A COST COMPARISON BETWEEN A REINFORCED CONCRETE AND A STRUCTURAL STEEL BUILDING FRAME

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

George Seymour

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A COST COMPARISON BETWEEN A REINFORCED CONCRETE AND A STRUCTURAL STEEL BUILDING FRAME

A Thesis Submitted to

The Faculty of

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of

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by

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INTRODUCTION

The comparison of cost is truly vital and necessary to all people both in business and out if they are to receive full value for their dollar. Neither a person's wage nor margin of profit is so great as to enable that person to ignore price variation when buying. It is with this in mind that I decided to make a cost comparison of two types of building frames now in common use: reinforced concrete and structural steel.

Generally speaking, the two frames are used in different types of buildings. The reinforced concrete frame is used where a fireproof building is required, as in schools, apartment houses, department stores--in general, fireproof buildings under twenty stories in height. Steel frame buildings generally are used for mill buildings, factories, or very tall structures. There is, however, much overlapping of the two where both might be addaptable

In this paper I wish to investigate the cost of the two types of frames where either type might be used--that is, in a building that does not have to be fireproofed and is small enough to be supported by a reinforced conrete frame. With this in mind I selected a small apartment building to be designed for both a reinforced concrete and a structural steel frame. It is true that for this size building, wood frame construction would be possible, and in many cases, practical. However, from a maintainence and life of structure standpoint, wood frame construction was ruled out. Also, while fireproof construction was not necessare, the added protection of a concrete or steel structure was a large factor in eliminating wood construction from consideration.

ARCHETECTURAL CONSIDERATIONS

While the general layout of the building would ordinarily concern the archetect rather than the engineer, it is closely enough tied to the structural design to deserve brief mention here.

The initial problem was providing the required floor area with the least outside perimeter while still satisfying the necessary light and ventilation requirements for each apartment as a whole and certain rooms in particular: namely the kitchen and bathroom. For this reason a cross plan was adopted as the plan of the building which allowed two apartments per wing with a central loaation of stairwell. With this plan, corner ventilation is secured for each apartment and adequate window is available for lighting.

A ceiling height of eight feet and a story to story height of nine feet was decided upon. In the steel frame building, the eight foot ceiling height was modified somewhat in the north-south wings where it was necessary to use ten inch beams with a four inch slab, leaving seven feet ten inches clear. This was still further reduced two inches for floor and ceiling finishes leaving seven feet seven inches clear. In this respect, the steel frame building would furnish less desireable apartments than the concrete frame building. It is true that the story to story height would have been increased for the steel frame building. That, however, would have eliminated one necessity in this cost comparison of the two buildings: that of making both designs identiaal. Again it is true that for both types the story to story height could have been increased. This, however, would increase the cost of the concrete frame structure with no proportionate advantage.

The nine foot story to story height also necessitated a more or less radical departure from the conventional bean and slab type of reinforced concrete building. Instead of the typical narrow, deep beam, a shallow, wide, and relatively heavily reinforced beas was used. A beam of this type requires more concrete and steel than the conventional beam but this is overshadowed by its many advantages. I believe the greatest of these advantages is the reduction in story height which in this case would amount to about a foot. Archetecturally and economically this is desireable. Secondly, the need of a drop ceiling to conceel the beams is obviated, a considerable saving. The wide shallow beam, protruding no more than four inches below the rest of the ceilingbecomes a harmonious archetectural feature of the room and need not be concealed. The third distinct advantage is the decrease in clear span lingth for the floor slabs with a subsequent decrease in slab steel resulting from reduced critical moment. This offsets in a large way the necessary increase in beam steel. Lastly, the wide, shallow beam is easier and cheaper to form than the deep, narrow beam. In times of high labor and material cost, both of which exist today, this is a substantial saving. For these reasons, the slab-band method of framing was chosen for the reinforced conrete building.

Exterior walls of the building are to be of cavity wall construction--brick exterior with a two inch air space and four inch cinder block plastered on the inside. All interior partitions are to be of three inch gypsum block plastered both sides. Ceilings are of accoustical tile applied, in the concrete frame building, directly to the concrete with mastic, and in the steel building, to furring strips attached to the steel floor beams. Floors are to be wood block laid in mastic directly on the cementslab.

DESIGN OF CONCRETE STRUCTURE

The reinforced concrete frame building was designed in accordance with the American Concrete Institute Specifications, and the building code requirements of the City of Midland, Mich. where the building would be erected. Two departures from the ACI code were made. The first was decreasing the recommended area of the columns from one hundred twenty squart inches to one hundred square inches. This change was made because, in most cases, the collumns were considerably understressed. However, the recommended minimum thickness of ten inches was held. The second departure was in using one inch fire protection for beams instead of one and one half inches. However, inasmuch as the beams, except spandrel beams, are at least eighteen inches wide, such beams could easily be classed as slabs and the one inch protection is adequate for a four hour rating.

SLAB DESIGN

Both one way and two way slabs were used in the building, depending upon the spans.

One way slab Mark S1

Loading

design live load	30 #/sq ft
slab weight	50 #/sq ft
roofing and ceiling finish	10 #/sq ft
Total load W	90 #/sq ft

specifications

$fc = 1350$ psi $n = 10$ $fs = 20{,}000$ psi

moment

effective span = 11 - 3 + 0.7 = 8.7 ft

Positive = $1/14\ Wl^2 = 1/14 \times 90 \times 8.7^2 = 486$ ft-lb

negative = $1/10\ Wl^2 = 1/10 \times 90 \times 8.7^2 = 683$ ft-lb

$K = M/bd^2 = 683 \times 12 \div 12 \times 3^2 = 75.9$ OK

steel area

positive $As = M \div fsjd = 470 \times 12 \div 20{,}000 \times .87 \times 3 = .112\ in^2$

min allowed $.0025 \times 12 \times 4 = 0.12\ in^2$

negative $As = 683 \times 12 \div 20{,}000 \times .87 \times 3 = 0.157$

bond

$V = 90 \times 8{,}7 \div 2 = 378$ #

$u = 150$ psi

sum of perimeters = $V \div ujd = 378 \div 150 \times .87 \times 3 = .97\ in^2$

steel use 1/2" deformed bars at 10"

shear

$v = V \div bjd = 378 \div 12 \times .87 \times 3 = 12$ psi   OK

Two way slab mark S2

moment

effective length = 14'

short span

pos mom = $.054\ Wl^2 = .054 \times 90 \times 14^2 = 1130$ lb-ft

neg mom = $.071\ Wl^2 = .071 \times 90 \times 14^2 = 1380$ lb-ft

long span

pos mom = $.049\ Wl^2 = .049 \times 90 \times 14^2 = 652$ lb-ft

neg mom = $.037\ Wl^2 = .037 \times 90 \times 14^2 = 868$ lb-ft steel area

short span

neg As = M ÷ fsjd = 1380 x 12 ÷ 20,000 x .87 x 3 = .321 in^2

pos As = 1130 x 12 ÷ 20,000 x 87 x 3 = .263 in^2

long span

neg As = 863 x 12 ÷ 20,000 x .87 x 3 = 201 in^2

pos As 652 x 12 ÷ 20,000 x ,87 x 3 = .152 in^2

use 1/2" bars at 7" for short span

1/2" bars at 10" for long span

shear

V = 90 x 7 = 630 #

v = V ÷ bjd = 630 ÷ 12 x .87 x 3 = 20 psi OK

BEAM DESIGN

The maximun end moments in beams was determined by the Cross method of moment distribution. The proceedure, outlined briefly, was this. By the use of end coeffecients, a beam size was determined which was then checked by moment distribution. The first choice of beams turned out to be small and the size of the beams had to be increased and rechecked. A sample of the moment distribution computations are included on the next page.

Maximun shear at the supports of the beams was obtained by adding to the shear due to the total load, the difference of the two end moments divided by the length. Shear in the beams was not critical because of the width of the beams. Stirrups, therefore, were needed only in the spandrel beams of the first or second floors.

	Col A	Col B	Beam		Beam	Col A	Col B	Beam		Beam	Col A	Col B
Dist. Factor		.46	.54	T.L.	.35		.30	.35	D.L.	.54		.46
F.E.M.			+41.0		-41.0			+28.0		-28.0		
		+19.0	-22.0		+ 4.6		+ 3.8	+ 4.6		+15.0		+13.0
		- 6.5	+ 2.3		-11.0		- 1.0	+ 7.5		+ 2.3		+ 8.0
		+ 1.9	+ 2.3		+ 1.6		+ 1.3	+ 1.6		- 5.6		- 4.7
		-23.6	+23.6		-45.8		+ 4.1	+41.7		-16,3		+16.3
Dist Factor	.31	.31	.38	D.L.	.27	.23	.23	.27	T.L.	.38	.31	.31
F.E.M.			+42.0		-42.0			+51.0		-51.0		
	-13.0	-13.0	-16.0		- 2.5	- 2.0	- 2.0	- 2.5		+19.0	+16.0	+.6.0
	- 9.5	- 8.0	- 1.2		- 8.0	+ 1.9	+ 1.0	+ 9.5		- 1.2	+ 6.5	+ 6.0
	+ 5.8	+ 5.8	+ 7.1		- 1.2	- 1.1	-1. 0	- 1.2		- 4.3	- 3.5	-3.5
	-16.7	-15.2	+31.9		-53.7	- 1.1	- 2.0	+56.8		-37.5	+19.0	+18.5
Dist. Factor	.31	.31	.38	T.L.	.27	.23	.23	.27	D.L.	.38	.31	.31
F.E.M.			+51.0		-51.0			+42.0		-42.0		
	-16.0	-16.0	-19.0		+ 2.5	+ 2.0	+ 2.0	+ 2.5		+16.0	+13.0	+13.0
	- 6.5	0	+ 1.2		- 9.5	- 1.0	0	+ 8.0		+ 1.2	+ 8.0	0
	+ 1.3	+ 1.3	+ 1.7		+ .7	+ 0.6	+ 0,7	+ 0.7		- 3.6	- 3.1	- 3.1
	-21.2	-14.7	+34.9		-57.3	+ 1.6	+ 2.7	+53.2		-28.4	+17.9	+ 9.9

B C D

Column Line 2

Center moment

average end moment	(34.9 + 57.3) ÷ 2 =	41.1
center moment—simply supported beam		88.5
Corrected center moment		47.4 lb-ft

Shear

shear due to loading	16.1 k
shear sue to end moments	0.9
max shear	17.0 k

BEAM DESIGN

Specifications

fc = 1350   fs = 20,000   n = 10   u = 150   v = 90

Beam mark CD2

use d = 7" b = 48"

neg mom = 56.8 k-ft   pos mom = 44.2 k-ft

neg steel

$R = M \div bd^2 = 56800 \times 12 \div 48 \times 7^2 = 299$

Compression steel req'd

p' = 1% for compression steel

$A's = .01 \times 48 \times 7 = 3.36\ in^2$ 8 - 3/4" bars

p = 1.75% for tension stees

$As = .0175 \times 48 \times 7 = 5.87\ in^2$ 14 - 3/4" bars

pos steel

$R = 44{,}200 \times 12 \div 48 \times 7^2 = 226$ OK

$As = M \div fsjd = 44{,}200 \times 12 \div 20{,}000 \times .87 \times 7 = 4.36$

10 - 3/4" bars

shear

$V = 17$ k

$v = V \div bjd = 17{,}000 \div 48 \times .87 \times 7 = 58.2$ psi OK

Bond

sum of perimeters $= V \div ujd = 17{,}000 \div 150 \times .87 \times 7 = 18.7$psi OK

COLUMN DESIGN

column mark B2

col load 71.1 k

use 4 - 3/4" bars

allowable load

$P = .8\,Ag\,(.225\,f'c + fspg)$

$= .8 \times 100\,(.225 \times 3000 + 20{,}000 \times .0176)$

$= 82.4$ k

ties

use 1/4" bars spaced at 48d = 12"

FOOTING DESIGN

footing mark A3

loading 46.1 k

est footing weight 3 k

total weight 49.1 k

size of footing

allowable soilpressure 4 k/ft^2

area required $49.1 \div 4 = 12.3\ ft^2$

use 3.5 x 3.5' footing

net pressure $46.1 \div 12.3 = 3.78\ k/ft^2$

depth of footing

governed by moment

$M = 1.33 \times 3.5 \times .67 \times 3.78 = 10.8$ k-ft

$d = (M \div Rb)^{\frac{1}{2}} = (10{,}800 \times 12 \div 236 \times 42)^{\frac{1}{2}} = 3.6''$

governed by shear

assume d of 12"

$V = .33 \times 3.5 \times 2 \times 3.78 + .33 \times 2.8 \times 2 \times 3.78 = 15.85$ k

$b = 136''$ $v = 75$ psi

$d = V \div vjb = 15{,}850 \div 75 \times .87 \times 136 = 1.8''$

use a d of 12" + 3" cover. This is a larger d then called for by the computations but to allow for construction variables, a d of at least 12" is desireable.

steel

$As = M \div fsjd = 10{,}800 \times 12 \div 20{,}000 \times .87 \times 12 = .62$ in^2

bond

$V = 1.33 \times 3.5 \times 3.78 = 17.5$ k $u = 135$ psi

sum of perimeters $= 17{,}500 \div 135 \times .87 \times 12 = 12.4$ in^2

use 6 - 3/4" deformed bars

DESIGN OF STEEL FRAMEWORK

The steel frame building was designed in accordance with the American Institute of Steel Construction specifications. The beams and girders were designed as simply supported members with no end restraint. In a building of this size, the restraint developed by the concrete floor is sufficient to counteract wind loading. This accounts for the lack of sway bracing. All beams, girders and columns are rolled sections which is common in a building of this size.

FLOOR DESIGN

The floor system as in the concrete frame building, is composed of a combination of one and two way slabs. However the span length is smaller in the steel building resulting in lighter reinforcing. The span length was decreased in order to obviate the need for large heavy beams to conserve head room. Floor thickness was not decreased because any lessening of the sound insulating value of a four inch concrete floor was undesireable. Computations for the floor system are the sameas for the reinforced concrete frame building and will not be repeated here.

BEAM DESIGN

beam mark BC1

loading W = 875 #/ft

span l = 22'

Min depth = 1/24 x 22 x 12 = 11" use 10" WF

decrease allowable unit stress 10/11 x 20,000 = 18,200 psi moment == $1/8\ Wl^2 = 1/8 \times 925 \times 22^2 = 52{,}900$ lb-ft

section modulus = $M \div f = 52{,}900 \times 12 \div 18{,}200 = 34.9$

use 10" WF 33

COLUMN DESIGN

col mark C2

loading 156k

try 10" WF 29

$P = (17{,}000 - .485(96/1.34)^2)\ 8.53 = 124$ k too small

try 10" WF 39

$P = (17{,}000 - .485\ (96/1.98)^2)\ 11.48 == 182$ k OK

BASE PLATE DESIGN

col mark C2

base plate loading 156 k + 1 = 157 k

$fc = .8\ k/in^2$

area required = $157 \div .8 = 197\ in^2$

size of plate

col size 8" x 10"

length = .8 width

$lw = 197\ in^2$

or $.8\ w^2 = 197\ in^2$

w = 15.7 or 16"

l = 13"

max overhang = 3"

thickness of plate = $(\ .15 \times .75 \times 3^2)^{\frac{1}{2}} == 1.05"$

use 13" x 16" x 1" base plate for all cols

PIER DESIGN

All piers 14" x 17"

$p = 1.0\%$

allowable load

$P = 0.80\ (.225\ f'c\ Ag + pg\ Ag\ fs) = 167\ k$

steel area

$As = .01 \times 14 \times 17 = 2.38\ in^2$

use 4 - 7/8" deformed bars

ties

use 1/4" bars at 48d = 12"

FOOTING DESIGN

Footing C2

loading = 157 + 3 = 160 k

size of footing

allowable soil pressure = $4\ k/in^2$

area required = $160 \div 4 = 40\ ft^2$

use 6.5 x 6.5'

depth

governed by moment

$M = 3.7 \times 2.6 \times 6.5 \times 1.3 = 81.2$ k-ft

$d = (M \div Rb)^{\frac{1}{2}} = (81{,}200 \times 12 \div 236 \times 78)^{\frac{1}{2}} = 7.3"$

governed by shear

assume d = 12"

$V = 2 \times 1.5 \times 6.5 \times 3.7 + 2 \times 1.7 \times 3.5 \times 3.7 = 116.2\ k$

$$d = V \div vjb = 116.2 \div 75 \times .87 \times 154 = 11.7''$$

$$\text{use } d \text{ of } 15'' + 3'' = 15''$$

COST ESTIMATION

Estimating the cost of the two types of framing systems is probably the least accurate portion of this thesis. It is possible to predict with reasonable accuracy the performance of steel and concrete in a building, but the placing of these materials in the structure involves many variables, two of the greatest being the weather and the willingness and/or ability of human beings to work efficiently. Consequently, any attempt to predict anything with as many variables as construction work is apt to go awry. In making up this estimate, great reliance was placed on the "Builders Estimating Referance Book," by Harry Walker, and somewhat less reliance on my own experience on construction jobs. Even with two such authorities as guides, however, estimating is rather hazardous.

No account was taken of any functions that would be duplicated in the two building. For instance, no estimate was made of the excavating costs inasmuch as they would be identical for both structures.

In all the cost estimating, an attempt was made to break the operation or quantity into its smallest units. This, it is felt, leads to more accurate estimating. On the following page, the floor forms will be estimated in this manner. Thereafter, only total quantities and prices will be noted.

REINFORCED CONCRETE FRAME ESTIMATION

Floor forms

lumber required per 100 ft^2 of forms

1" sheathing + 20% waste	120 bf
2 x 6 joists at 24"	63 "
3 x 8 stringers	46 "
4 x 4 shores	86 "
beam sides	10 "
braces	21 "
total	346 bf

labor cost

cost per 1000 board feet

carp	32 hrs at	$2.07		$66.20
lab	28 hrs at	1.35		27.80
total				$104.00

form area needed = 17,100 sq ft

lumber needed = 171 x 346 = 59,200 bf

total labor cost = 104 x 59.2 = $6150.00

Materials cost

forms to be used three times

total lumber needed = 17,100 ÷ 3 = 5,700 sq ft

cost per 100 sq st of forms

1" lumber (.120 + .021 + .010) x 130 =	$19.62
2" lumber .063 x 105 =	6.63
3" lumber (.046 + .086) x 140	19.05
total per 100 sq ft	$45.29

total for the bldg = $45.29 x 57 = $2580.00

total cost—labor and materials	
$2580.00 + $6150.00 =	$8730.00
col forms	
labor cost	$780.00
material cost	373.00
total	$1153.00
footing forms	
labor cost	$36.40
material cost	16.05
total	$52.45
reinforcing steel	
material cost 10.75 ton	$3920.00
labor cost	878.50
total	$4798.50
concrete	
material cost 331 yds	$3730.00
labor cost	1315.00
total	$5045.00
Grand total, reinforced concrete frame	$19,778.95

STRUCTURAL STEEL FRAME ESTIMATE

Estimating the cost of the steel, that is, the fabrication and erection, was a job that I was not qualified to handled. Therefore, I obtained from the Jarvis Engineering Co., Lansing, Mich. an estimate for the job. To that I added the cost of the concrete floors, piers and footings to get the final estimate.

Steel frame, fabricating and erecting	
70.5 ton at $285.00	$20,500.00
Floor forms	
Material cost	$1,645.00
labor cost	5,180.00
total	$6,8250.00
Footing forms	
Material cost	$13.50
labor cost	27.30
total	$40.80
Pier forms	
material cost	$62.00
labor cost	128.60
total	$190.60
Reinforcing steel	
material cost	$1,680.00
labor cost	437.60
total	$2,117.60
Concrete	
material cost 186 yds	$2,090.00
labor cost	818.60
total	$2,908.60
Grand total, structural steel frame	$32,602.37

CONCLUSION

As noted from the previous figures, the cost of the steel frame runs substantially highter than the cost of the reinforced concrete frame. The total difference is $12,823.42. On the basis that the entire building will cost about $150,000.00, this represents about an 8% difference between the two types of framing.

The dominant factor in the greater cost of the steel frame building is the cost of fabricating and erecting the steel. I believe the reason for this lies in the type of building for which the cost analysis was made. The analysis clearly shaws that for a small structure with light loading, it is more economical to erect a reinforced concrete frame than one of structural steel.

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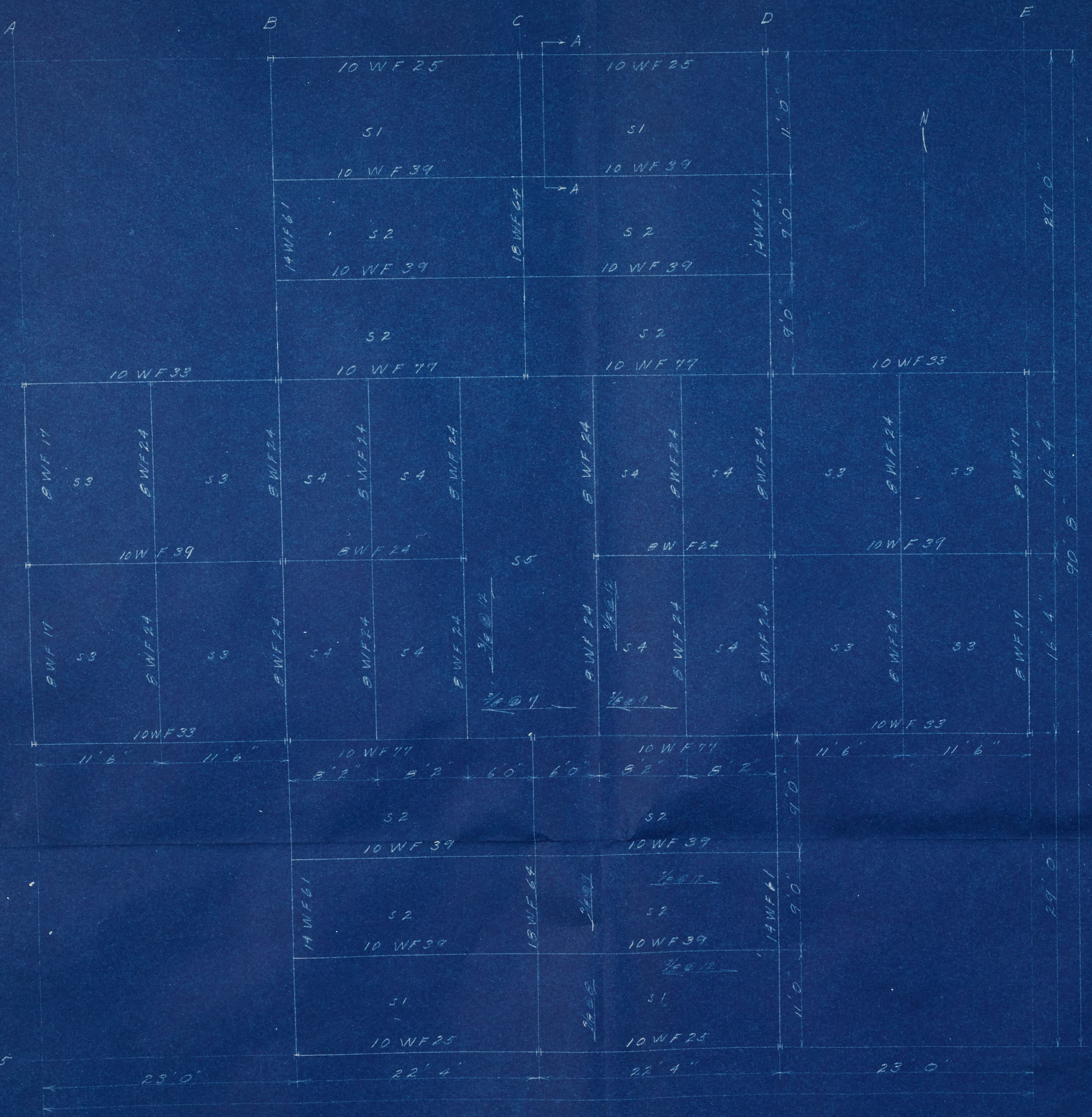
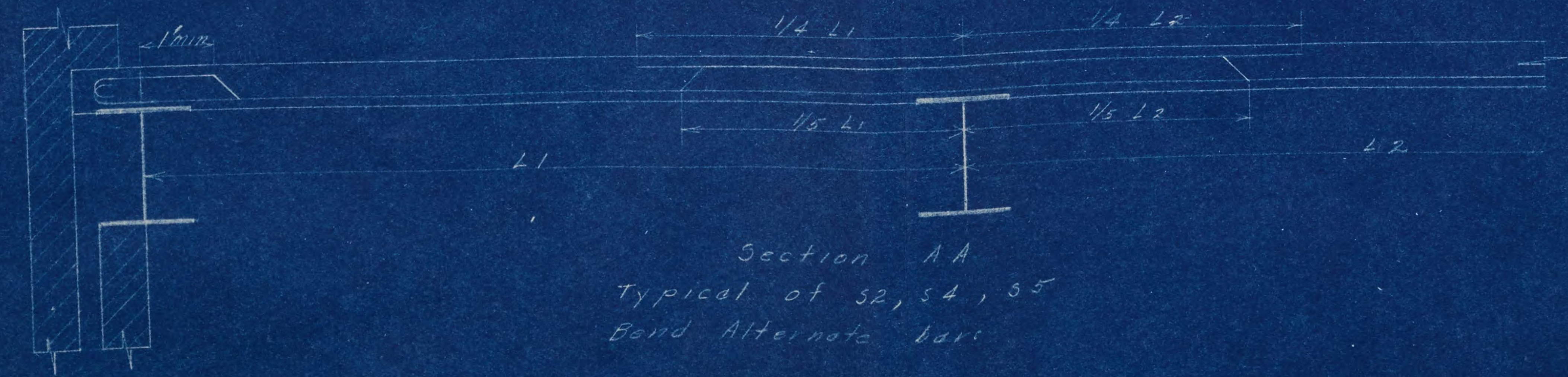
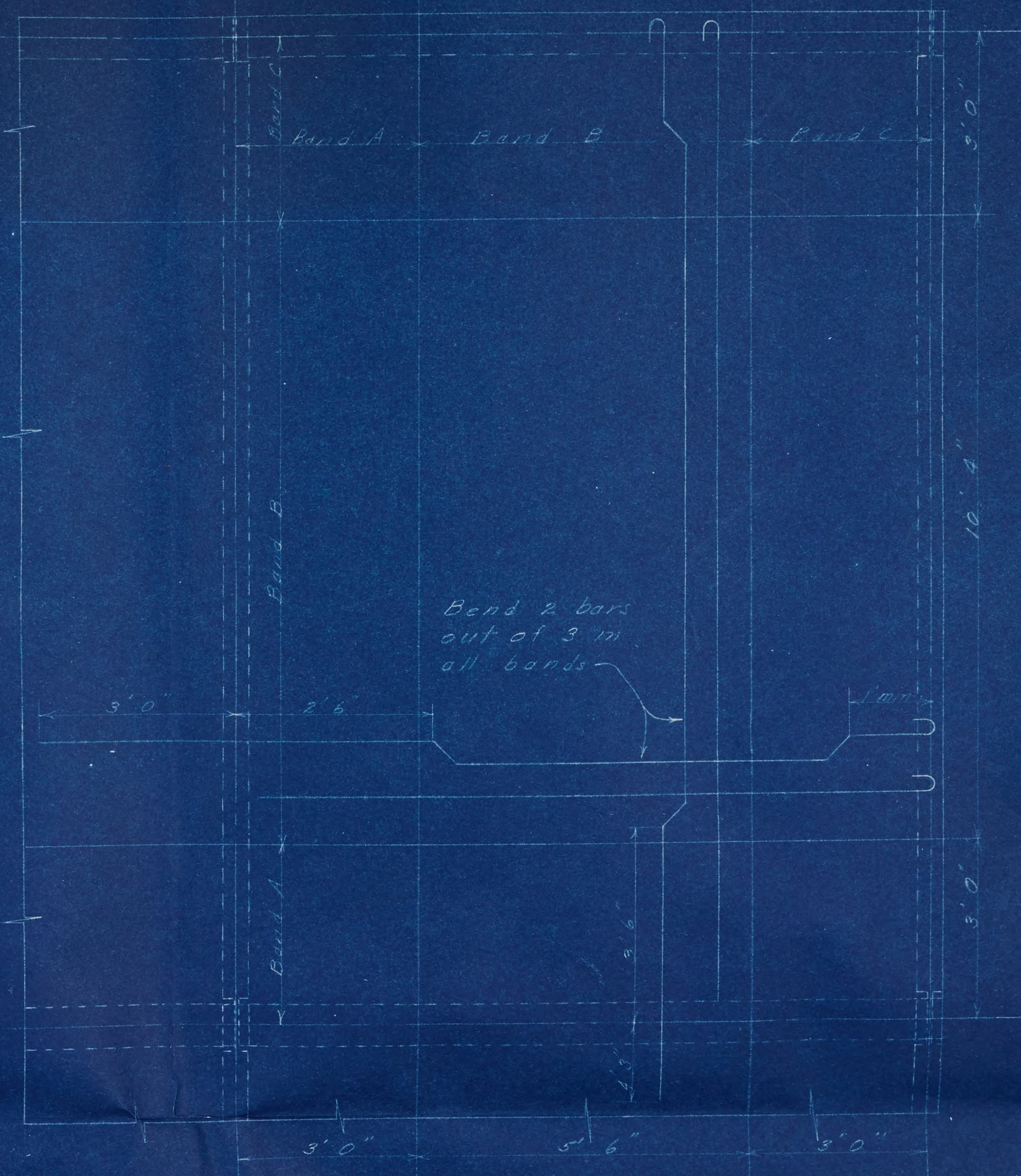


Roof Framing Plan
scale 1/8" = 1'



Section AA
Typical of S2, S4, S5
Bend Alternate bars



Plan S3

Slab Schedule						
Slab mark	direction	bar size	band A	band B	band C	Remarks
S1	NS	3/8				3/8 @ 8 bot
	EW	3/8				3/8 @ 12 top
S2	NS	3/8				3/8 @ 9 bot
	EW	3/8				3/8 @ 12 top
S3	NS	3/8	12	9	9W	
	EW	3/8	11	8	8W	
S4	NS	3/8				3/8 @ 12 top
	EW	3/8				3/8 @ 9 bot
S5	NS	3/8				3/8 @ 12 top
	EW	3/8				3/8 @ 4 bot

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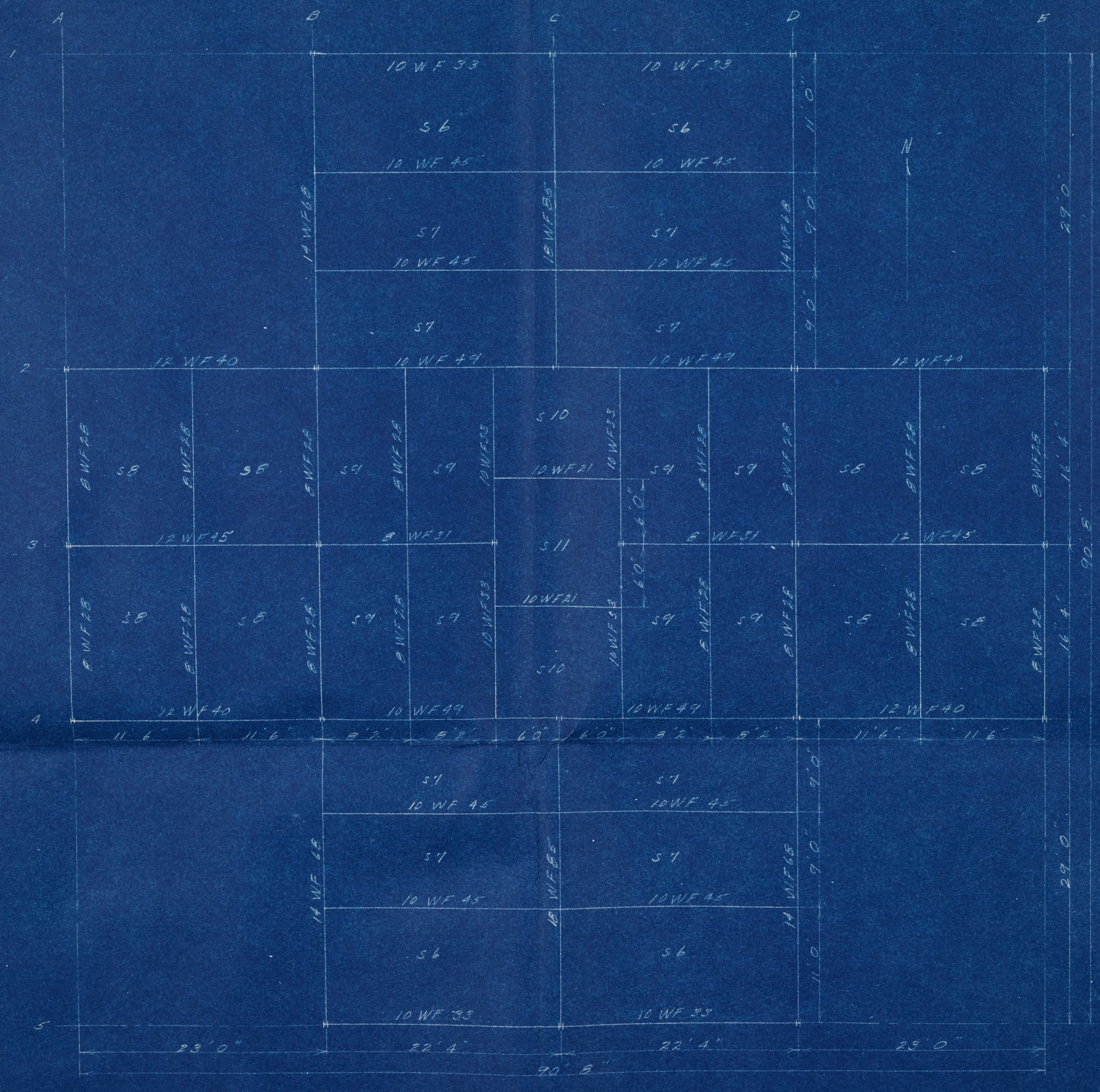
Sheet 1 of 3



1st & 2d Floor Framing
Scale: 1/8" = 1'

Slab Schedule						
slab mark	direction	bar size	band A	band B	band C	Remarks
6	NS	3/8				3/8 @ 7½ bot
	EW	3/8				3/8 @ 12 top
7	NS	3/8				3/8 @ 9 bot
	EW	3/8				3/8 @ 12 top
8	NS	3/8	11	8½	8½W	
	EW	3/8	10	7½	7½W	
9	NS	3/8				3/8 @ 12 top
	EW	3/8				3/8 @ 9 bot
10	NS	3/8	12	9	12	
	EW	3/8	12	9	12	
11	NS	3/8	12	9	12	
	EW	3/8	12	9	12	

W denotes band adjacent to wall

Notes

1. For slab bending details see page 1
   S8, 10, & 11 are similar to S3. S6, 7, & 9 are similar to S1, 2, 4, & 5
2. All concrete to have 3000 psi compressive strength in 28 days
3. All reinforcing to be intermediate grade billet steel deformed bars

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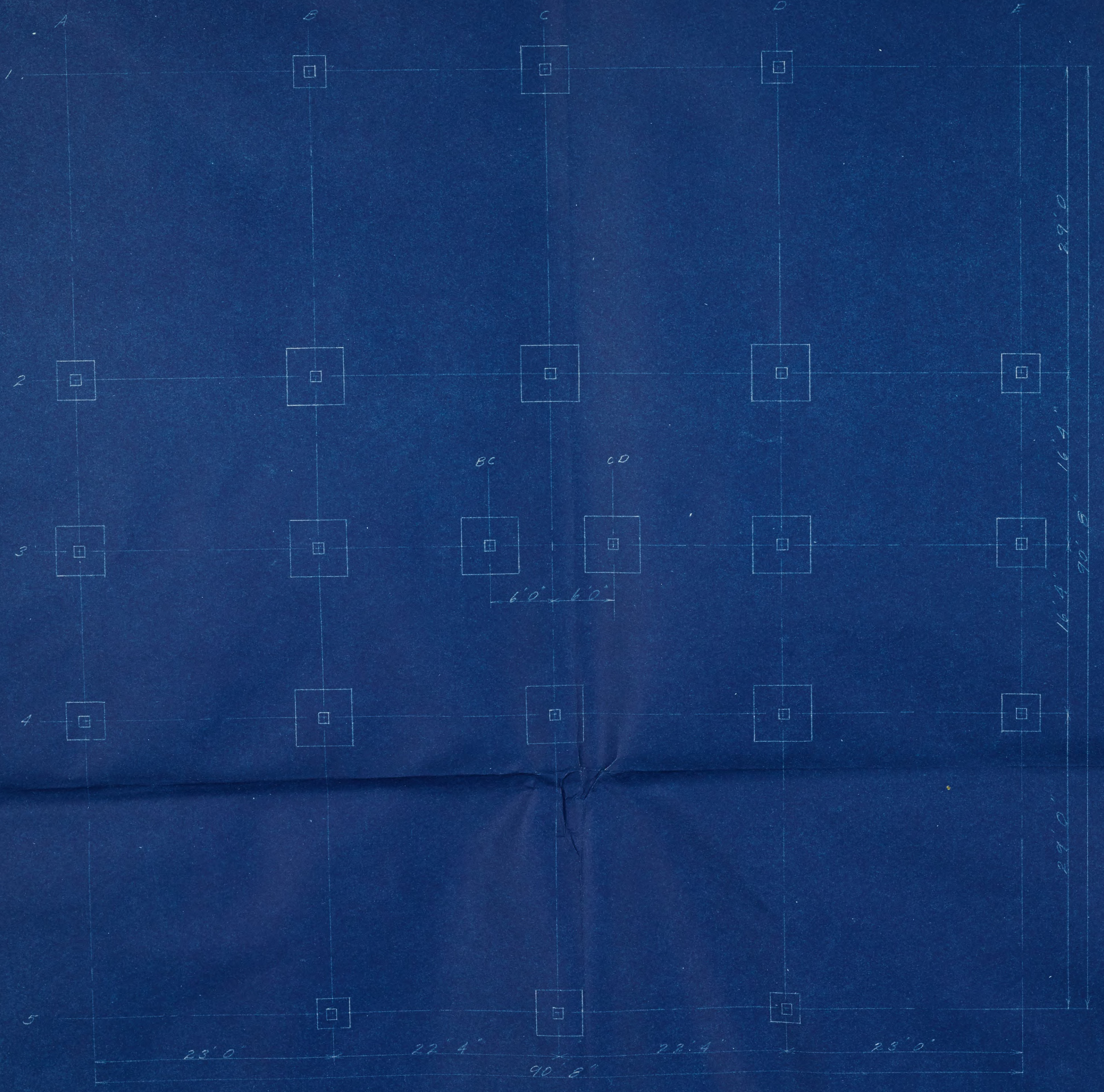
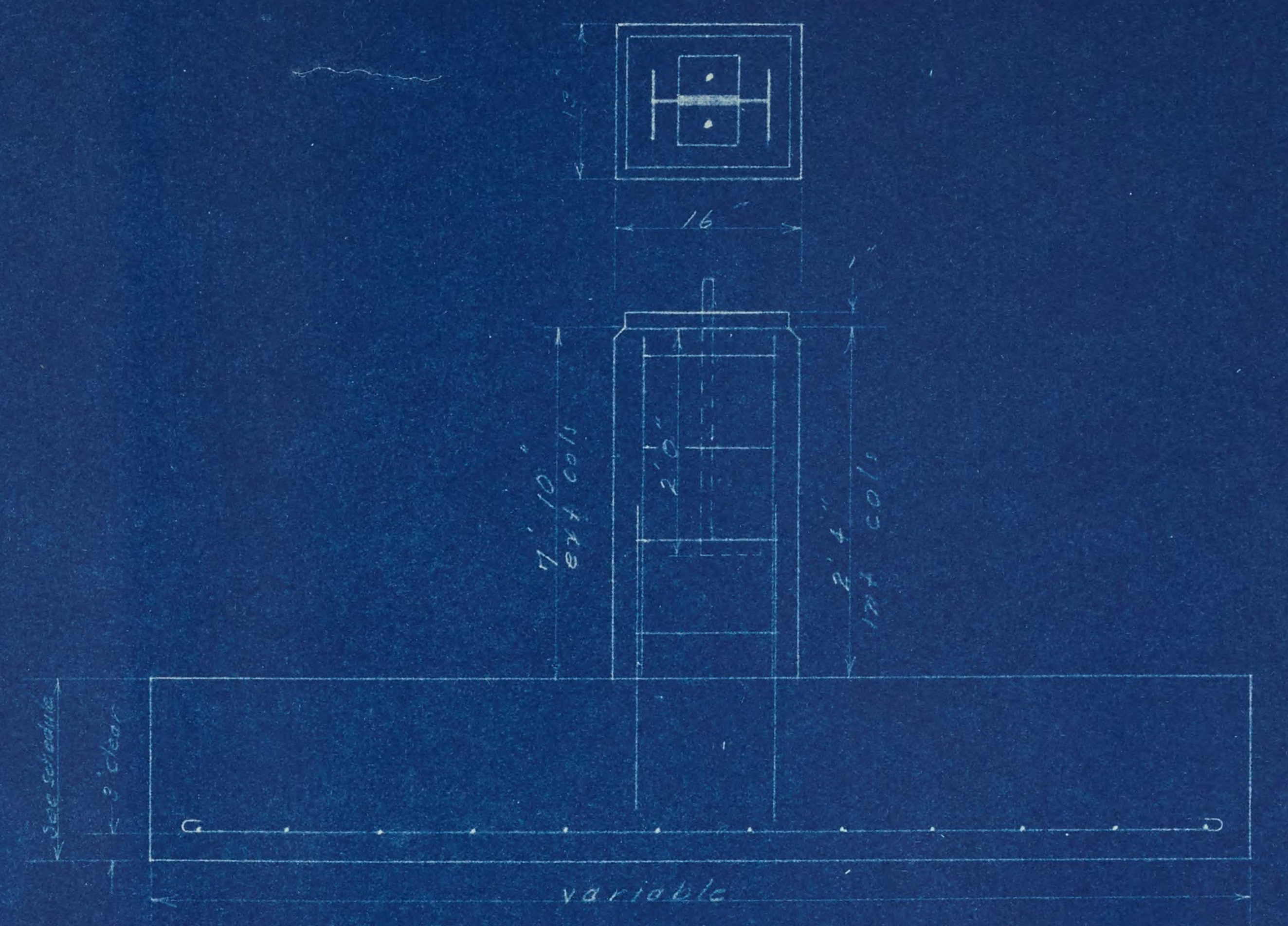
sheet 2 of 3



Foundation plan
scale 1/8" = 1'



Typical Footing

Column Schedule		
column	size	lgth
A2,3,4; E2,3,4, B1,5	10 WF21	20'2"
BC3, CD3	10 WF21	25'8"
B2,4 D2,4	10 WF29	20'2"
B3 D3	10 WF25	25'8"
C1, 5	10 WF25	20'2"
C2, 4	10 WF39	25'8"

Pier schedule		
Pier	lgth	steel
A2,3,4, B1,2,4,5 C1,5 D1,2,4,5 E2,3,4	7'10"	4-3/4"
B3, BC3, CD3 C2,4 D3	2'4"	4-3/4"

Footing schedule		
Ftg.	size	steel
C2, 4	6'6" x 6'6" x 18"	EW 14-1/2"
C1,5 B2,3,4 D2,3,4	5'6" x 5'6" x 18	EW 10-1/2"
B1,5 BC3 D1,5 CD3 A2,3,4 E2,3,4	4'6" x 4'6" x 18"	EW 11-3/8"

Notes
1. Use 4-3/4" dowels to anchor each pier
2. All ties are 1/4" ϕ @ 12"

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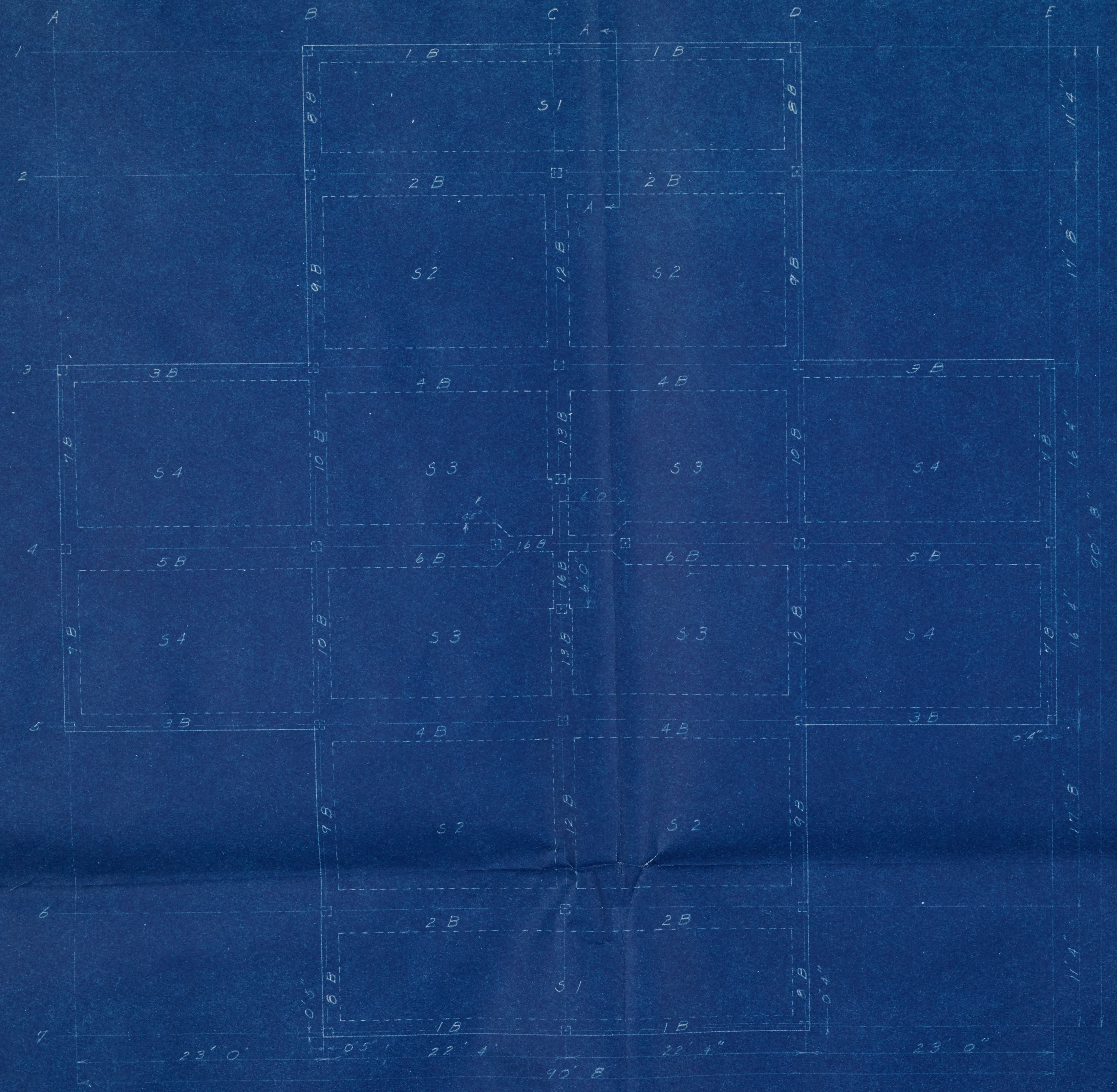
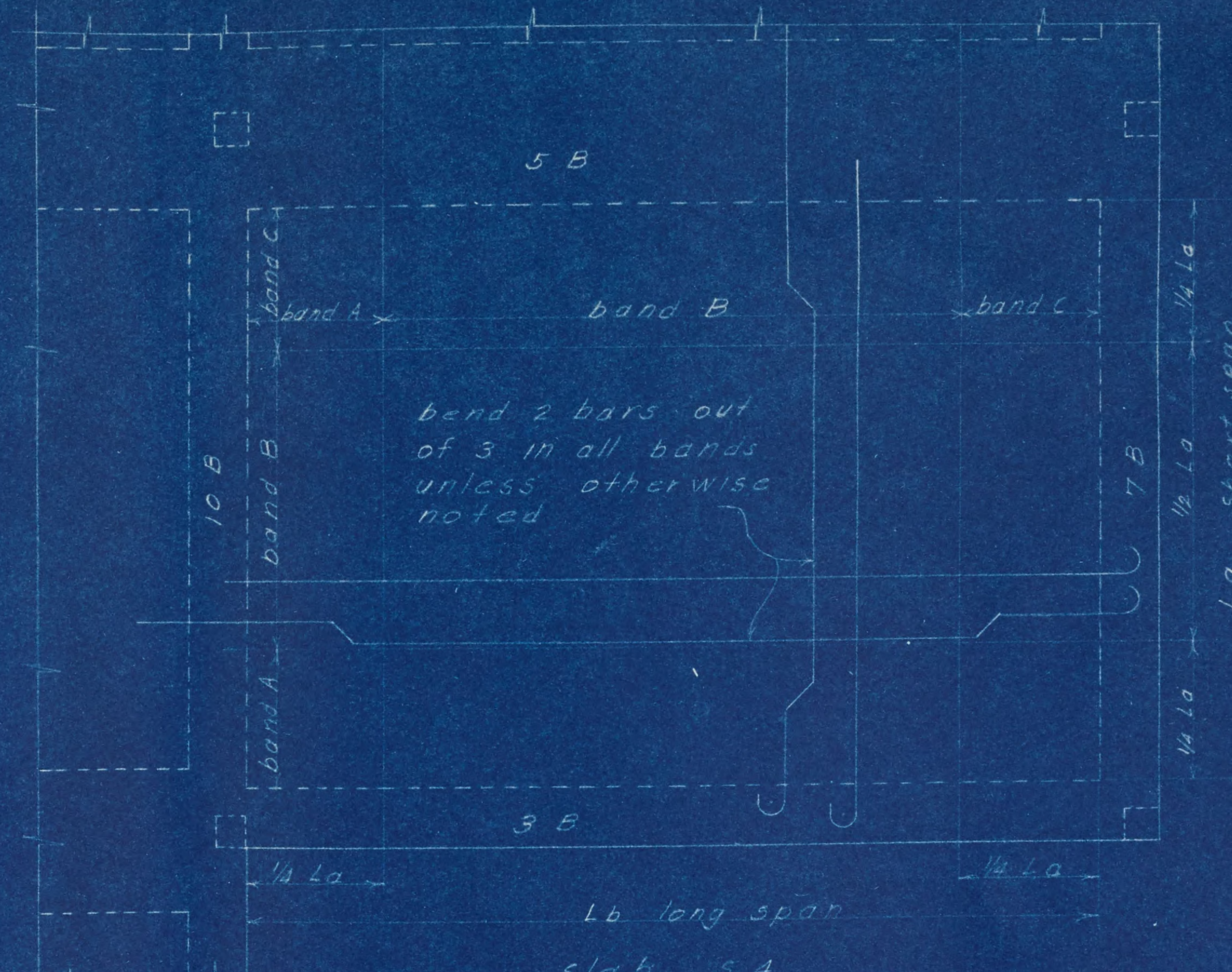
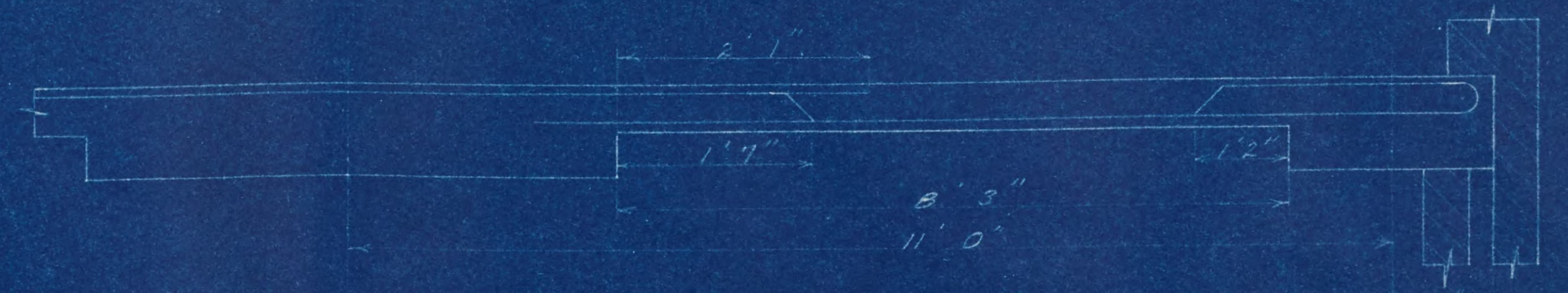


Roof Framing Plan

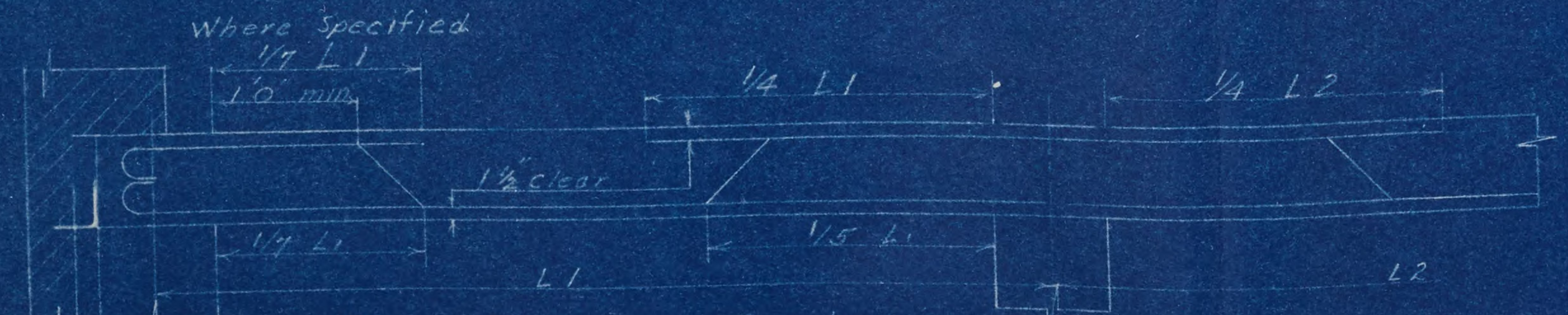


slab S4
Typical plan



Section AA

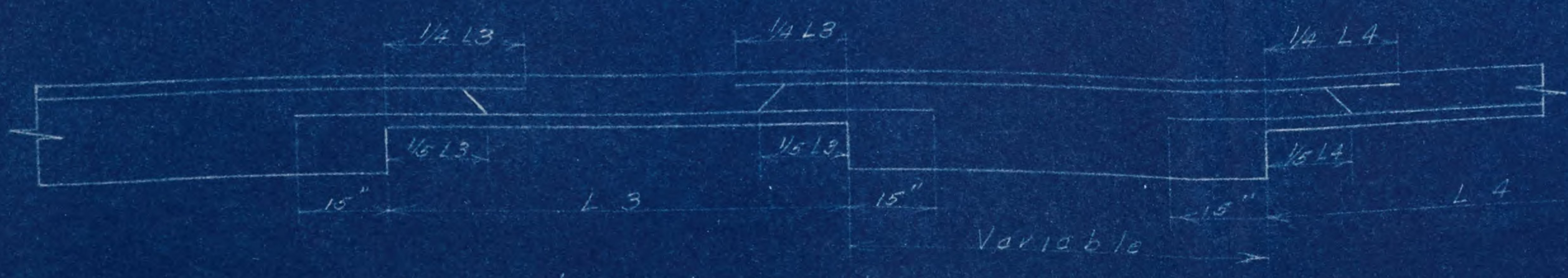
BEAM SCHEDULE							
beam	size		straight		Truss		Remarks
mark	W	D	No.	size	No	size	
1B	18	8	2	3/4	1	3/4	2 - 3/4 top ext col 4 - 3/4 top int col
2B	48	8	3	3/4	3	3/4	3 - 3/4 top ext col 4 - 3/4 top int col
3B	18	8	3	3/4			3 - 3/4 top ext col 5 - 3/4 top int col
4B	48	8	3	3/4	2	3/4	5 - 3/4 top int col
5B	48	8	3	3/4	4	3/4	2 - 3/4 top ext col 5 - 3/4 top int col
6B	48	8	3	3/4	2	3/4	2 - 3/4 top int col
7B	18	8	2	3/4	1	3/4	2 - 3/4 top both cols
8B	18	8	2	3/4	1	3/4	
9B	18	8	2	3/4	1	3/4	2 - 3/4 Top both col extend across top 8B
10B	18	8	2	3/4	1	3/4	2 - 3/4 top ext col 3 - 3/4 top int col
12B	24	8	2	3/4	2	3/4	2 - 3/4 top ext col 3 - 3/4 top int col
13B	24	8	2	3/4	2	3/4	2 - 3/4 top int col
16B	18	8	2	3/4	2	3/4	



Typical Beam Bending

SLAB SCHEDULE						
slab mark	direction	bar size	band A	band B	band C	Remarks
1	NS	1/2				@10" bot
	EW	1/2				@12" top
2	NS	1/2	11	7 1/2	7 1/2 W	
	EW	1/2	12	10 1/2	12	
3	NS	1/2	12	10	12	
	EW	1/2	12	12	12	
4	NS	1/2	10	7	7W	
	EW	1/2	12	11	11W	

W denotes band adjacent to wall



Typical Slab Bending

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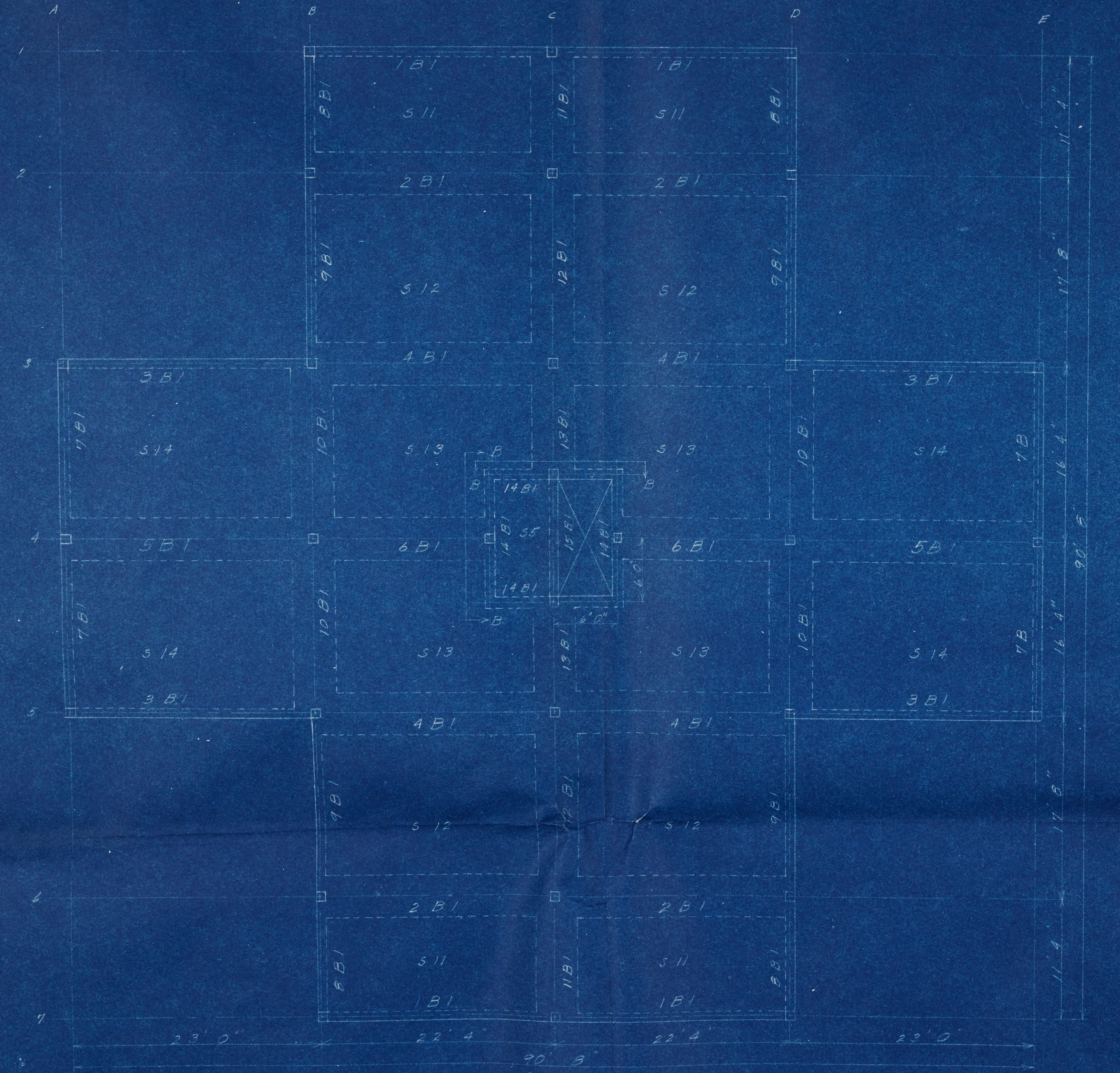
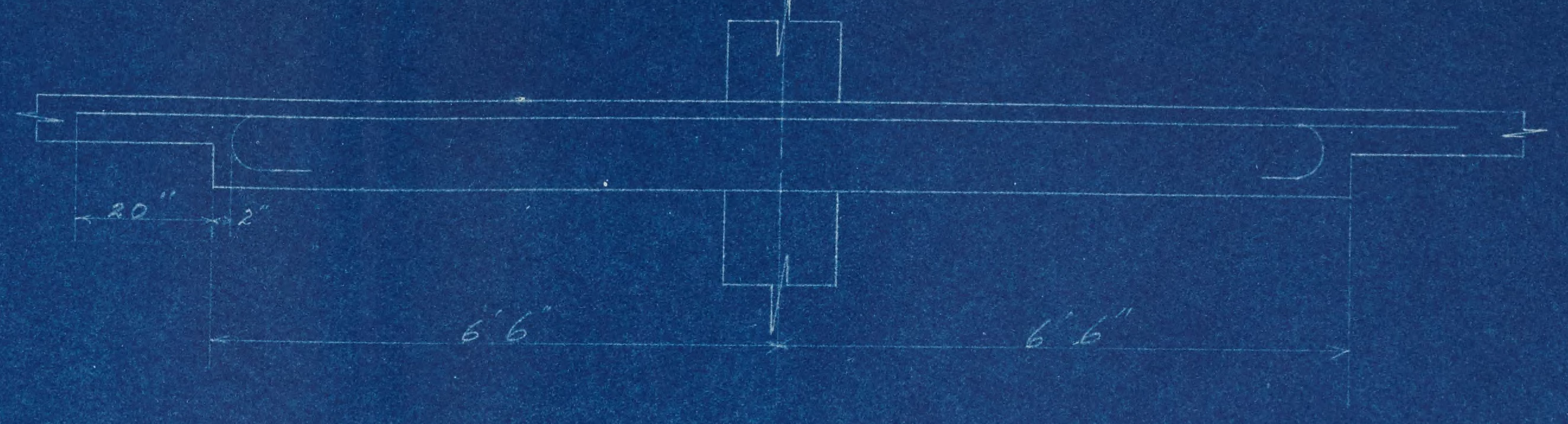


1st & 2d Floor Plan
Scale: 1" = 1/8"



Section BB

BEAM SCHEDULE

beam mark	Size W	Size D	Reinforcing str.	truss	stirrups	remarks
1B1	10	14	4		col B1 2@4, 1@6 col C1 2@4, 8@6	3 top ext col 3 top int col
2B1	4 8	8	7	3		6 top ext col 4 top int col
3B1	10	14	2	2	col A3 2@4, 1@6 col B3 2@4, 8@6	1 top ext col 4 top int col
4B1	4 8	8	2	4		4 top ea col
5B1	4 8	8	6	2		7 Top ea col
6B1	4 8	8	2	4		1 Top int col
7B1	10	14	2	1		2 Top ext col 1 top int col
8B1	10	14	2	1		2 top run clear across
9B1	10	14	2	1		2 top ea end run truss across top 3B1
10B1	4 8	8	3	3		2 top int col
11B1	4 8	8	3	2		2 top ext col run clear across
12B1	4 8	8	3	2		2 top ext col 4 top int col
13B1	4 8	8	4			2 top ext col 4 top int col run 2 bars clear across
14B1	12	8	4			hook 2 extend 2 into floor slab
15B1	10	8	2	2		2 top ea col

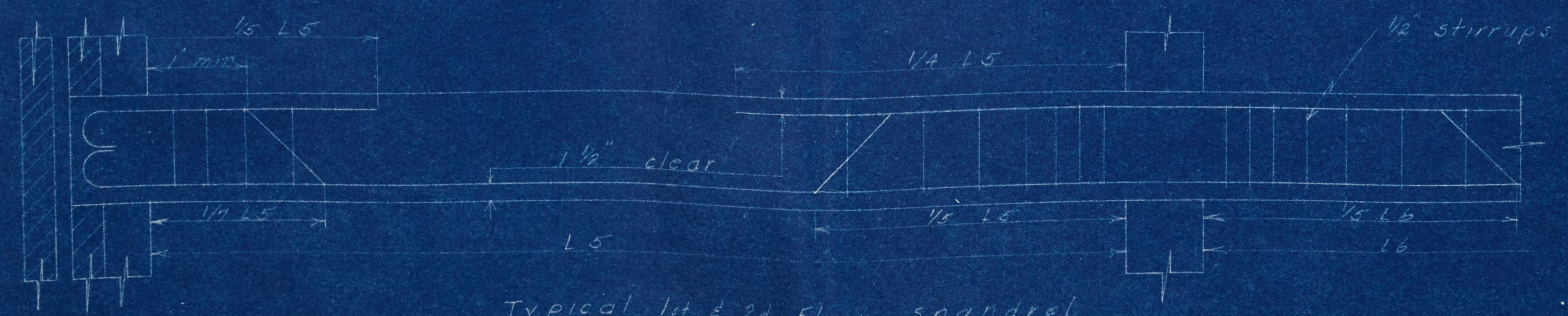
All main steel is 3/4" Φ deformed bars intermediate grade
Stirrups are 1/2" Φ deformed bars intermediate grade

Notes
1. For typical interior beam bending & typical slab bending see page 1.
2. All slabs are 4" thick
3. All concrete to have 3000 psi compressive strength in 28 days

SLAB SCHEDULE

slab mark	direction	bar size	band A	band B	band C	Remarks
11	NS	1/2				10" bot
	EW	1/2				12" top
12	NS	1/2	10	7 1/2	7 1/2 W	
	EW	1/2	12	10 1/2	10 1/2 W	
13	NS	1/2	12	9	12	
	EW	1/2	12	12	12	
14	NS	1/2	9	6	6 W	
	EW	1/2	12	10	10 W	
15	NS	1/2				10" bot
	EW	1/2				12" top

W denotes band adjacent to wall



Typical 1st & 2d Floor spandrel

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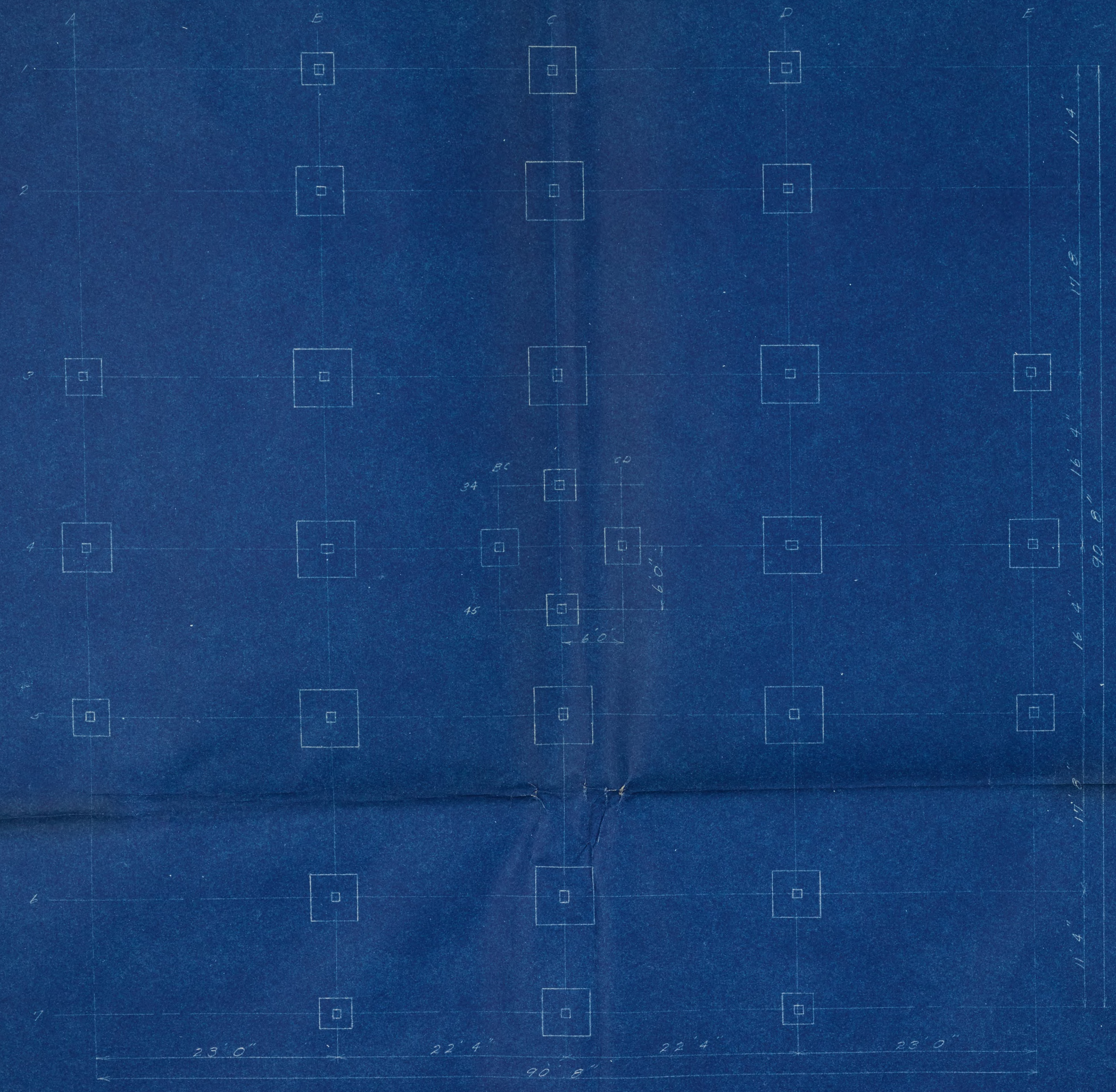
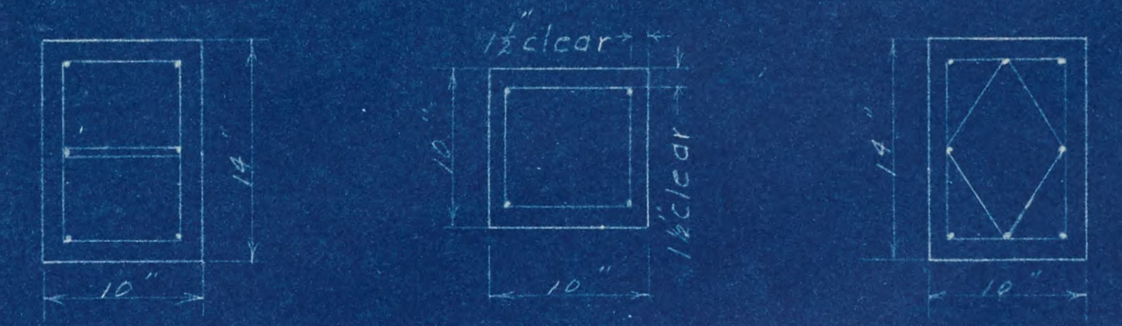
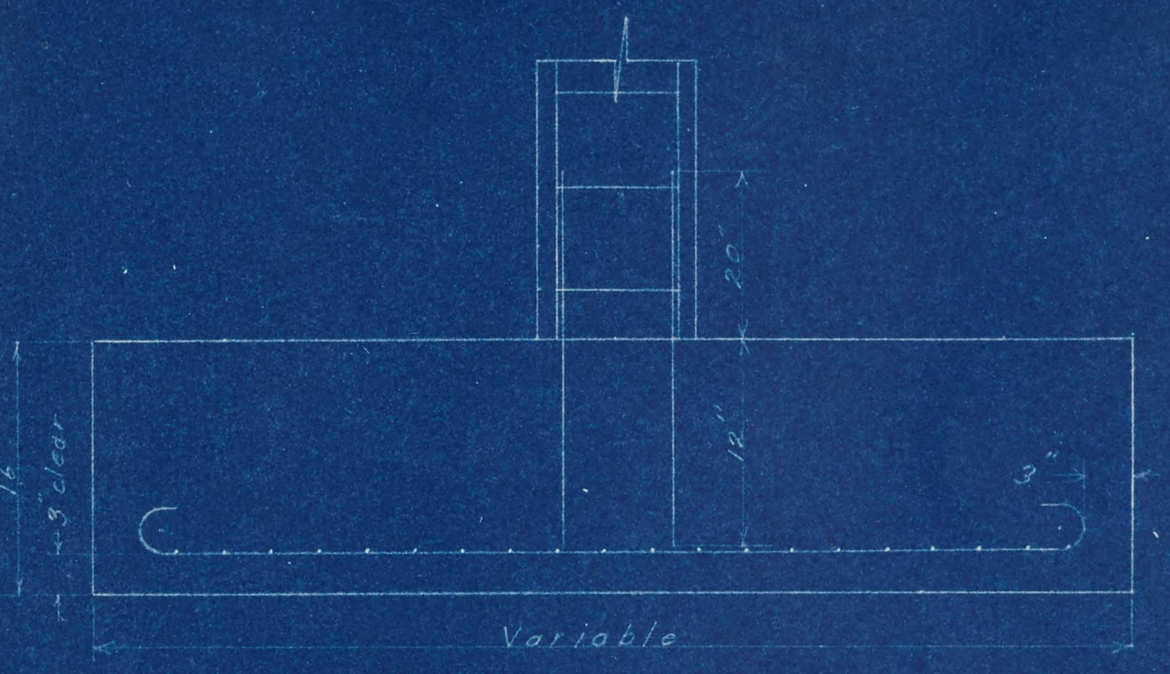


Foundation Plan
scale: 1/8" = 1'

Typical Column Ties





Typical Footing

Column & Footing Schedule							
Mark	A3, A5, E3 E5, BC4, CD4	A4 E4	B1, B7, D1 D7, C34, C45	B2, B6, D2 D6, C1, C7	B3, B5 D3, D5	B4, D4	C2, C3 C5, C6
roof — Size	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10 x 10
Vert steel	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4
Ties	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12
2d floor — Size	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10 x 10
Vert steel	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4
Ties	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12
1st floor — Size	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10" x 10"	10" x 14"	10" x 14"
Vert steel	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	8 - 3/4	6 - 3/4	8 - 3/4
Ties	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12	1/4 @ 12
Bsmt floor — Dowels ftg top	4 - 3/4	4 - 3/4	4 - 3/4	4 - 3/4	8 - 3/4	6 - 3/4	8 - 3/4
Ftgs Size	3'9" x 3'9"	4'9" x 4'9"	3'0" x 3'0"	4'6" x 4'6"	5'6" x 5'6"	5'6" x 5'6"	5'6" x 5'6"
Ftgs Steel - 1/2 E.W.	12 - 3/4	16 - 3/4	10 - 3/4	16 - 3/4	20 - 3/4	20 - 3/4	20 - 3/4

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