AN INVESTIGATION OF THE STRUCTURAL DESIGN OF THE MICHIGAN STATE COLLEGE BRICK APARTMENT BUILDINGS

Thesis for the Degree of B. S.

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THESIS

An Investigation of the Structural Design of
the Michigan State College
Brick Apartment Buildings

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INTRODUCTION

The purpose of this thesis is to check the structural design of the Brick Apartment Buildings on the Michigan State College Campus.

From the standpoint of structural design, the different Brick Apartment Buildings do not vary from each other. The main difference in the various buildings is their size and location of basements. With this in mind, the structural design of one of the buildings was completely analysed and taken as representative of the whole group.

These Brick Apartment Buildings have been constructed and are now being occupied by the faculty and students of Michigan State College. They were built on property owned by Michigan State College, south of Shaw Lane and east of Harrison Road, east of the Michigan State Police (East Lansing Post), and just off the Michigan State College Campus at East Lansing, Michigan.

These apartment buildings are of reinforced concrete footings and concrete block foundation walls. The frame is composed of formed strip steel framing members with a self formed nailing strip to facilitate attachment of exterior sheathing and interior base for wall and ceiling finish. Slabs are 2½" thick concrete on #26 guage corrugated iron sheets supported on the steel joists. The roof is textured asbestos shingles on wood roof boards supported on steel rafters.

The exterior is face brick with cut stone sills. Sash and entrances are wood. The interior partitions are steel studs with gypsum lath and plaster to make an overall thickness of 6", for all bearing partitions and dividing partitions between apartments, and 4" overall finished thickness for all others. Floors are asphalt tile, terrazzo, cement or other finishes as indicated.

In examining the structural design of these buildings, several text books and pamphlets were consulted and the methods and procedures as taught in the courses at Michigan State College, were followed. Hence, more time may have been spent on this investigation than would have been in ordinary practice. In many cases, however, the computed design was checked by following the methods an actual designer would employ, using handbooks, graphs and tables.

The specifications and plans used in this investigation were those actually used in the construction of these buildings. Whenever there was a question of interpretation of the plans or of actual construction procedure the firm of O. J. Munson was consulted.

Investigation of Design of Roof Truss

Weight of roof covering:

Wooden sheathing 1" thick — 3 lbs./sq. ft.

Asbestos shingles and felt — 4 lbs./sq. ft.

Snow load:

Minimum value of 25 lbs./sq. ft. for all slopes up to 20°. Load may be reduced 1 lb. for each degree of slope above 20°.

Hence:

Slope 2 to 1 or 26.5°

Use: — 20 lbs./sq. ft.

Wind pressure: (by Duchemin)

$$P_n = P_h \left(\frac{2 \sin A}{1 + \sin^2 A}\right)$$

P_h = the horizontal pressure per sq. ft. on a vertical surface

P_n = the normal pressure per sq. ft. of sloping surface

A = the angle of inclination of the sloping surface

$$P_n = 30 \left(\frac{2 \sin 26.5}{1 + \sin^2 26.5^\circ}\right)$$

$$= 30 \left(\frac{2(.446)}{1 + (.446)^2}\right)$$

$$= 30 \left(\frac{.892}{1.199}\right)$$

P_n = 22.35 lbs./sq. ft. — 22.35 lbs./sq. ft.

Weight of truss:

$$w = \frac{pl}{150 + 5l + \frac{ps}{3}}$$

w = weight of truss in lbs./sq. ft. of horizontal covered surface.

l = span of truss in ft.

s = the distance center to center of trusses in ft.

p = load in lbs./sq. ft.

Combinations of loads:

(1) Dead and snow load
$20 + 7 = 27$ lbs./sq. ft.

(2) Dead and wind load
$7 + 22.4 = 29.4$ lbs./sq. ft.

(3) Dead, one-half snow, and wind loads
$7 + 10 + 22.4 = 39.4$ lbs./sq. ft.

$$w = \frac{pl}{150 + 5l + \frac{ps}{3}} = \frac{(40)(38)}{150 + (5)(38) + \frac{(40)2}{3}}$$

w = 7.42 lbs./sq. ft. of horizontal covered surface.

Length and areas of panel:

Length of upper chord = 21.1 ft.

Length of panel = 10.55 ft.

Total load per panel:

Asbestos shingle	$(3.7)(2)(10.55)$	= 77.8 lbs.
Roof felt	$(0.3)(2)(10.55)$	= 6.3 lbs.
Wood sheathing	$(3)(2)(10.55)$	= 63.0 lbs.
Truss, per panel	$(7.42)(2)(38)(\frac{1}{4})$	= 44.0 lbs.
Dead panel load		288.1 lbs.
Snow panel load	$(20)(2)(10.55)$	= 420 lbs.
Wind panel load	$(22.3)(2)(10.55)$	= 469 lbs.
Ceiling load	$(\frac{2.5}{12})(38)(2)(150)$	= 2375 lbs.

Determination of stresses:

$2R = 5100$
$R = 2550$ lbs.

(see drawings for sketch of roof truss and joint and member numbers).

Joint (1):

$$1\text{-}2 \sin 26.6^\circ + 205 + 235 \cos 26.6^\circ = 2550$$
$$1\text{-}2\,(.447) + 205 + 235\,(.894) = 2550$$
$$1\text{-}2\,(.447) = 2135$$
Member 1-2 = 4770 lbs.

$$1\text{-}4 = 4770 (\cos 26.6) - 235 (\sin 26.6)$$
$$= 4260 - 105$$
Member 1-4 = 4155 lbs. tension

Joint (4):

$$\frac{1187}{\text{Member } 2\text{-}4} = \cos 26.4^\circ$$
$$\text{Member } 2\text{-}4 = \frac{1187}{.894}$$
Member 2-4 = 1400 lbs. tension

$$4\text{-}5 = 4155 + 1400 (\sin 26.6)$$
$$= 4155 + 625$$
Member 4-5 = 4780 lbs. tension

Joint (2):

$$2\text{-}3 = 4770 - 410\ (.447)$$
$$2\text{-}3 = 4770 - 184$$
$$\text{Member } 2\text{-}3 = 4586 \text{ lbs.}$$

Design of tension members:

Maximum stress = 4780 lbs.

Using allowable unit stress of 18,000 lbs./sq. in.

Net area required:

$$\frac{4780}{18,000} = 0.266 \text{ sq. in.}$$

8" Joists used:
area of section = 2.90 O.K.

6" Joists used for member 2-4
area of section = 2.50 O.K.

The 8" and 6" joists used are both large enough to support the required load, hence design is O.K.

Design of upper chord:

Since direct compression in the end panel 1-2 is greater than the other panel of the top chord, the top chord section will be fixed by the stresses in this panel 1-2.

At center total direct compression is equal to maximum compressive strength in member = 4800 lbs.

Allowable compressive unit stress:

$$f = \frac{18000}{1 + \frac{l^2}{18000 r^2}}$$

$$l = 126"$$

$$r = 2.43$$

$$f_1 = \frac{18000}{1 + \frac{(126)^2}{18000 \times (2.43)^2}}$$

$$f_1 = 15,700 \text{ lbs.}$$

Moment:

Where a member is continuous over panel points, twenty-five percent reduction in the bending moment is allowed for simple beams.

Weight per running foot on a panel:

$\frac{1177}{10.55} = 111.5$ lbs./ running foot.

Moment at support:

$M = 1177 \times \frac{10.55}{2} \times 12 \times 3/4 = 56{,}000$ in-lb.

$$A = \frac{N}{f_1} + \frac{Me}{f_2 r^2}$$

A = effective area of the cross-section.

N = total direct stress.

f_1 = unit direct stress.

f_2 = critical extreme fiber stress.

M = bending moment.

e = distance from the neutral axis to the extreme fiber.

r = radius of gyration.

$$A = \frac{4800}{15700} + \frac{(56000)(2.94)}{18000\ (2.43)^2}$$

A = 1.86 sq. inches.

Area furnished by a 6" joist = 2.50 sq. in., hence design is O.K.

All joints are to be welded:
(From Lansings Building and Safety Code - Amended 2-16-48)

All welding shall be done by skilled workmen and the Commissioner may refuse to accept work done by any workman who cannot give satisfactory proof of his skill and ability in welding. All welding metal shall be in shear as far as possible. The shearing stress shall not exceed five thousand (5,000) pounds per square inch based on the minimum section of welding metal and welding metal in tension shall not be stressed beyond six thousand (6,000) pounds per square inch. All working shall be done in a neat, accurate and workmanlike fashion. Connections showing unsound metal or important defects shall be removed and replaced.

Before approving any type of welded connection, the Commissioner may require one or more full sized specimens to be constructed and tested to destruction. The designed load to be allowed on such connection shall not be more than one-sixth of the minimum test load resisted.

Investigation of Room Slab Design

Specifications:

f_c = 800 lbs./sq. in.

f_s = 16,000 lbs./sq. in.

V = 40 lbs./sq. in.

M = 1/12 Wl^2

U = 100 lbs./sq. in.

n = 15

Loads:
(From Lansings Building and Safety Code - Amended 2-16-48)

Minimum live load for an Apartment is 40 p.s.f.

Live load	=	50 p.s.f.
Asphalt tile and asphalt base	=	5 p.s.f.
2" slab $(150)(\frac{2}{12})(1)(1)$	=	25 p.s.f.
W	=	80 p.s.f.

$$f_c = 800$$
$$\frac{f_s}{n} = \frac{16000}{15} = 1067$$
$$\frac{x}{d} = \frac{800}{1067 + 800}$$
$$x = .429\ d$$
$$jd = d - \frac{x}{3} = d - \frac{.429}{3}d$$
$$jd = d - .143d = .857d$$

Clear span = 1 foot 10 inches

Moment:

$$M_{max} = \frac{1}{12}wl^2$$
$$= \frac{1}{12}(80)(1.83)^2(12)$$
$$= 267 \text{ in.-lbs.}$$

Depth (d)

$$M = Cxjd$$
$$= (\frac{800}{2})(12)(.857d)(.429d)$$
$$d^2 = \frac{267}{(400)(12)(.429)(.857)}$$
$$d = .39 \text{ inches} \qquad \text{O.K.}$$

A 2" slab is used, hence, O.K.

End shear:

$$V = \frac{(\text{unit load})(\text{span})}{2}$$
$$= \frac{(80)(2)}{2}$$
$$= 80 \text{ lbs./ft. wide strip of slab.}$$

Unit shear:

$$v = \frac{V}{bjd} = \frac{80}{12(.857)2} = 3.9 \text{ lbs/sq. in.}$$

Well within allowable 40 lbs./sq. in.

For first and second floor slabs, concrete with no reinforcement is used on No. 26 guage corrugated sheet iron forming on steel joists.

Investigation of Corridor Slab Design

Loads

Live load	50 lbs./sq. ft.
Terrazzo $1\frac{1}{2}"$	19 lbs./sq. ft.
$2\frac{1}{2}"$ slab	31 lbs./sq. ft.
	100 lbs./sq. ft.

Clear span = 22 inches.

Moment:

$$M = \frac{1}{12}wl^2$$

$$= \frac{100}{12}(1.83)^2(12)$$

$$= 334 \text{ in.-lbs.}$$

Depth (d)

$$M = Cxjd$$

$$334 = \left(\frac{800}{2}\right)(12)(.429d)(.857d)$$

$$d^2 = \frac{334}{(400)(12)(.429)(.857)}$$

$$d = .434 \text{ inches}$$

We have 2.5 inches, hence O.K.

End shear:

$$V = \frac{(100)(2)}{2}$$

$$= 100 \text{ lbs./ft. wide of slab.}$$

$$v = \frac{V}{bjd}$$

$$= \frac{100}{(12)(.857)(2.5)}$$

$$= 3.89 \text{ lbs./sq. in.}$$

Well within allowable 40 lbs./sq. in.

The thickness of $2\frac{1}{2}"$ for the corridor slabs is found to be safe.

Investigation of Basement Slab Design

Loads

Live load	50 lbs./sq. ft.
4" slab	50 lbs./sq. ft.
	100 lbs./sq. ft.

Clear Span = 14 ft. 8 in.

Moment:

$$M = 1/12\ wl^2$$
$$= 1/12\ (100)(14.67)^2(12)$$
$$M = 21{,}500 \text{ in.-lbs.}$$

Depth (d)

$$M = Cxjd$$
$$21{,}500 = \left(\frac{800}{2}\right)(12)(.429d)(.857d)$$
$$21{,}500 = 1768d^2$$
$$d^2 = 12.15$$
$$d = 3.48 \text{ inches}$$

4 inches is O.K.

Steel area:

$$A_s = \frac{M}{f_s jd}$$
$$= \frac{21{,}500}{(16000)(.857)(4)}$$
$$= \frac{21{,}500}{54{,}800}$$
$$A_s = 0.392 \text{ sq. in/ft. width required.}$$

6"x6" No. 10 wire guage:

Thickness of one wire = .130 inches

In one sq. ft. 4 x .130 = .520 sq. inches O.K.

Hence, the 4" concrete slab with 6"x6" No. 10 mesh is structurally safe.

Investigation of the Design of Beams

The complete design of a steel beam may include the consideration of a number of items such as deflection, shear, flexure, etc. However, the basic principle underlying the design of a beam is that the beam cross section shall have a resisting moment equal to or greater than the bending moment. Thus:

$$M = fS$$
$$\text{or } S = \frac{M}{f}$$

in which M is the bending moment and the quantity fS is the resisting moment.

Here are the steps that will be followed in the design of a beam:

1. Compute the loads the beam will be required to support.
2. Compute M, the maximum bending moment in inch-lbs.
3. Find S by $S = \frac{M}{f}$, the flexure formula.
4. Refer to the tables giving the properties of beams and select a beam having a section modulus equal to or greater than the required section modulus found in step 3.
5. Investigate the beam for shear and deflection.

Corridor Joists:

Loads:

Joists spaced every 2'-0".

w = 100 lbs./sq. ft. for corridor.

(2)(100) = 200 lbs./ running foot evenly distributed.

Maximum Bending **Moment** (Span 5.67 ft.)

$$M = \frac{Wl^2}{8}$$

$$= \frac{(200)(5.67)(5.67)}{8} = 800 \text{ ft.-lbs.}$$

or 9600 in.-lbs.

Section modulus:

$$S = \frac{M}{f} = \frac{9600}{16000}$$

$$S = .60 \text{ in.}^3$$

2"x8" Joists have $S = 4.6 \text{ in.}^3$ O.K.

The weight of the beam was ignored in computing loads but a beam having a larger section modulus was chosen, hence there is additional strength to provide for the weight of the beam.

Weight of 2"x8" joists.

(6.5 lbs./ft.)(5.67) = 36.8 lbs.

Total weight = 36.8 ≠ 1130
" " = 1169 lbs.

Shear:

$$V = \frac{1169}{2} = 585 \text{ lbs.}$$

$$R_1 = R_2$$

Depth of section = 8 inches

Thickness (t) = 0.13 inch

To find the actual shearing unit stress:

$$f = \frac{V}{dt} = \frac{585}{(8)(.13)} = 562.5 \text{ lbs/sq. in.}$$

The shearing unit stress is found to be 563 lbs./sq. in. and as the allowable stress is 13,000 lbs./sq. in; the beam is acceptable for shear.

Deflection of Beam:

The allowable deflection is limited to 1/360 of the span.

$$D = \frac{(5.67)(12)}{360} = .189 \text{ inches}$$

For uniformly distributed load simple supports, the actual deflection is:

$$D = \frac{5Wl^3}{384EI}$$

$$= \frac{5}{384}\ \frac{(1169)(68)^3}{(30{,}000{,}000)(18.7)} = .0085 \text{ inches}$$

$$= .0085 \text{ inches actual deflection}$$

As the actual deflection is less than the allowable .189 inches, the deflection is not excessive.

Room Joists:

Loads:

Joists are also spaced 2'-0" apart.

w = 80 lbs/sq. ft. for the rooms.

2x80 = 160 lbs./ running ft. evenly distributed.

Maximum Bending Moment: (Span 15'-0")

For simply supported beam, uniformly distributed load.

$$M = \frac{Wl^2}{8}$$

$$= \frac{(160)(15)(15)}{8}$$

$$= 4500 \text{ ft.-lbs.}$$

$$= 54{,}000 \text{ in.-lbs.}$$

Section Modulus:

$$S = \frac{M}{f}$$

$$= \frac{54,000}{16,000}$$

$$= 3.38 \text{ in.}^3$$

2"x9" joists were used and hence from the tables
$S = 5.8 \text{ in.}^3$

Total weight:

Weight of 2"x9" joists = (7.5)(15)	=	112.5	lbs.
w = (160)(15)	=	2400	lbs.
		2513	lbs.

Shear:

$R_1 = R_2$ and therefore
$$V = \frac{2513}{2} = 1257 \text{ lbs.}$$

Depth of section (d) = 9.

Thickness, t = 0.145

To find the actual shearing unit stress:

$$f_s = \frac{V}{dt}$$

$$f_s = \frac{1257}{(9)(145)}$$

$$f_s = 963 \text{ lbs./sq. in.}$$

As the allowable stress is 13,000 lbs./sq. in., the beam is acceptable for shear.

Deflection:

The allowable deflection is limited to 1/360 of the span.

$$D = \frac{(15)(12)}{360} = 0.50 \text{ inches}$$

For uniformly distributed load simple supports, the actual deflection is:

$$D = \frac{5Wl^3}{384EI}$$

$$= \frac{5(2513)(180)^3}{384(30,000,000)(26.2)}$$

$$\doteq 0.33 \text{ inches}$$

The actual deflection is less than the allowable, hence the deflection is not excessive.

Design of Girder

Maximum bending moment:

Maximum moments for the concentrated loads and uniformly distributed load will be found separately.

$R_1 = R_2 = 5526$ lbs.

$$M_{max} = (5526)(5) - \left[(1842)(5) + (1842)(3) + (1842)(1)\right]$$
$$= 11{,}052 \text{ ft.-lbs. or } 132{,}624 \text{ in.-lbs.}$$

Section modulus:

$$S = \frac{M}{S} = \frac{132{,}624}{16{,}000} = 8.3 \text{ in.}^3$$

Beam weight:

Maximum moment due to beam weight.

$$M = \frac{wl^2}{8} = \frac{17(10)^2(12)}{8} = 2550 \text{ in.-lbs.}$$

$$S = \frac{M}{f} = \frac{2550}{16{,}000} = 0.1595 \text{ in.}^3$$

Section modulus required for concentrated load and the uniformly distributed load is $8.3 + .16 = 8.46$ in.^{3}. The 8"WF17# has a section modulus of 14.1 in.^{3}, hence is acceptable.

Design of Stairways

Stairs are designed as simply supported slabs with a span equal to the horizontal distance between the floor beam and the landing beam. In computing the dead weight per horizontal foot for use in design the inclined part should be considered to have the total vertical thickness.

Loads:

Live load	=	80 lbs./sq. ft.
9" slab 9/12 x 150	=	113 lbs./sq. ft.
		193 lbs./sq. ft.

Span: (8 ft.)

$$M = \frac{wl^2}{12}$$

$$= \frac{(193)(8)^2(12)}{(12)}$$

$$= 12,350$$

Depth (d):

$$M = Cxjd$$

$$12,350 = \left(\frac{800}{2}\right)(12)(.429d)(d)$$

$$d^2 = \frac{12,350}{1765}$$

$$d^2 = 7$$

$$d = 2.65"$$

There is an effective depth of 6", hence O.K.

End shear:

$$V = \frac{(\text{unit load})(\text{span})}{2}$$

$$V = \frac{(193)(8)}{2}$$

$$V = 772$$

$$v = \frac{V}{bjd}$$

$$= \frac{772}{(12)(.857)(6)}$$

$$v = 12.5 \text{ lbs./sq. in.}$$

Within allowable 40 lbs./sq. in.

Steel:

$$A_s = \frac{M}{f_s jd}$$

$$= \frac{12,350}{(16,000)(.857)(6)}$$

$$= 0.150 \text{ sq. in./sq. ft.}$$

$\frac{1}{2}$"ϕ @ 6" o.c. have a steel area of .40 sq. in./sq. ft., hence O.K.

Bond check:

$$U = \frac{N}{\Sigma o jd}$$

$$= \frac{772}{(3.1)(.857)(6)}$$

$$= 48.5 \text{ lbs./sq. in.}$$

O.K., within 100 lbs./sq. in. allowable.

As to further check the table on page 83 of the "Simplified Design of Concrete Floor Systems" by the Portland Cement Association was referred to:

With a span of 8'-0" the thickness should be 5 inches and the reinforcing steel should be $\frac{1}{2}$"ϕ-7$\frac{1}{2}$" o.c.

This design has an effective depth of 6 inches and reinforcing steel of $\frac{1}{2}$"ϕ-6" o.c., hence O.K.

Investigation of Column Design

In the design of the steel columns the following allowable unit stresses obtained from the Lansing Building and Safety Code - amended 2-16-48, were used:

With values of $\frac{l}{r}$ not greater than 120:

$$f = 17{,}000 - 0.485 \frac{l^2}{r^2}$$

With values of $\frac{l}{r}$ greater than 120:

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

In which l = the unbraced length of the column, in inches, and r = the least radius of gyration of the section, in inches.

Attic columns:

Load:

Total load per panel	1177 lbs.
Weight of 8"WF 17# ridge	17 lbs.
	1194 lbs.

Computation of allowable axial load:

For 4"x4" H @ 7.5#
Unbraced length of 7 ft.
Slenderness ratio:

$$\frac{l}{r} = \frac{(7)(12)}{.94} = 89.5$$

It is seen that $\frac{l}{r}$ does not exceed 120 and therefore the allowable unit stress is found by the equation:

$$f = 17{,}000 - 0.485\left(\frac{l}{r}\right)^2$$

$$f = 17{,}000 - 0.485\,(89.5)^2$$

$$f = 17{,}000 - 3055$$

$$f = 13{,}945 \text{ lbs./sq. in.}$$

Area of section = 2.22 sq. in.

$(f)(A) = P =$ allowable axial load.

$(13{,}945)(2.22) = 30{,}950$ lbs. O.K.

Steel studs (supporting roof truss):

Load or steel studs = 2550 lbs.

Computation of allowable axial load:

Unbraced length of 9 inches.

$$r = \sqrt{\frac{I}{A}}$$

$$I = \frac{bd^3}{12}$$

$$= \frac{(3.63)(3.63)^3}{12}$$

$$I = 14.4 \text{ in.}^4$$

$$r = \sqrt{\frac{14.4}{(3.63)^2}}$$

$$r = 1.045 \text{ inches}$$

$$\frac{l}{r} = \frac{9}{1.045}$$

$$\frac{l}{r} = 8.6$$

The slenderness ratio is less than 120, hence:

$$f = 17{,}000 - 0.485\ \left(\frac{l}{r}\right)^2$$

$$= 17{,}000 - 0.485\ (8.6)^2$$

$$f = 16{,}964 \text{ lbs./sq. in.}$$

$$P = f(A)$$

$$= (16{,}964)(3.63)^2$$

$= 224{,}000$ lbs. Allowable axial load. O.K.

Investigation of Loads on steel studs adjacent to door opening (maximum stress condition)

Weight of Brick masonry:

45 lbs./cu. ft.
$(45)(4.67)(8)(4/12) = 560$ lbs.

Weight of plaster $= 200$ lbs.

Weight of a 3-5/8" stud per ft.

$$(490)\frac{(3.63)^2}{144}(1) = 45 \text{ lbs.}$$

Reactions:

$$R_1 = 2550 + (4.67)(45)$$
$$R_1 = 2760 \text{ lbs.} = R_2$$

$$R_3 = (2760) + (2500) + (500) + (210)$$
$$R_3 = 5970 \text{ lbs.} = R_4$$

Computation of allowable axial load:

$$r = \sqrt{\frac{I}{A}}$$

$$I = \frac{bd^3}{12}$$

$$= \frac{(3.63)(3.63)^2}{12}$$

$$I = 14.4 \text{ in.}^4$$

$$r = \sqrt{\frac{14.4}{(3.63)^2}}$$

$r = 1.045$ inches

Maximum unbraced length of 8 ft. 5 inches.

$$\frac{l}{r} = \frac{8.42(12)}{1.045}$$

$$\frac{l}{r} = 96.7$$

$\frac{l}{r}$ less than 120, therefore:

$$f = 17{,}000 - 0.485\ (\tfrac{l}{r})^2$$
$$f = 17{,}000 - 0.485\ (96.7)^2$$
$$f = 17{,}000 - 4540$$
$$f = 12{,}460 \text{ lbs./sq. inch.}$$

$$P = fxA$$
$$P = (12{,}460)(3.63)^2$$
$P = 164{,}000$ lbs. allowable axial load.

O.K. to use 3-5/8" steel studs.

Investigation of design of steel studs (ordinary case).

All exterior walls and corridor partitions have 3-5/8" steel studs spaced at 24 inches.

Loads:

From roof and roof truss	2550 lbs.
Brick masonry (45)(2)(16)(4/12)	480 lbs.
Plaster (5)(2)(16)	160 lbs.
Floor and joist reaction	1300 lbs.
Weight of stud (45)(18)	811 lbs.
	5301 lbs.

Slenderness Ratio:

$$\frac{l}{r} = \frac{(8.42)(12)}{1.045}$$

$\frac{l}{r}$ = 96.7 less than 120, hence:

$$f = 17{,}000 - 0.485\left(\frac{l}{r}\right)^2$$
$$f = 12{,}460 \text{ lbs./sq. in.}$$

$$P = fxA$$
$$= (12{,}469)(3.63)^2$$
P = 164,000 lbs. allowable axial load.

3-5/8" steel studs are O.K.

Investigation of Design of Steel studs in cross partitions (2-5/16" steel studs spaced @ 24 inches).

Loads:

Weight of 2-5/16" steel stud	405 lbs.
Weight of 24" of steel joist	20 lbs.
Reaction from floor and joist	1300 lbs.
	1725 lbs.

$$\frac{l}{r} = 96.7$$

$$f = 12{,}460 \text{ lbs./sq. in.}$$

$$P = (f)(A)$$
$$= (12{,}460)(2{,}312)^2$$
P = 66,000 lbs.

The allowable is more than the actual load, hence steel stud is O.K.

Investigation of Design of Columns in Unexcavated Part (Interior Column)

Loads:

From roof truss and roof	1500 lbs.
Steel stud (45)(8.45)(2)(3)	2280 lbs.
Wall Partitions (14)(10.17)(9)(2)	2560 lbs.
From Second floor joists (1842)(3)	5526 lbs.
Weight of 8" WF 17# = 173 lbs. uniformly distributed:	
Reaction (1842)(3) + 85	5611 lbs.
	17,477 lbs.

From "Lansing Building & Safety Code"- amended 2-16-48:

Compression in Bulk Masonry equals 90 lbs. per square inch for concrete blocks and the bearing pressure for concentration loads equals 115 lbs./sq. in.

Hence:

For a 16" x 16" masonry column.

$(16)^2(90) = 23,000$ lbs.

The allowable is more than the actual load, hence design is O.K.

Investigation of Design of Masonry Walls.

Exterior basement wall:

Analysis will be made for section of one foot thickness.

Loads:

From roof and roof truss	2550 lbs.
From steel studs(double)(45)(16)(2)	1440 lbs.
From brick masonry	500 lbs.
From plaster	200 lbs.
From second floor slab, live load, partitions and joist	1300 lbs.
From first floor slab, live load, partitions and joist	1300 lbs.
	7290 lbs.

From "Lansing Building & Safety Code" - amended 2-16-48:

Compression for Bulk Masonry equals 90 lbs./sq. inch for Concrete Blocks and the Bearing Pressure for Concentrated Loads equals 115 lbs./sq. inch.

Area of 144 sq. inches

Hence (144)(90) = 12,960 lbs.

The Code also states that no masonry wall shall have a heighth between horizontal lateral supports of more than twenty-two times its thickness:

(22)(1) = 22 ft. allowable
Design has 10 ft., hence O.K.

Wall is twelve inches thick and hence, conforms to the Lansing Code which states that the minimum masonry wall thickness shall be twelve inches.

Investigation of Design of Twelve Inch Concrete
Wall at Stair Section.

This wall rests on a continuous footing and resists earth pressure increasing with the depth. The span is ten feet.

Using $w = 100$ lbs/.sq. ft.

$P = Cwh$

$= (0.30)(100)(10)$

$P = 300$ lbs.

$R_A + R_B = 300$ lbs.

$(R_A)(10) = (300)6.67$

$R_A = 200.0$ lbs.

$R_B = 100$ lbs.

Maximum bending moment:

$M = (6.6)(200)(12)$

$M = 15{,}850$ in.-lbs.

Area of steel:

$$A_s = \frac{M}{f_s jd} = \frac{15{,}850}{(16{,}000)(.857)(12)}$$

$A_s = .0964$ sq. in.

$\frac{1}{2}"\phi$ @ 12" o.c. = 0.20 sq. in. Hence O.K.

Depth:

$M = Cxjd$

$$d^2 = \frac{15{,}850}{1768} = 90$$

$d = 3"$ Design has 12" O.K.

Shear:

$$v = \frac{300}{(12)(.857)(12)}$$

$v = 2.42$ lbs./sq. in.

Bond:

$$u = \frac{b}{\Sigma o} = \frac{(2.4)(12)}{(1.6)}$$

$u = 18$ lbs./sq. in. O.K.

Investigation of Design of Footings

Wall Footings:

Wall footings are continuous and can be designed as cantilever beams one foot wide when the wall loads are constant.

Loads:

Wall load	=	7290 lbs./ft.
Footing load$(\frac{20}{12})(1)(1)(150)$	=	167 lbs./ft.
		7457 lbs./ft.

Using an allowable bearing capacity of 4500 lbs./sq.ft. (Allowable used by O. J. Munson Firm in designing footings).

Area of footing $\frac{7457}{4500} = 1.6$ sq. ft./ft. length.

Footing is 20" x 12", hence area is just enough.

Bending moment:

The footing is 1 ft. 8 inches long and projects 4 inches on each side of the wall, forming cantilever beams.

net pressure $p = \frac{7457}{1.667} = 4500$ lbs./sq. ft.

Take a width of 1 ft. of this footing as a unit beam.

$$\text{Maximum moment} = \frac{wl^2}{2}$$

$$= \frac{(4500)(4)^2}{(2)(12)}$$

$$M_{max} = 3{,}000 \text{ in.-lbs.}$$

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

$$= \sqrt{\frac{3{,}000}{(157)(12)}}$$

$$d = 1.2 \text{ inches}$$

A depth of 12" is being used, hence safe.

Shear stress:

$$v = \frac{V}{bjd}$$

$$= \frac{(4500)(4)}{(12)(.857)(12)(12)}$$

$$v = 12.1 \text{ lbs./sq. in.}$$

Within allowable shear stress of 120 lbs./sq. in.

Steel:

$$A_s = \frac{M}{fjd}$$

$$= \frac{3000}{(16,000)(.857)(12)}$$

$$A_s = .0182 \text{ sq. inches}$$

An area of 0.12 sq. inches is obtained from the 3-$\frac{1}{2}$"ø continuous bars used, hence is ample.

Column footings:

Loads:

Column load	= 18,477
Weight of footing $(2.33)^2(1.67)(150)$	= 1,360
	19,837

Area of base:

$$\frac{19,837}{4500} = 4.41 \text{ sq. ft.}$$

Design has (2.33)(2.33) = 5.41 sq. ft., hence safe.

Net pressure:

$$p = \frac{19,837}{5.41}$$

$$p = 3660 \text{ lbs./sq. ft.}$$

Moment:

$$M = \frac{wl^2}{2}$$

$$= \frac{(3660)(28)(6)^2}{(12)(12)(2)}$$

$$M = 12{,}800 \text{ in.-lbs.}$$

Depth:

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

$$= \sqrt{\frac{12{,}800}{(157)(28)}}$$

$$d = 1.71 \text{ inches} \qquad \text{O.K.}$$

Steel:

$$A_s = \frac{M}{fjd}$$

$$= \frac{12{,}800}{(16{,}000)(.857)(16)}$$

$$A_s = .0585 \text{ sq. inches}$$

Check for bond:

$$\Sigma 0 = \frac{V}{ujd}$$

$$= \frac{(3660)(28)(6)}{(144)(140)(.857)(16)}$$

$$\Sigma 0 = 2.23 \text{ in.}$$

Steel used:

$6\text{-}\frac{1}{2}"\phi$ @ $4"$ o.c.

$A_s = .60$ sq. in. O.K.

$\Sigma 0 = 4.7$ in. O.K.

Summary

Structurally, the design of the Brick Apartment Buildings is very safe. In most instances there is some overdesigning. It is not so much that large structural members were chosen, as it is that the spans were very short and the members were not stressed under a very large maximum moment. There is only one feature that this author found to be bordering on the unsafe side and that is the design of the wall footings.

The size of the wall footings are 20" x 12" which gives an area of 1.67 sq. ft. per lineal foot. The allowable bearing soil pressure used by the designing firm is 4500 lbs. per sq. ft. Thus with the computed foundation load of 7450 lbs. per lineal foot on the wall footings, the area of 1.67 sq. ft. is just barely enough to withstand the load. It is this authors opinion that the width of the wall footings should have been enlarged to an even 2 ft. to safeguard against excessive detrimental settlements. Other than this one aspect, the apartments are definitely structurally safe.

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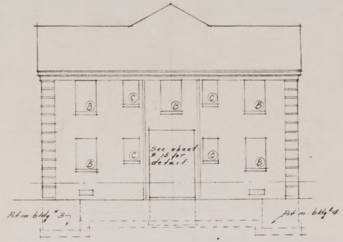
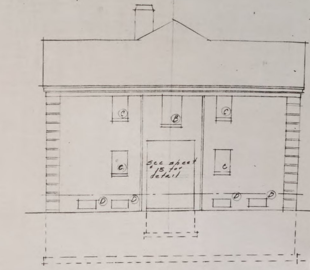
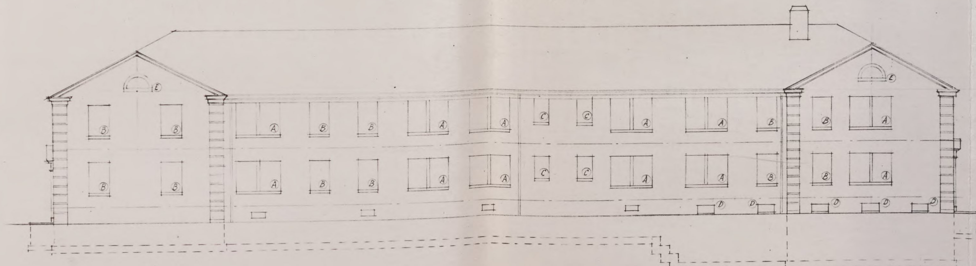


END ELEVATION
One elevation thus.

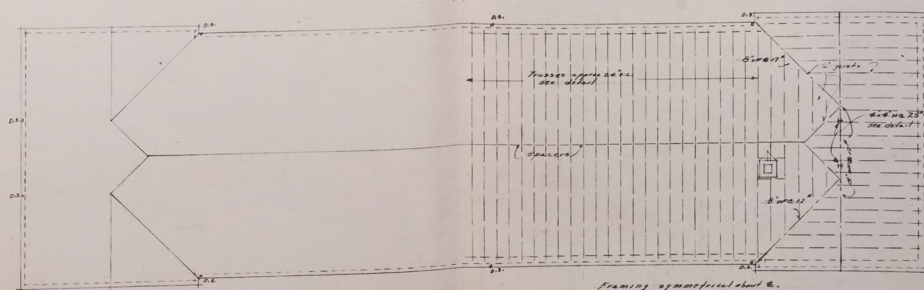


END ELEVATION
One elevation thus.

SCALE 1/8"=1'-0"

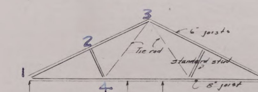


SIDE ELEVATION
Opp. side similar.

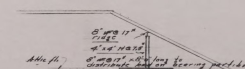


Framing symmetrical about ℄.

ROOF PLAN
scale 1/8"=1'-0"



TYPICAL TRUSSES 24" o.c.
All joints welded.
No scale

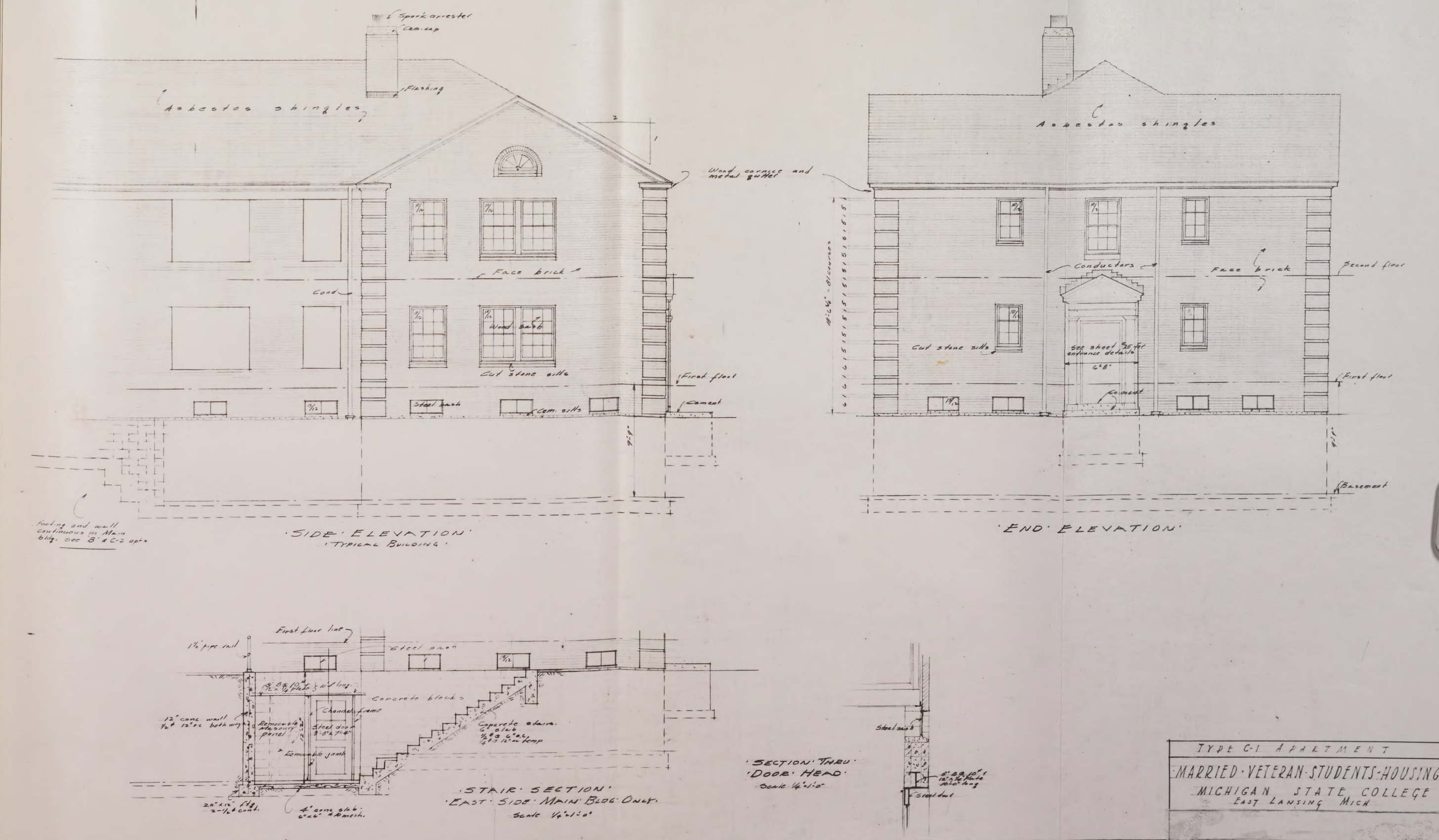


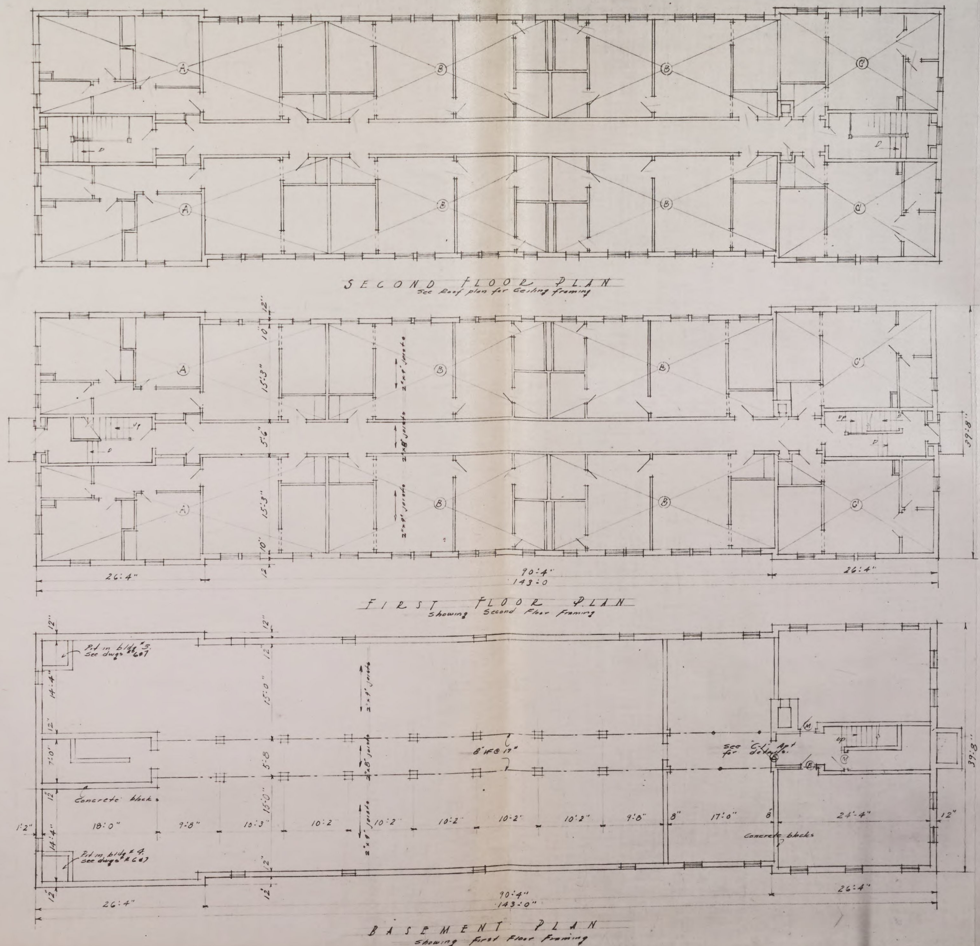
COLUMN DETAIL
No scale

TYPICAL BUILDING

MARRIED VETERAN STUDENTS HOUSING

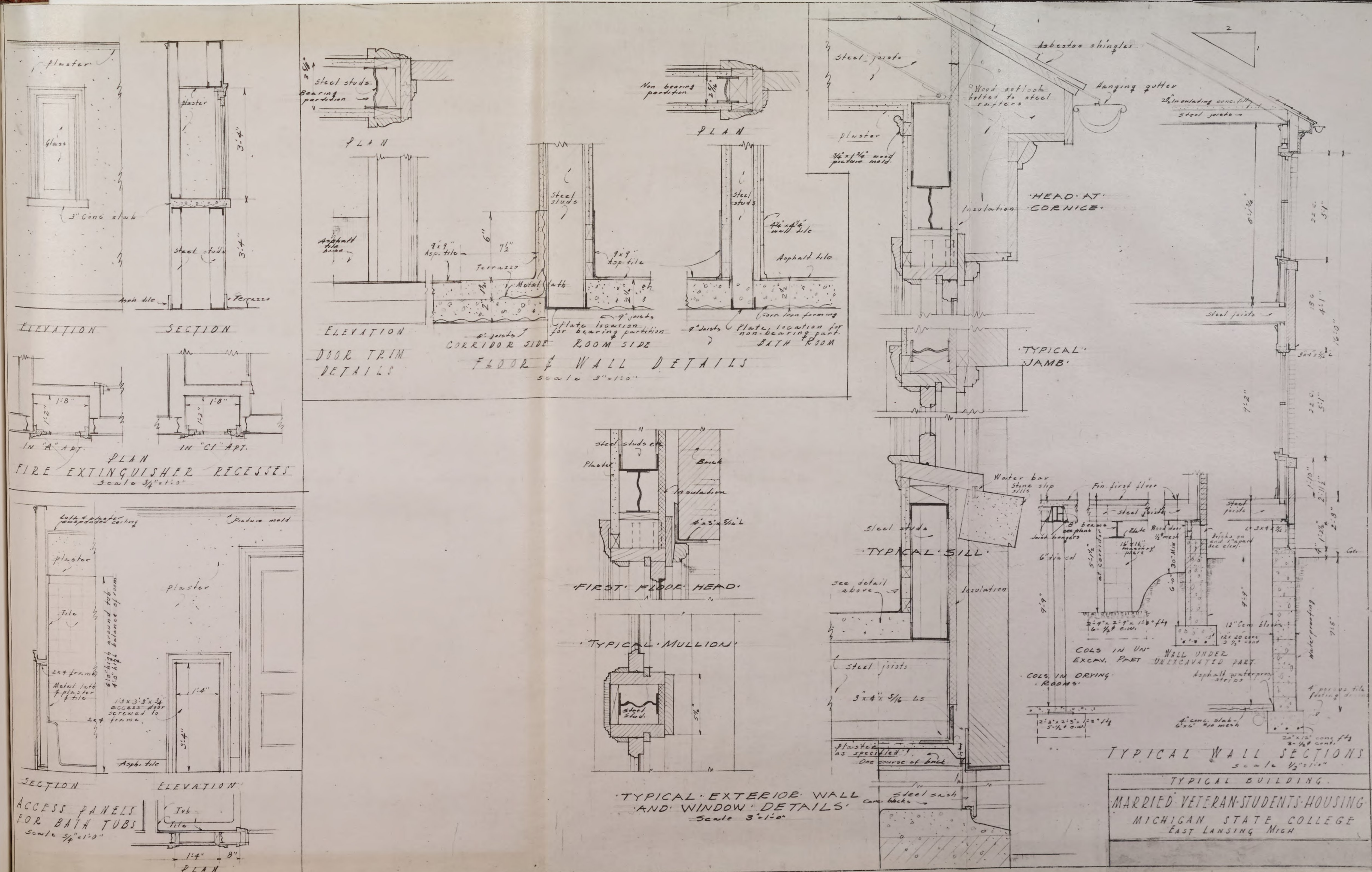
MICHIGAN STATE COLLEGE
EAST LANSING MICH

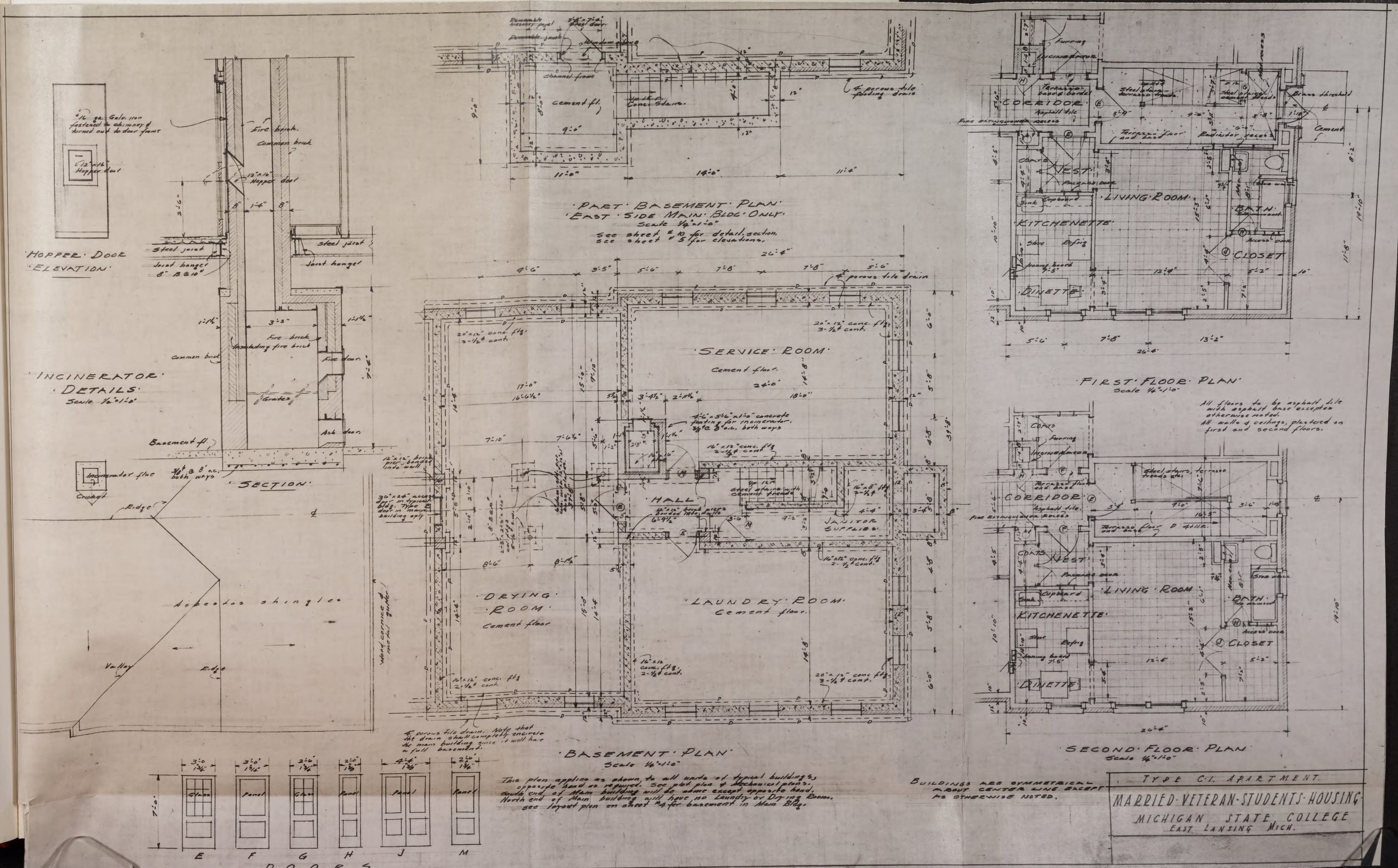

Spark arrester
Cem. cap
Flashing
Asbestos shingles
Wood cornice and metal gutter
Face brick
Cond.
Wood sash
Cut stone sills
First floor
Cement
Steel sash
Cem. sills
Footing and wall continuous in Main bldg. See B & C-2 apts.
SIDE ELEVATION
TYPICAL BUILDING
Asbestos shingles
Conductors
Face brick
Second floor
Cut stone sills
See sheet 25 for entrance details
6'8"
Cement
First floor
Basement
END ELEVATION
First floor line
1½" pipe rail
Steel sash
Concrete blocks
12" conc. wall
Removable masonry panel
Channel frame
Steel door
Removable jamb
Concrete stairs
20"x10" ftg.
4" conc. slab
STAIR SECTION
EAST SIDE MAIN BLDG. ONLY
Scale ¼"=1'-0"
SECTION THRU DOOR HEAD
Scale ¼"=1'-0"
Steel sash
Steel dowl
TYPE C-1 APARTMENT
MARRIED VETERAN STUDENTS HOUSING
MICHIGAN STATE COLLEGE
EAST LANSING MICH



NOTE.- BUILDING #3 SHOWN
BLDGS. #5 & #6 SIMILAR.
BLDGS. #2 & #4 OPP. HAND
REVERSED.
SEE PLOT PLAN ARRANGEMENT.

TYPICAL BUILDING
MARRIED-VETERAN-STUDENTS-HOUSING- MICHIGAN STATE COLLEGE EAST LANSING MICH


DOOR TRIM DETAILS
ELEVATION
SECTION
PLAN
Plaster
Glass
3" Conc slab
Steel studs
Asph tile
Terrazzo
FIRE EXTINGUISHER RECESSES
Scale 3/4"=1'-0"
IN "A" APT.
IN "C1" APT.
Steel studs
Bearing partition
Non bearing partition
Asphalt tile base
9"x9" Asp. tile
Metal lath
Asphalt tile
4¼"x4¼" wall tile
Corr. Iron forming
Plate location for bearing partition
Plate location for non-bearing part.
CORRIDOR SIDE
ROOM SIDE
BATH ROOM
FLOOR & WALL DETAILS
Scale 3"=1'-0"
Steel joists
Asbestos shingles
Hanging gutter
Wood outlook bolted to steel rafters
3/4"x3/4" wood picture mold
Insulation
'HEAD AT CORNICE'
'TYPICAL JAMB'
'TYPICAL SILL'
Water bar
Stone slip sills
Steel studs
See detail above
Steel joists
3"x4"x5/16" Ls
Plaster as specified
One course of brick
Steel sash
Conc. blocks
'FIRST FLOOR HEAD'
Brick
Insulation
4"x3"x5/16" L
'TYPICAL MULLION'
Steel Stud.
'TYPICAL EXTERIOR WALL AND WINDOW DETAILS'
Scale 3"=1'-0"
Lath & plaster suspended ceiling
Picture mold
Tile
2x4 frame
Metal lath & plaster & tile
6'0" high around tub 4'0" high balance of room
1/3x3'3"x3/4 access door screwed to frame
Asph. tile
Tub
ACCESS PANELS FOR BATH TUBS
Scale 3/4"=1'-0"
Fin first floor
Steel joists
8" beams
Joist hangers
6" dia col
Plate
16"x16" masonry piers
COLS IN UNEXCAV. PART
COLS. IN DRYING ROOMS
WALL UNDER UNEXCAVATED PART
12" Conc blocks
Asphalt waterproof strips
Waterproofing
4" conc. slab 6"x6" mesh
4" porous tile footing drains
TYPICAL WALL SECTIONS
Scale 1/2"=1'-0"
TYPICAL BUILDING
MARRIED VETERAN STUDENTS HOUSING
MICHIGAN STATE COLLEGE
EAST LANSING MICH


HOPPER DOOR ELEVATION
INCINERATOR DETAILS
Scale ½"=1'-0"
SECTION
PART BASEMENT PLAN
EAST SIDE MAIN BLDG ONLY
Scale ⅛"=1'-0"
See sheet #10 for detail, section.
See sheet #5 for elevations.
SERVICE ROOM
Cement floor
HALL
JANITOR SUPPLIES
DRYING ROOM
Cement floor
LAUNDRY ROOM
Cement floor
BASEMENT PLAN
Scale ¼"=1'-0"
This plan applies as shown to all units of typical buildings, opposite hand as required. See plot plan & mechanical plans.
South end of Main building will be same except opposite hand.
North end of Main building will have no Laundry or Drying Room.
See layout plan on sheet #4 for basement in Main Bldg.
Asbestos shingles
DOORS
E F G H J M
FIRST FLOOR PLAN
Scale ¼"=1'-0"
CORRIDOR
VEST
LIVING ROOM
KITCHENETTE
BATH
CLOSET
DINETTE
All floors to be asphalt tile with asphalt base except as otherwise noted.
All walls & ceilings plastered on first and second floors.
SECOND FLOOR PLAN
Scale ¼"=1'-0"
BUILDINGS ARE SYMMETRICAL ABOUT CENTER LINE EXCEPT AS OTHERWISE NOTED.
TYPE C-1 APARTMENT
MARRIED VETERAN STUDENTS HOUSING
MICHIGAN STATE COLLEGE
EAST LANSING MICH.

ROOM USE ONLY