the east the window would pop out with a velocity pressure of 32.6 psf. suction. With wind from the north, the window would again pop out with a velocity pressure of 37.8 psf. suction. With wind from the south, the window would rupture inward under a pressure of 17.3 psf.

The companion units on each side of the picture window are double glazed horizontal gliding units. These are units having panes 30 $\frac{5}{8}$ in. or 2.55 ft wide and 1 ft. high. Area is 2.55 sq. ft.

$\frac{1}{2.55} = \frac{3.92}{10}$ F = 1.45 with width and height dimensions reversed

$$t = \sqrt{\frac{PAS}{21,000\ F}} = \sqrt{\frac{37.8 \times 2.55 \times 5}{21,000 \times 1.45}} = 0.1255 = \frac{1}{8} \text{ in.}$$

Use $\frac{3}{16}$ in. sheet glass

Gallery Windows

As shown in Fig. 12, panes are 24 $\frac{5}{8}$ in. or 2.05 ft. wide and 10 in. or 0.834 ft. high. These units are double glazed horizontal gliding units. The area of one pane is 1.7 sq. ft. F = 1.45 with width and height dimensions reversed

With wind from the west a pressure of 62.1 psf. is exerted.

$$t = \sqrt{\frac{PAS}{21,000\ F}} = \sqrt{\frac{62.1 \times 1.71 \times 10}{21,000 \times 1.45}} = 0.188 = \frac{3}{16} \text{ in.}$$

Use $\frac{3}{16}$ in. sheet glass

All small panes will withstand a 125 mph. wind. Picture windows on the south and west sides will rupture and have an equal chance of popping out. These windows should be insured with windstorm insurance.

Chimney Footing

The footing will have its base five feet below the floor slab to duplicate the worst conditions that might be encountered such as soft soil bearing areas or a slope that would require a fill to the level of the floor slab. Weights of the concrete blocks will average approximately 40 pounds with mortar joints where concrete blocks are used below grade and cinder blocks above grade.

Fireplace dimensions approximate 32 in. by 56 in. The footing is 8 in. thick and 8 in. wider than the width and length of the fireplace making its dimensions 40 in. x 64 in. Weight of the footing is

$$\frac{40 \times 64 \times 8}{1,728} \times 150 = 1,545 \text{ lb.}$$

Above the footing are 4 tiers with 9 blocks around the circumference of each tier. The weight of these blocks are

$$40 \times 9 \times 4 = 1,440 \text{ lb.}$$

On these blocks is a 4 in. concrete slab which is the floor of the ash pit. This slab weight is

$$\frac{32 \times 56 \times 4}{1,728} \times 150 = 622 \text{ lb.}$$

Above the slab are 2 tiers of blocks with 9 blocks around the circumference of each tier. These blocks form the ash pit. One block will be left out of the outside wall to remove the ashes. Weight of these blocks is

$$17 \times 40 = 680 \text{ pounds}$$

Assume that the weight of one half the floor span across the width of the fireplace is carried down to the footing. Then this weight is

$$\frac{6 \times 56 \times 9.5}{144} \times 150 = 3,320 \text{ lb.}$$

The top of the fireplace mantel is 6 blocks high. There are 5½ blocks on the back side and the ends in each tier. They weigh

$$5.5 \times 6 \times 40 = 1{,}320 \text{ lb.}$$

The smoke shelf weighs approximately

$$\left(\frac{8 \times 8 \times 40}{1{,}728} + \frac{8 \times 16 \times 40}{2 \times 1{,}728}\right)(150) = 444 \text{ lb.}$$

The precast concrete mantel and concrete forming the smoke chamber weigh

$$\left(\frac{8 \times 8 \times 56}{2 \times 1{,}728} + \frac{8 \times 4 \times 56}{1{,}728}\right)(150) = 360 \text{ lb.}$$

There are ten more tiers to the top of the chimney. There are two 8 x 8 in. flue linings. One tier consists of 9 masonry units and 8 concrete bricks with 4 hollow units 4 in. thick. The bricks weigh 5.5 lb. The weights of these units are

$$10 \times 11 \times 40 + 10 \times 5.5 \times 8 = 4{,}840 \text{ lb.}$$

For details of fireplace and chimney construction refer to Fig. 38, Facts About Concrete Masonry by the National Concrete Masonry Association.

The total weight bearing on the soil is

$$1{,}545 + 1{,}440 + 622 + 3{,}320 + 444 + 360 + 4{,}840$$

$$= 12{,}571 \text{ lb.}$$

Sec. 2802, H.B.C. gives the allowable bearing capacity of soft clay, sandy loam or silt as 1 ton per sq. ft. This section also says that the footing shall be designed so that the allowable bearing capacity specified for the particular soil shall not be exceeded. Pressure exerted on the soil is

$$\frac{12{,}571}{3.33 \times 5.55} = 708 \text{ lb. per sq. ft.}$$

The allowable soil pressure is not exceeded.

Sec. 2622, H.B.C. gives the allowable compressive unit stresses in footings as $0.25\ f_c'$ without reinforcement.

This is 0.25 x 3,000 = 750 psi. Pressure exerted on the footing with gross area of each block equal to 128 sq. in. is

$$\frac{11,026}{9 \times 128} = 9.6 \text{ psi.}$$

This does not exceed the allowable.

Wall Footing

The heaviest load that could be brought on the footing would be the case where the core areas of all blocks above the floor slab were filled with concrete. Roof slab has a dead weight of 70.3 psf. and a span of 21.58 ft. including the overhang. The floor slab has a span of 19 ft. and a dead load of 75 psf. Load along a 16 in. length of wall is

$$1.33 \times 70.3 \times \frac{21.58}{2} + \frac{8 \times 16 \times 7.5}{144} \times 150 + 75 \times 1.33 \times \frac{19}{2}$$

$$+ 6 \times 32 = 3,152 \text{ lb.}$$

The footing extends out 4 in. on each side of the wall and is 8 in. thick. Its weight per 16 in. length is

$$\frac{16 \times 16 \times 8}{1,728} \times 150 = 178 \text{ lb.}$$

The compression in the footing is

$$\frac{3,152}{128} = 24.6 \text{ psi.}$$

Sec. 2622, H.B.C. gives the allowable compressive unit stress as 0.25 f_c' without reinforcement. This is 0.25 x 3,000 = 750 psi.

Pressure exerted on the soil is

$$\frac{(3,152 + 178)}{16 \times 16} \times 144 = 1,875 \text{ psf.}$$

Sec. 2802, H.B.C. gives allowable bearing capacity of soft clay, sandy loam or silt as 1 ton per sq. ft. This allowable bearing capacity is not exceeded.

Forms for Roof Slab

Forms for the roof slab will be designed as suggested on pages 50 to 60, Reinforced Concrete Design. For soffit forms use 1 x 8 in. square edge timber. Finished size is $\frac{3}{4} \times \frac{7}{8}$ in.

Live load of pouring crew	75 psf.
Wt. of concrete slab	70 psf.
Formwork, etc.	155 psf.

$f_w = \frac{Mc}{I}$ and $\frac{I}{c} = S$

$M = f_w S$ where S for a rectangle $= \frac{\frac{1}{12} bh^3}{\frac{h}{2}} = \frac{bh^2}{6}$

$M = f_w \frac{bh}{6}$

f_w is commonly taken as 1,800 psi. and

$M = \frac{wl^2}{12} = f_w \frac{bd}{6}$

As the stringers will probably require closer spacing than the sheathing, check their capacity first. Try 4 x 10 in. stringers and compute the maximum spacing from $wsl^2 \frac{12}{8} = f_w S$ with a 19 ft. span.

$$155s \times 19 \times 19 \times 1.5 = 1,800 \times 56.41$$

$$s = \frac{1,800 \times 56.41}{155 \times 361 \times 1.5} = 1.21 \text{ ft. or } 14.5 \text{ in.}$$

The flexual stress of the decking, using M - wl for full continuity is

$$155 \times 1.21^2 = 12 \times \frac{3}{4} \times \frac{3}{4} \times \frac{1}{6} \times f_w$$

$$f_w = \frac{155 \times 1.46 \times 96}{12 \times 9} = 202 \text{ psi.}$$

-

This is within the allowable so a 14 in. spacing will be used.

The decking can be left with spaces between the boards since corkboard is placed on the decking before the concrete is poured. The ceiling forms are constructed 1 in. deeper than the thickness of the concrete slab. Before placing the reinforcing steel and the pouring of the concrete, one course of corkboard 1 in. thick shall be laid down in the forms. All transverse joints shall be broken and all joints made tight. Galvanized wire nails shall be driven obliquely into the corkboard-two to the square foot-the heads to be left protruding approximately $1\frac{1}{2}$ in. The reinforcing steel shall then be put in and the concrete poured.

With 4 x 4 in. shoring spaced 6 ft. apart, the load on a shore with the slab weighing 70 psf. and forms, etc. weighing 25 psf., is

$$95 \times 6 \times \frac{14}{12} = 665 \text{ lb.}$$

Cross sectional area of the shoring is 13.14 sq. in. and the shoring is stressed to

$$\frac{665}{13.14} = 51 \text{ psi.}$$

Allowable stress is

$$1{,}200\left(1-\frac{L}{80\ D}\right) = 1{,}200\left(1-\frac{76}{80 \times 4}\right) = 916 \text{ psi.}$$

All lumber must be graded No. 2 or better to use these stresses.

Shoring has 4 x 4 in. tee-head braced with 1 x 6 in. boards. In erecting provide a sill piece under the shores to distribute the weight over the slab. 1 x 4 in. ribbon bracing is placed high enough above the floor to permit walking below.

Heat Transmission Losses

Heat transmission of the various materials will be given as a matter of comparative efficiency as an insulating material.

Heating and ventilating engineers measure heat losses in terms of British thermal units (B.t.u.)-units used to measure heat in much the same manner as we use pounds to measure weight. Standard wood frame construction with $\frac{3}{4}$ in. plaster on metal lath has a heat loss of 0.26 B.t.u. per sq. ft. per hour per degree F° difference in temperature. This figure, 0.26 B.t.u. loss, is considered by heating and ventilating engineers as not excessive as far as efficiency is concerned. In fact, it is quite generally accepted as the basing point on which efficiency or inefficiency of heat losses may be measured.

The University of Minnesota in tests conducted , found that filling the cores of a standard 3-oval core 8 x 8 x 16 in. unit with regranulated cork, rock wool, or similar granular or loose fill material with equivalent insulation values would reduce the heat losses through the plain concrete masonry wall substantially 50 per cent. The coefficient of heat transmission of U of a plain wall, either with or without plaster and with loose or granular fill in cores is 0.20 B.t.u. per sq. ft. per hour per degree F° difference in temperature. Two coats of portland cement paint reduce the value of U on plain concrete masonry walls 0.03 B.t.u. Without cores filled U is 0.40 B.t.u.

While a certain amount of wall insulation is desirable in all types of modern buildings, in general the importance of additional wall insulation in reducing fuel costs has been over-emphasized. The increasing use of larger window areas in buildings of all types is placing greater emphasis on window insulation than on wall insulation. If the window area of a building is increased only 3 per cent, the relative importance of wall insulation in reducing fuel costs is reduced 6 per cent. This is true because ordinarily a greater amount of heat is lost through the windows than through the wall of each building.

The coefficient U for double Thermopane is 0.57 and U for double glazing is 0.61 As a comparison a single glass pane has a U of 1.07 B.t.u.

Heat losses of a 27 x 34 ft. uninsulated 1½ story house built with light-weight concrete masonry exterior walls, furred metal lath and plastered interior finish, with 25 per cent of the wall area in windows and doors, will be approximately

35 per cent through windows and doors
30 " " " the roof
20 " " " the walls
15 " " around all wall openings

Heating engineers generally agree that little is to be gained by reducing the heat loss beyond 0.15 B.t.u. In other words, the cost of insulation to provide a wall with a heat loss below this point may be out of proportion to the fuel saved. To secure this condition in 8 in. concrete masonry or exterior frame walls requires the addition of 1 in. of rigid insulation or its equivalent. This amount of insulation in the above typical house would reduce the actual fuel bill only 8 per cent.

Providing storm windows and doors for this house would reduce the fuel cost about 18 per cent and providing insulation in the roof equivalent to $3\frac{1}{2}$ in. of rock wool or similar material would reduce the heating cost 15 per cent.

The floor slab is poured on a layer of Foamglas 4 in. thick which has a thermal coefficient of 0.40 B.t.u. at 50 F. Foamglas is glass made in cellular form and cut into accurately sized blocks. Each cell is closed and impervious to air or water. Foamglas is permanent and fireproof and has a crushing strength of 150 psi.

The roof slab with 1 in. corkboard and standard roofing has a heat transmission coefficient U of 0.20.

The refrigerated cold storage room will require 4 inches of corkboard to keep the room at the recommended 30 F.

FINIS

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