DESIGN OF WIND BRACING FOR
EDSON HOTEL BUILDING
BEAUMONT, TEXAS.

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By

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PREFACE

The importance of properly designing the frames of tall buildings to safely withstand the lateral pressure exerted by wind is generally recognized, but opinions of engineering authorities differ regarding the distribution of stresses created by wind pressure. Architectural treatment usually prevents the use of diagonal bracing in vertical planes, making it necessary to provide stability against wind pressure by rigid connections between beams and columns. A building frame of this type is indeterminate and approximate methods of analysing the stresses must be used..

It is the purpose of this paper to present the design of wind bracing for the Edson Hotel building to be erected in Beaumont, Texas, a city near the Texas coast subject, at times, to winds of hurricane intensity. The subject is treated from the practical standpoint of the engineering office where methods of analysis and design must be rapid, practical and reliable. The subject of wind bracing in general is discussed only to the extent that it bears on this particular building.

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WIND VELOCITIES AND PRESSURE

Before designing a building to resist wind pressure some assumption must be made regarding the maximum velocity of wind that may occur, and the resulting lateral pressure upon building walls.

It is not definitely known what velocity wind may attain during a hurricane or tornado. During the storm at Galveston, Texas in 1900, the anemometer at the government weather station registered a velocity of one hundred twenty miles per hour before breaking. A similar velocity was recorded at Houston, Texas in 1925. During the Florida storm of 1926 an anemometer at Miami Beach registered one hundred twenty eight miles per hour before being blown away. It is reasonable to expect similar storms at any city along the coast of the Gulf of Mexico. Professor Morris in his article on Pracitical Design Of Wind Bracing gives a list of corrections to apply to the velocities indicated by the anemometer to obtain true velocities, showing that an indicated velocity of one hundred thirty miles per hour the true velocity would be ninety nine miles per hour.

There are various formulas for computing wind pressure from wind velocity. The United States weather bureau has adopted the following formula:

$$P = .004 \text{ (V squared)}.$$

in which P is the pressure in pounds per square foot and V is

THE FLORIDA STORM OF 1926.

The Florida Storm of September 1926 has furnished opportunity for study of high buildings subject to pressure of intense winds.

A condensed report of a special committee of the Structural Division, American Society of Civil Engineers, was published in The Engineering News Record of March 1, 1928. Among buildings studied by this committee were The Meyer-Kiser Building, The Realty Board Building and The Daily News Building. A report by E. A. Stuhrman of the repairs to the first two buildings was published in the Engineering News Record December 29, 1927. The Meyer-Kiser Building suffered the greatest damage of any of the high buildings. It was a seventeen story building, 45 feet wide at front and 39 feet wide for the rear 105 feet of building, total length being 140 feet. This building was bent about 2 feet out of plumb Westward at the front or South face between the third and tenth floors. The rear, or North face, was about 8 inches out of plumb Eastward, and the whole building leaned about 6 inches to the rear. The upper ten stories have been removed and the building is now only seven stories high. Wind bracing consisted of ½ inch thick clip angles at top and bottom of beams. These clip angles failed, some bending and some breaking. Columns were built up of plate and angle sections and failed by bending above and below the beams.

Professor Morris, a member of the committee, has computed

that it would require about 60 pounds per square foot wind pressure to bend the columns in the front wall of the Meyer Kiser Building. Other factors aside from wind may have influenced the columns such as the possible failure of the walls, thus permitting the building to sag quickly to the leeward, applying a blow of knknown force to the columns, rendering calculation of wind pressure of doubtful value. Computations by the committee indicate that the steel frame, based on number and size of rivets, and an elastic limit of 36000 pounds per square inch in steel, should have resisted a wind pressure of 15 pounds per square foot, but that the clip angles were so thin that resistance to bending they did not develop more than twenty five per cent of this amount.

The Realty Board Building was severely damaged during the storm, walls and partitions were badly cracked, but the steel frame work was not seriously distorted, indicating that damage was due for the most part to lack of stiffness rather than lack of strength in the steel frame.

The Daily News Building has a tower 40 feet square and about 255 feet high. This building was designed to resist a wind pressure of 20 pounds per square foot in each direction at a unit stress of 24000 pounds per square inch in the steel work. Details were carefully designed and special effort was made to have the connections stiff as well as strong. This building suffered no structural damage during the storm.

The committee reached the following conclusions regarding wind bracing:

"The main lessons to be learned from the foregoing considerations are:

1. Adequate wind bracing is necessary in the construction of a building.

2. The details must be carefully designed for the same strength as that used in design of main members.

3. In designing different types of wind braces for one floor level the designer should take the relative stiffness of the members into account and not depend too much on the arithmetical summation of the total strength of the connections, as some members may be greatly overstressed before the others take any large proportion of the stress. In buildings of moderate size, however, it is permissible to take care of this point by the use of an adequate unit wind pressure.

4. Stiffness as well as strength is a prime requisite for good steel design. This is especially realized by our best engineers in designing narrow and high buildings used for living quarters, such as hotels and apartment houses, where the confidence of tenants in the stability of the structure is of the utmost importance.

5. The effects of the Florida hurricane indicated that the floor system acted as a stiff plate or horizontal girder

and that all columns were subject to the same horizontal deflection, but the entire structure twisted where one end of the building was stiffer than the other.

In designing wind bracing this must be given full weight and the bracing arranged, if possible, so that no twisting action from the wind can come into play. This means that the wind pressure at one end of a building and the bracing designed to resist it must balance the relation between the corresponding pressure and bracing at the other end, so that the structure will deflect about the same amount at both ends. This principle applies especially where one end is higher or narrower than the other."

The committee further concluded that the results of the storm showed that for buildings of the size and shape found in Miami, the common methods of analysis, as given by Fleming, will produce structures capable of standing up under the most severe conditions, but buildings so designed will not necessarily be sufficiently stiff to give tennants confidence in their safety. Moreover, in case of very high or narrow structures, the obvious errors involved render results less secure than is desirable.

FACTORS EFFECTING STABILITY

A number of factors contribute to the stability of a building against lateral pressure in addition to the strength of the frame.

The extreme velocities of wind are of very short duration. It is shown by weather bureau reports that maximum velocities vary as much as 35 percent from the mean velocity during a five minute period. It is obvious that the statical inertia of a building will offer considerable resistance to the pressure exerted by gusts.

Partitions, walls and floor construction stiffen a building and add stability against wind pressure though the exact amount of pressure thus resisted is problematical and will vary with different buildings.

Building Codes generally recognize these factors and do not require any special provision for wind resistance in buildings, the height of which does not exceed some fixed proportion of the least horizontal dimension. For example, The New York code requires that buildings over 150 feet in height and of a height more than four times the minimum horizontal wind pressure of 30 pounds per square foot.

A building frame is indeterminate in so far as lateral stresses produced by wind are concerned. Several attempts have been made to produce a workable method of exactly analyzing wind stresses, but to date the nearest approach to an exact method, that is also workable, is the slope deflection method of Professors Wilson and Maney, published in "Wind Stresses In The Steel Frames Of Office Buildings," Bulletin No.80, Engineering Experiment Station, University of Illinois.

This method is generally conceded to be the most accurate of any of the approximate methods. The method is long and cumbersome. Before applying the method a preliminary design is necessary, then the stresses are calculated. For a bent of a building twenty stories in height and three bays wide, this method would require setting up and solving sixty simultaneous equations containing sixty unknowns. This would then give the stresses in the preliminary design which would be corrected, and the bent refigured, for any change in size of beams or columns will cause a new distribution of stresses. This process would be repeated a number of times before the columns and beams would be properly proportioned. While this method is not practical for use in design it is valuable as a guide to the accuracy of other approximate methods.

In the May 1928 Proceedings of the American Society of Civil Engineers, Albert Ross and Clyde T. Morris present an approximate solution of wind stresses based on slope deflection methods of Wilson and Maney. This method, it is asserted, is practical and is more nearly in accord with true stresses than any other approximate method now in use except the Wilson and Maney method. First a preliminary analysis is made according to certain assumptions as to the relative sizes of members which would produce zero direct stresses in interior columns and keep points of contra-flexure of the members at their mid points. This bent is called "theoretically

proportioned." If the relative sizes of the members are as assumed in the alalysis, the stresses calculated upon the above assumptions will be correct. Due to gravity loads, in addition to wind loads, it will seldom accur that the relative sizes of the theoretical proportioned bent can be maintained. A chart is furnished from which the errors in the shears in the girders may be found for any probable variation in sizes. With the errors in girder shears known, the stresses in the other members may be corrected.

The best known and most widely used approximate methods are known as Fleming Number 1, Number 2, and Number 3.

Method Number 1, known also as the cantilever method, makes the following assumption:

1. The points of Contra-Flexure of the columns are at their mid heights.

2. Points of Contra-Flexure of the girders are at their mid lengths.

3. Direct stresses in columns are directly proportioned to their distances from the neutral axis of the bent.

This method assumes the bent of a building to be a cantilever beam with rectangular openings cut in the web. What was the horizontal shear in the beam will now be a shear at the point of contra-flexure of the floor beams. What was vertical shear in the beam becomes shear at the points of contra-flexure of the columns.

This method obviously departs from exact analysis in that

floor beams of relative light section will be subject to different deformations from the web of a homogeneous beam.

Fleming Method Number 2, makes the following assumptions:

1. The points of contra-flexure of the columns are at their mid heights.

2. The shears in all the columns of a story are equal.

3. The direct stresses in the interior columns of a bent are zero.

A modification of Fleming Method Number 2, known as the Portal Method makes the same assumption except that the structure is considered equivalent to a series of independent portals. The total horizontal shear on any plane is divided by the number of bays. For unequal spacing of columns, the horizontal shear is divided among the bays in proportion to the span. Thus, in all cases all the direct stress from overturning is taken by exterior columns. This method is open to the objection that it assumes a condition of independent portals that in reality cannot exist in a rigidly connected frame.

Fleming Method Number 3, makes the following assumptions:

1. The points of contra-flexure of the columns are at their mid height.

2. The shears in all the columns of a story are equal.

3. The direct stresses in the columns are directly proportional to their distances from the neutral axis of the bent.

John C. Van Der May, and Felix H. Spitzer in the Engineering

News Record of January 19, 1928 present a method of computing stresses in columns due to lateral pressure by use of deflections in columns. It is assumed that all columns deflect equally and that the floor system and connections are absolutely rigid. Effective length of columns is considered between gusset plates. This method has merit in so far as it recognizes that increasing the moment of inertia of one column or shortening its effective length will increase its proportion of the total horizontal shear. However, the assumption that there is no deformation in floor beams is in error. The change in slope of the columns connection due to deflection in connecting girder would in the writer's opinion cause a distribution of horizontal shears in the columns materially different from the distribution calculated by this method.

The method can be useful in conjunction with other methods as a guide to the distribution of stresses where clear lengths of columns differ in any story.

THE EDSON HOTEL BUILDING

General Description

Bids were received from general contractors about May 1, 1928. The low bids for the general contract and the mechanical contract together amounted to approximately one million two hundred thousand dollars.

Architects for the building are F. W. and D. E. Steinman, of Beaumont, Texas. Hedrick and Gottlieb Inc., of Houston, Texas are associate architects.

The building is to be constructed in the city of Beaumont, Texas. It is 120 feet by 120 feet at first floor, mezzanine and second floor. Above the second floor the building is 54 feet by 120 feet except for an additional roof area 57 by 66 feet at the third floor level. The building is twenty one stories in height at rear, 34 feet and twenty stores in height at front. The basement extends under the entire building and about ten feet under the side walk on two street sides.

Walls in general are 4 inche brick with 8 inch tile backing, although in some cases they are of 4 inch brick with 12 inch tile backing.

Partitions are of 3 inch clay tile.

The building frame is of structural steel encased in concrete fire proofing. Beams not framing into columns are generally of reinforced concrete affording a saving in cost over structural steel beams with concrete encasement.

The first, second, eighteenth, nineteenth and twentieth floors are of beam and slab construction. The mezzanine floor is of clay tile and concrete joist construction in order to maintain a flat ceiling below. The typical floors third to seventeenth inclusive are of clay tile and concrete joist construction in the outer bays, and beams, and slab construction in the center bay.

The joist construction was used in the outer panels to maintain flat ceilings in the rooms. The beam and slab construction was adopted for the center bay on account of numerous and irregular openings required in the slabs for plumbing in the bath rooms. A corridor is located in the center of the middle bay and the bath rooms are on each side of the corridor. Plate Number 10, shows the framing for the typical floors.

Footings under the high part of the building are supported on concrete piles spaced 3 feet on centers. Footings under the low part of the building are supported by wood piles 2 feet 6 inches on centers.

Columns Numbers 13 and 14 are supported on steel girders at the second floor.

The building was designed for the following live loads:

First Floor	- 125	pounds	per	square	foot
Mezzanine "	75	"	"	"	"
Second Floor of Main Building	30	"	"	"	"

Second Floor, Roof Garden	75	pounds per square foot
Third to Seventeenth Floors, inclusive	30	" " " "
Eighteenth Floor	75	" " " "
Nineteenth & Twentieth	50	" " " "

In addition to the live loads a partition load was used on the second to seventeenth floors of 30 pounds per square foot in outer bays, and 60 pounds in the panels adjacent to the corridor.

Due to the presence of ground water a 12 inch slab was designed for the basement.

The following stresses were used in the design of the building:

Reinforced Concrete

Extreme fibre stress in bending	750	pounds per square inch
Diagonal Shear	40	pounds per square inch on area b x d
Punching Shear	120	pounds per square inch
Reinforcing steel in Tension	18000	" " " "
Bond stress	125	" " " " for one way reinforcing.
Bond stress	100	pounds per square inch for two way reinforcing.
Structural Steel extreme fibre stress in bending	18000	pounds per square inch
Extreme fibre stress in columns	15000	pounds per square inch where length divided

for the least radius of gyration is 60 or less.

Working stress for other columns as determined by column formula of The American Institute of Steel Construction.

Rivets in tension	12000 pounds per square inch
Rivets in single shear	12000 pounds per square inch
Bearing value concrete piles	30 tons
Bearing value wood piles	20 tons

For members subject to stress from wind, the allowable stress was increased fifty percent for the combined stresses but in no case was the stress from gravity loads alone allowed to exceed the regular working stresses.

The following reductions of live load on columns were made:

At Eighteenth	floor	15%
" Seventeenth	"	20%
" Sixteenth	"	25%
" Fifteenth	"	30%
" Fourteenth	"	35%
" Thirteenth	"	40%
" Twelfth	"	45%
" First to Eleventh		50%

The above reductions are for the total live load down to, and including the floors specified.

Gravity loads on columns are listed in Plate Number 4.

WIND STRESSES IN EDSON HOTEL BUILDING

The various factors influencing the amount of lateral pressure that a building frame must resist have been discussed. From this study it was decided that this building will be stable under action of the severest winds if the steel frame is properly designed to resist a lateral pressure 20 pounds per square foot, upon the entire wall surface above the third floor. It was further decided to use the Cantilever or Fleming Number 1 Method in analysing the stresses across the building. Plate Numbers 1 and 1A show the stresses computed by this analysis assuming all stress taken by the columns in the ends of the building.

A strict application of this method considers the direct stress in the columns to vary as the distance from the neutral exis and also with the sectional area of columns. In computing the direct stress in columns in the present case, the difference in size of columns was neglected. A later check of a typical floor showed a maximum error of seven percent in bending moments on this account. The moments in columns 30 and 31 would be increased and the moments in columns 29 and 30 would be decreased, had the difference in column sizes been considered. On the other hand, an examination of results by the Ross method indicates that the Cantilever method results in computes stresses higher in the center bay, and lower in the outer bay than will actually exist. Therefore, in this case a nearer approach to the true

stresses is obtained by neglecting the difference in column sizes.

As an example of the computations required for solution of the stresses consider the columns supporting the tenth floor. The overturning moment must be balanced by the direct stresses in columns acting about the neutral axis of the building.

Let direct stress in column 29 and 32 = 25.7x

" " " " " 30 and 31 = 10.3x

Then 5400 x 118.2 plus 11400 x 109.2 plus 12000 x 99.2x plus 14600 x 89.2 plus 14600 x 75 plus 12000 x (65 plus 55 plus 45 plus 35 plus 25 plus 15 plus 5) = 2 x (25.7x) X 25.7 plus 2 x (10.3x) X 10.3

OR

10.3x = 56,300 = direct stress in columns 30 and 31.

25.7x = 140500 = direct stress on columns 29 and 32.

The direct stress in the columns supporting the other floors can be computed in the same manner.

The vertical shear in a beam between column 31 and 32 will then be the difference between the direct stress in column 32 above and below the beam. The direct stress in a beam between column 30 and 31 will then be the shear in beam between columns 31 and 32 plus the difference between the direct stress in column 30 above and below the beam.

For example, refer to Plate Number 1A. Consider the tenth floor. Direct stress in column 32 below the tenth floor is

140.5 kips and above the tenth floor 118 kips. The vertical shear then in the beam between columns 31 and 32 is 140.5 minus 118 equals 22.5 kips.

The direct stress in column 31 below the tenth floor is 56.3 kips and above the tenth floor is 47.3 kips. The vertical shear in the beam between columns 30 and 31 will be 22.5 plus 56.3 - 47.3 = 31.5 kips.

the horizontal shear in columns now remains to be found. Consider the tenth floor connection at column 32.

Total horizontal shear under tehth floor equals 142000

" " " " eleventh " " 130000

Let X equal horizontal shear on column 32 under tenth floor.

Then $\frac{130000}{142000} X = .915x$ equals Horizontal shear on column 32 under eleventh floor

And X minus .915x equals .085x equals increment of shear at the tenth floor.

Moments of shears must hold the joint in equilibrium.

Taking moments about the mid point of column 32 below the tenth floor we have

(.915x)10 plus (.085x)5 equals225000 x 7.75

from which x	equals 18200	equals horizontal shear on column 32 under the tenth floor
and .915x	equals 16700	equals horizontal shear on column 32 under eleventh floor

Similarly taking moments about the mid point of column 31 we have

(.915 x) 10 plus (.085 x) 5 equals 225000 x 7.75 plus 31500 x 10.3

from which x equals 52000 equals horizontal shear on column 31 under tenth floor.

.915 x equals 47500 pounds equals horizontal shear on column 31 under eleventh floor.

The balance of the horizontal shears may now be found by proportion.

Having found the shears in columns and beams, the bending moments may be found by multiplying the shears by one half the beam or column length.

Slide rule calculations were used in computing the stresses and there will therefore, be some slight error.

No stress diagram was made for columns 1 to 4, but the stresses were calculated in the foregoing manner for this bent at the seventh floor with the following results:

Moment	in	column	1	equals	moment	column	32 x .845
"	"	"	2	"	"	"	31 x1.01
"	"	"	3	"	"	"	31 x1.07
"	"	"	4	"	"	"	32

Moment in beam column 1 to 2 equals moment in beam column 31 to 32 x .86

Moment in beam column 2 to 3 equals moment in beam column 30 to 31 x 1.11

Moment in beam column 3 to 4 equals moment in beam column 31 to 32 x 1.02

Design of wind bracing column 1 to 4 was made using these proportions.

Plates Numbers 2 and 2A show wind stresses from columns 1 to 32. The portal or modification of Fleming method number 2 of analysing stresses was used. This method has the advantage that for equal spacing the moments in beams are the same for all bays, permitting duplication of details. In as much as the height of the building in this direction is only twice its horizontal dimension, the selection of method of analysis is not of great importance, and any of the recognized methods will give satisfactory results. For the purpose of computing stresses equal spacing of columns was assumed introducing an error of about 3%.

Direct stresses being all taken by the exterior columns, the overturning moment is equated to (X) x the length of the building (X) being the direct stress in an exterior column. The vertical shear in all beams will then be the difference between the direct stress in the exterior column above, and below the floor. The horizontal shear on any interior column will be, for equal column spacing, the total horizontal shear divided by the number of bays and the shear on an exterior column will be one half that amount.

Plate Number 3, shows the wind stresses from column 4 to column 29.

This bent has unequal column spacing due to column 13 being supported at the second floor. It was necessary for

architectural reasons to consider the first and mezzanine floors as one story, in designing the wind bracing.

DESIGN OF COLUMNS

Calculated wind stresses in columns have been reduced, considering the clear height 2 feet less than the story to story height. Full advantage of the shortened effective length of columns has not been taken for the reason that the effective lengths of columns vary in the same story causing the columns to take different proportions of the total horizontal shear from that figured. If full advantage of the shortened effective lengths of columns is taken there is danger of some columns being over stressed.

In designing columns a column section is first assumed, and then the extreme fibre stress calculated for the combined loading.

$$f \text{ (direct) equals } \frac{P}{A}$$

$$F \text{ (bending) } \text{"} \quad \frac{M}{S}$$

In which f equals extreme fibre stress in pounds per square inch

p equals total axial load on column

a equals area of column section in square inches

m equals bending mement in inch pounds

s equals section modulus

Plate Number 4 lists the column loads due to gravity.

Plate Number 5 is a schedule of the columns resisting wind stresses.

DESIGN OF BEAMS AND BRACKETS

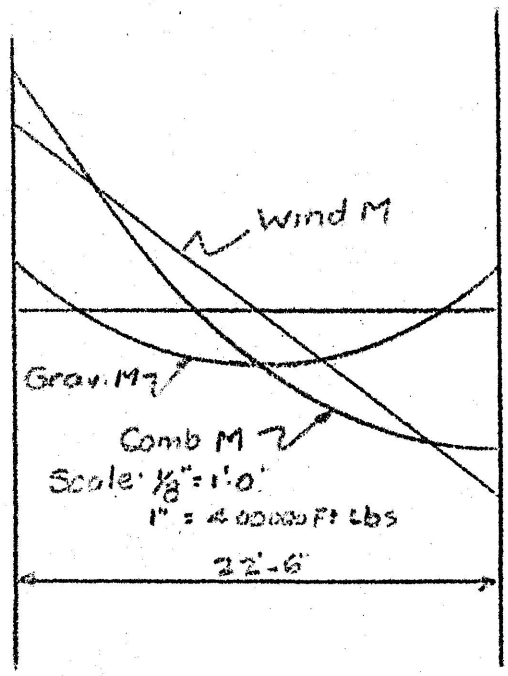
Beams are subject to combined bending moments due to gravity loads and wind. Both negative and positive moment due to gravity have been considered as $\frac{WL}{12}$ for figuring the combined moments, and the positive moment has been considered as $\frac{WL}{8}$ for gravity loads only. It is common practice to neglect the gravity negative moment when designing wind connections. This is reasonable in as much as the beam is rigidly connected at both ends and the columns are deflecting equally, and the bending moments in the columns must be balanced by the moments in the beams. All connections in the Edson Hotel Building however, have been designed on the basis of the combined moment at the column face being gravity $\frac{WL}{12}$ plus Wind Moment. This is on the side of safety and results in a stiffer connection which is very desirable.

It is also common practice to design connections of beams to columns not in the building walls, for a certain proportion of the wind acting upon them assuming that the balance of the wind pressure is carried to the ends of the building. This is open to the objection that if these connections are rigid the columns will be overstressed before any portion of the wind pressure can be taken to the ends of the building.

In the present case the 20 inch beams in the center bay of the building from the first to seventeenth floors are connected to columns with standard connections plus a 15 inch 33.9 pound channel clip top and bottom, each with four rivets into

the columns. This connection is capable of developing about 80,000 foot pounds in moment. Experiments prove that a connection of this type is not rigid in comparison with gusset plates, and it has therefore, been considered safe to allow these connections to take 30,000 foot pounds of moment allowing the balance of the wind pressure to be carried to the ends of the building. This permits a reduction in the stresses shown on Plates Numbers 1 and 1A, varying from 5% at the first floor to 30% at the seventeenth floor.

As an example of the design of girders and brackets, consider the beam between columns 2 and 3 at the sixth floor. Plate Number 25, shows details for this bracing. Moments are plotted as in Figure 1.



Comb. Moment
Col 2 to Col 3
at 6th Floor

Figure 1.

The combined moment at the column face is seen to be 470000 feet pounds.

Since working stresses are increased 50% for combined wind and gravity loads the value of a 3/4 inch rivet in tension will be .44 x 12000 x 1.5 equals 7900 pounds.

Based on straight line variation of stress the rivet furthest from the axis of rotation may be stressed to this amount and other rivets a proportion of this based on their distance from the axis of rotation. Let these distances be denoted by X1 X2 X3..... XN, XN being the distance of the furthest rivet.

The resisting moment will then be

$$7950 \frac{X1}{XN} X1 \text{ plus } 7950 \frac{X2}{X17} \ldots\ldots$$

$$\ldots\ldots \text{ plus } 7950 \frac{XN}{XN} \text{ equals } \frac{7950 \text{ (sigma X squared)}}{X}$$

Assume that the gusset plate is connected to column by two lines of 24 rivets each spaced 2-1/4 inches on centers.

Since the plate is pulling on rivets at one end and push-against metal at the other end, the axis of rotation will be assumed near the compressive end, in this case at the third rivet from the end. The resisting moment of the rivets connecting the gusset to the column will be

$$\text{R.M. equals } \frac{7950 \times 33392}{47.25} \text{ equals } 5{,}630{,}000 \text{ in pounds.}$$

or 470,000 foot pounds, showing that the number of rivets selected is satisfactory.

The required moment of inertia the gusset plate will be

$$\frac{470000 \times 12 \times 25}{27000} \text{ equals } 5250$$

therefore, the web of a 30 inch 121 pound Bethlehem beam will be suitable and is selected in preference to a plate with angles because in this case it permits a smaller gage distance between lines of rivets connecting to collumn.

From the moment diagram it is seen that the moment 2'-6" from the column face 320,000 foot pounds.

Therefore, the girder must be designed for this amount.

Architectural treatment limits the depth of girder to 1 foot 7 inches.

Required moment of inertia of girder equals

$$\frac{320,000 \times 12 \times 9.5}{27000} \text{ equals } 1360$$

Assume a plate 18 inches x 1/2 inch, four angles 6 x 4 x 5/8 with short legs out. Then the moment of inertia of the girder selected equals 1 of PL equals 209

1 of LS equals 1525

1 of girder 1734

Spacing of rivets in girder and number of rivets required to connect girder to gusset are computed by the usual methods followed in girder design and will not be repeated here.

Where trusses are used in the wind bracing stress diagrams are drawn in the usual manner, with the gravity loads and in addition the wind shears at column faces and the couples obtained by dividing the wind moment by distance between the neutral axis of the top and bottom chords.

Wind bracing details are shown on Plates 11 to 31.

It has been quite a problem to design details that do not interfere with architectural treatment, and at the same time are practical for fabrication and erection.

Welding of structural steel will simplify wind bracing by eliminating the necessity for rivet clearances in connections. Tests prove that welding is efficient, but ~~until~~ it has not been proven in general practice sufficiently to warrant its adoption on a building of this character.

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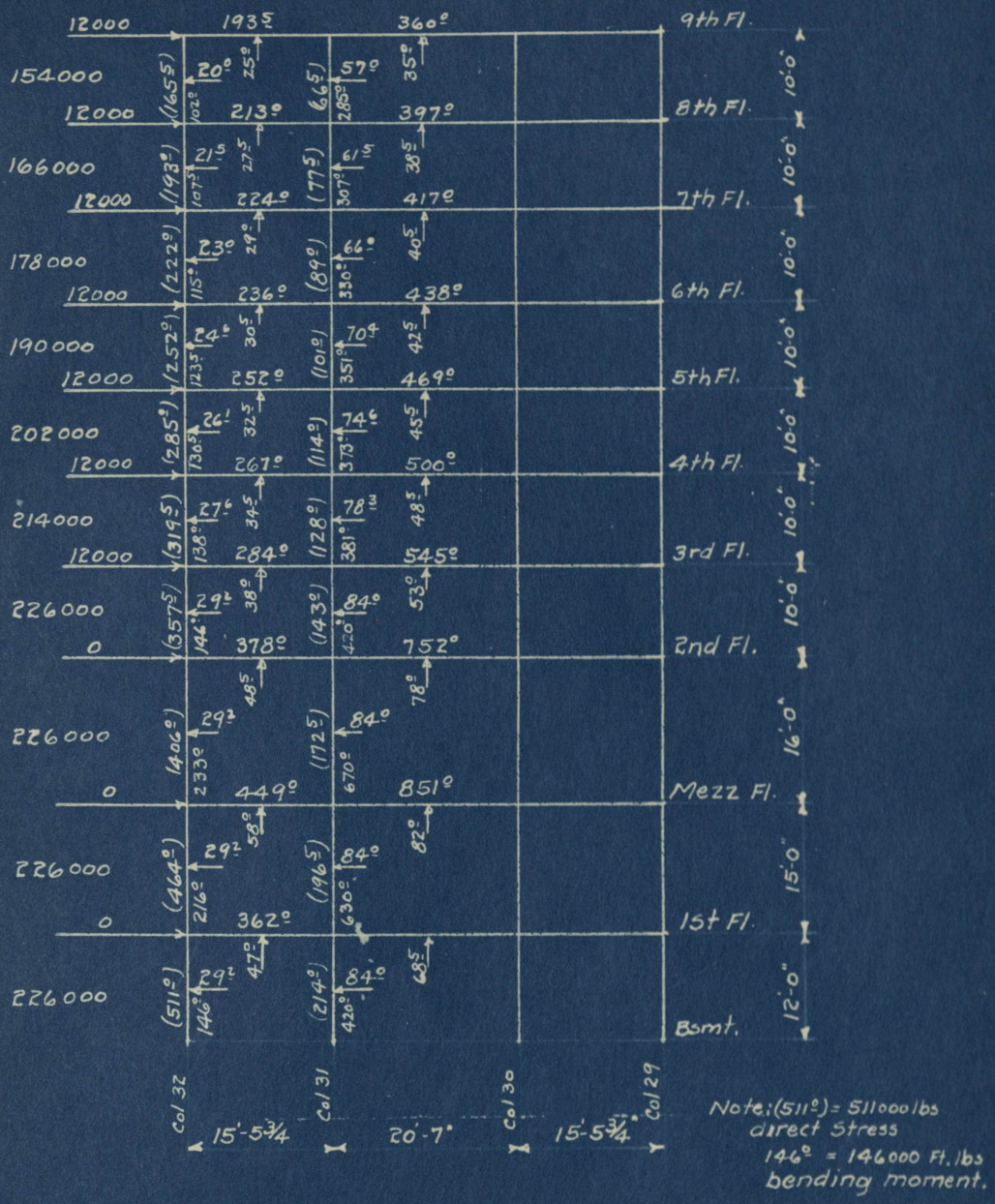
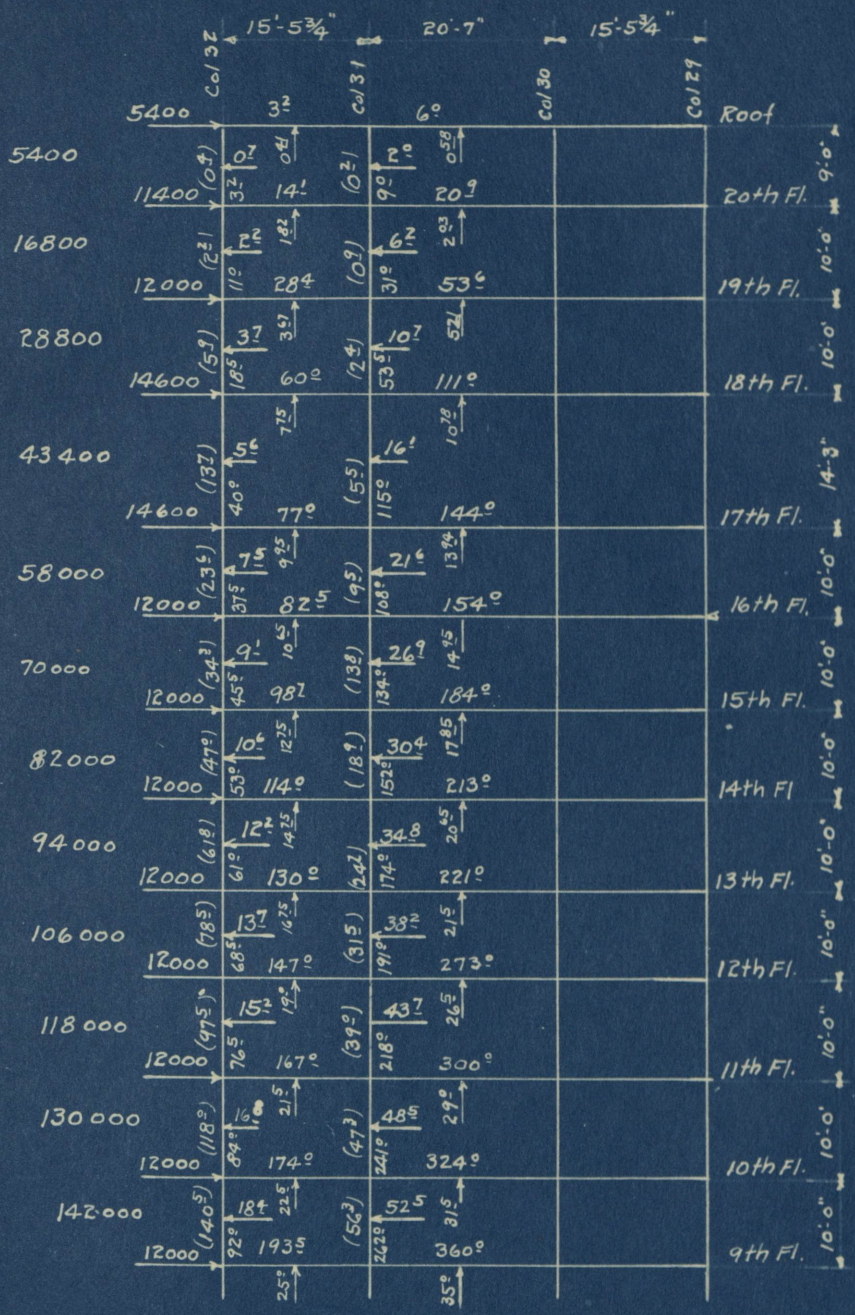


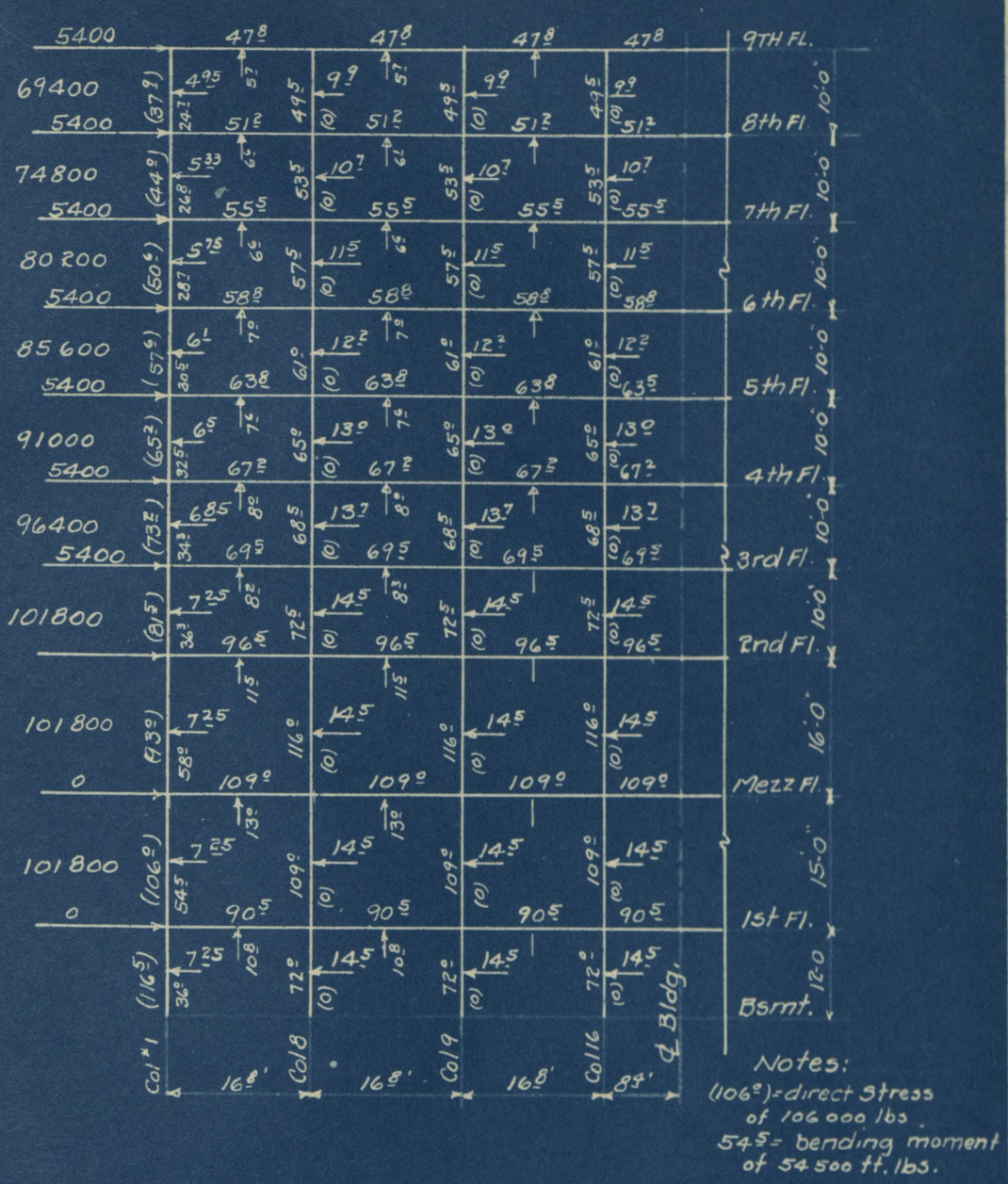
WIND STRESSES COL. 29 to COL 32.
CANTILEVER METHOD
1ST. to 9TH. FLOORS.

PLATE 1



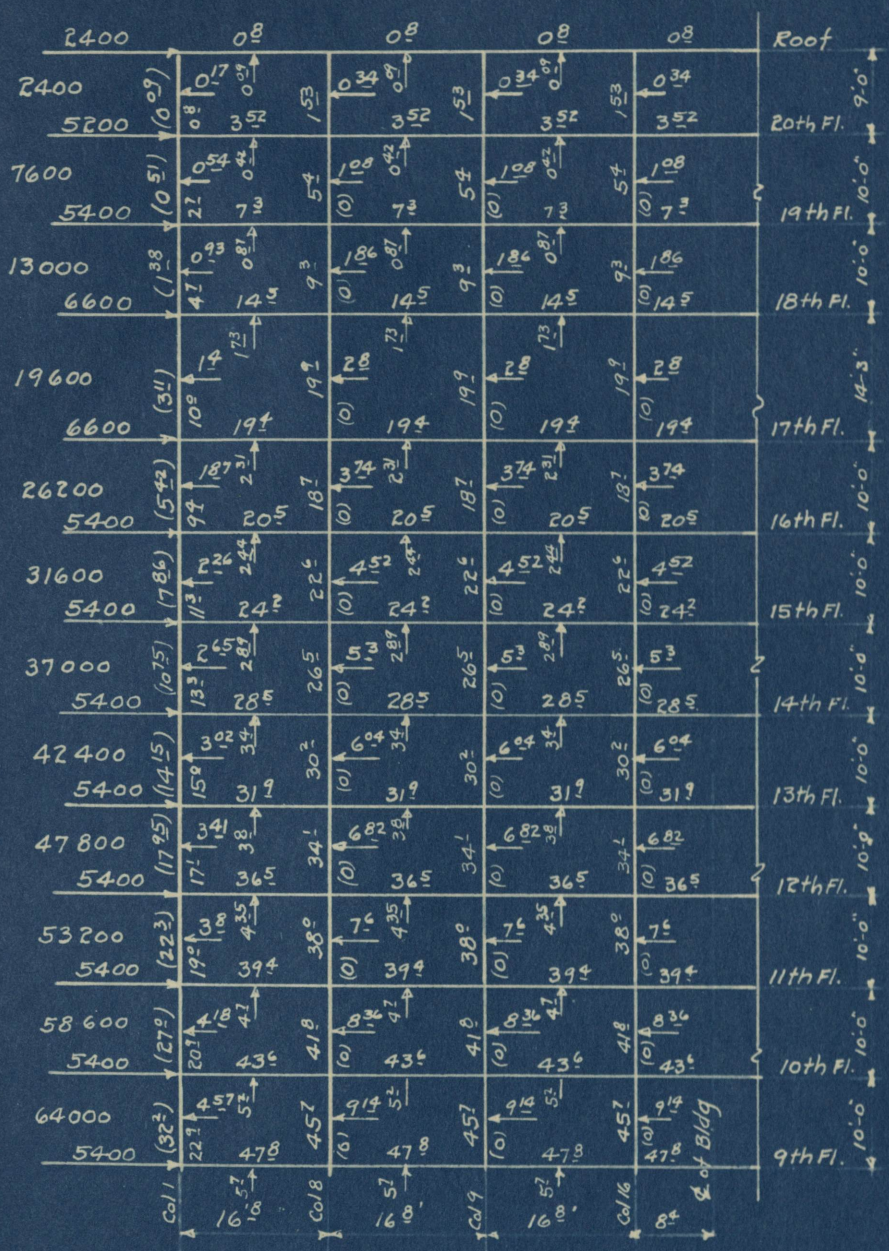
WIND STRESSES COL. 29 to COL 32.
CANTILEVER METHOD
9TH FL. to ROOF

PLATE 1A



WIND STRESSES COL 1 to COL 32
1ST. to 9TH FLOORS
PORTAL METHOD

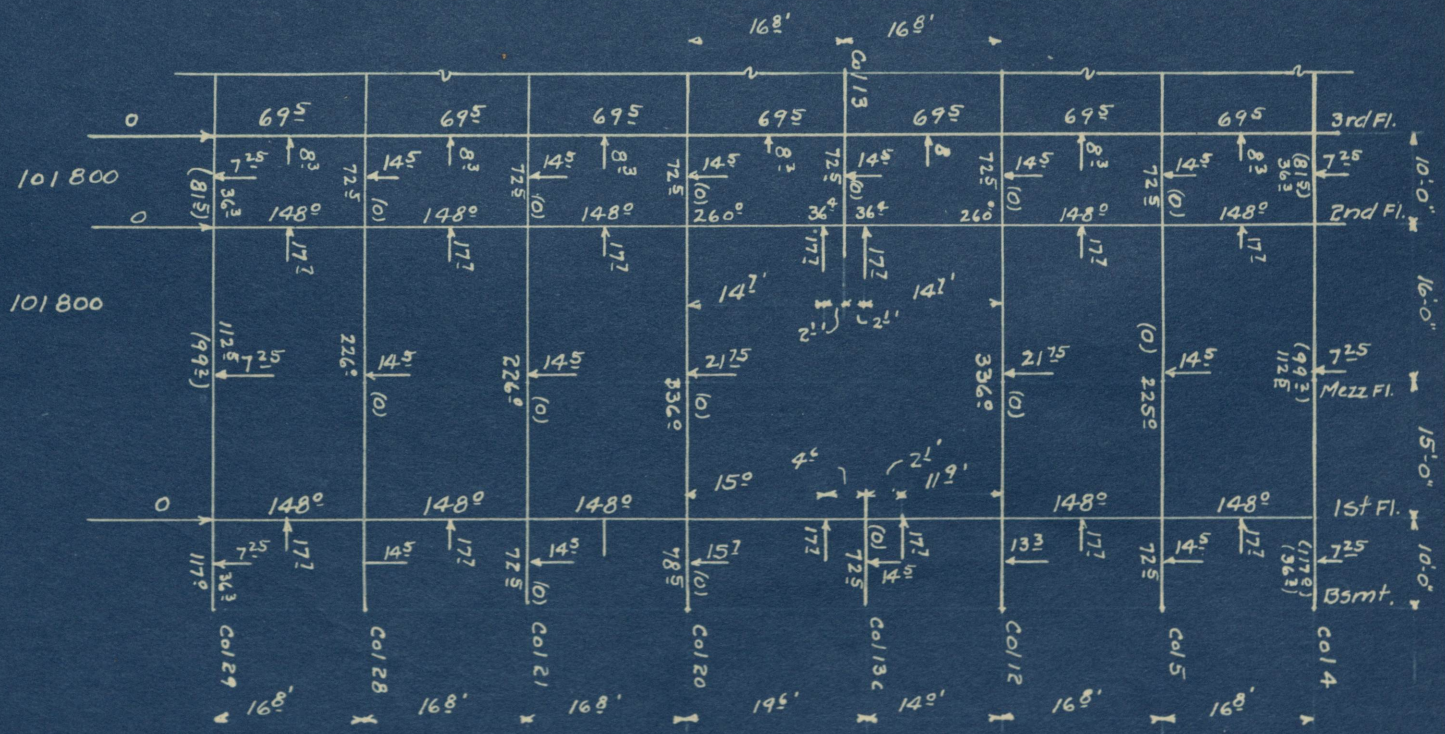
PLATE 2



WIND STRESSES COL. 1 to COL. 32
9TH FLOOR to ROOF
PORTAL METHOD

PLATE 2A

PLATE 3



WIND STRESSES COL 4 to COL 29 - PORTAL METHOD

Note: Stresses above 3rd Fl. same as Col. 1 to Col 32

PLATE 4.

GRAVITY COLUMN LOADS

FOR COLUMNS RESISTING WIND

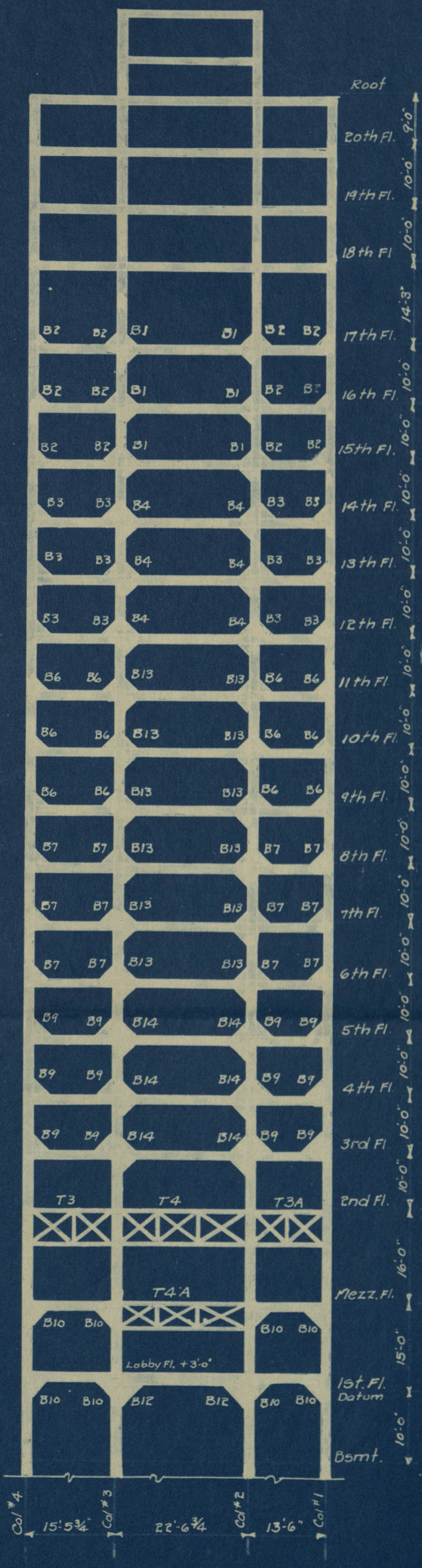
Loads are in Thousands of Pounds.
Live Load Reductions have been made

Col. No	1	2	3	4	5	8	9	12	13	16-17 24-25	20	21-28	29	30-31	32	13c
Supptg P.H Roof		27	27													
" P.H.Fl.		103	81													
" Roof	24	153	129	28	33	34	42	33	13		13		13	23	13	
" 20th Fl.	60	210	214	92	84	85	94	114	47		47		42	66	42	
" 19th "	99	254	268	134	177	129	124	143	-		-		-	-	-	
" 18th "	144	315	332	185	233	184	180	199	126	123	123	119	90	130	90	
" 17th "	178	363	383	222	280	228	224	244	170	168	167	162	126	179	126	
" 16th "	211	405	433	259	322	271	268	288	215	212	210	203	162	227	162	
" 15th "	243	452	483	295	366	313	310	332	258	256	253	246	197	274	197	
" 14th "	276	499	533	331	409	356	354	375	302	300	296	286	232	321	232	
" 13th "	309	539	582	367	452	397	396	418	345	343	338	328	267	368	267	
" 12th "	341	584	630	403	494	439	438	461	388	386	380	368	301	415	301	
" 11th "	374	632	682	440	539	483	483	505	430	429	422	410	335	460	335	
" 10th "	407	683	733	477	583	526	527	550	475	473	465	450	371	509	371	
" 9th "	440	723	784	503	628	570	569	595	519	518	509	494	406	557	406	
" 8th "	473	773	835	550	672	614	614	639	564	563	552	535	442	605	442	
" 7th "	506	821	887	587	717	657	658	684	609	608	595	578	477	653	477	
" 6th "	540	870	938	624	762	701	703	728	653	653	640	620	512	701	512	
" 5th "	573	919	990	650	807	744	747	773	698	697	683	665	548	750	548	
" 4th "	606	968	1041	707	850	788	792	818	742	742	728	706	583	798	583	
" 3rd "	639	1017	1092	752	900	832	837	903	807	786	776	747	619	846	619	
" 2nd "	679	1081	1152	810	966	875	883	1395		832	1288	814	676	899	657	
" Mezz "	720	1139	1221	891	1014	919	931	1422		878	1316	860	730	947	693	
" 1st	764	1204	1298	982	1104	998	1013	1493		951	1419	922	820	1031	765	85

COLUMN SCHEDULE OF COLUMNS WITH WIND BRACING

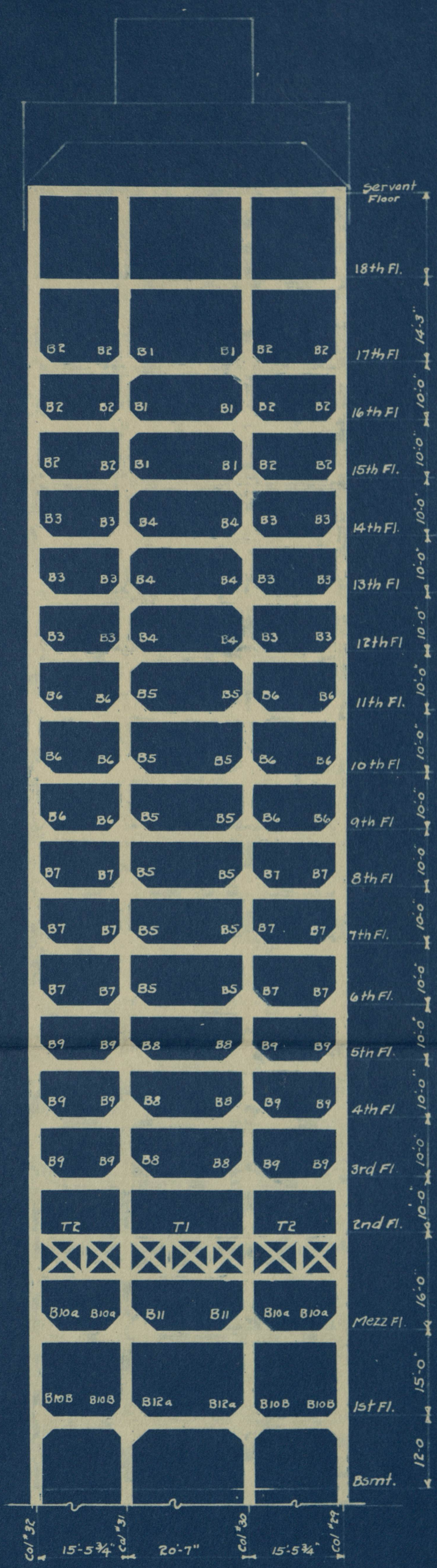
Col. No	P.H. Roof	P.H. Fl	Roof	20th Fl.	19th Fl.	18th Fl.	17th Fl.	16th Fl.	15th Fl.	14th Fl.	13th Fl.	12th Fl.	11th Fl.	10th Fl.	9th Fl.	8th Fl.	7th Fl.	6th Fl.	5th Fl.	4th Fl.	3rd Fl.	2nd Fl.	Mezz. Fl.	1st Fl.	Bsmt
2		8H35		10H60.5		12H79		14H100		14H115.5		14H147		16H177		16H203		16H238		16H265		16H342		16H356	
3		8H35		10H55		12H79		14H100		14H123.5		14H155		16H186		16H212		16H256		16H288		16H356		16H370	
1				8H32		10H49.5		10H55		12H72.5		14H84		14H107.5		14H131.5		14H155		16H177		16H230		16H238	
4				8H32		10H55		12H65.5		12H79		14H92		14H115.5		14H139		16H161		16H186		16H247		16H265	
5				8H32		10H60.5		12H79		12H97.5		14H115.5		14H139		14H155		14H177		14H194		14H227		14H254	
8				8H35		10H60.5		12H72.5		14H92		14H107.5		14H131.5		14H147		14H168		14H185		14H227		14H236	
9				8H32		10H49.5		12H65.5		12H85.5		14H100		14H123.5		14H147		14H161		14H185		14H227		14H236	
12				8H32		10H49.5		12H72.5		14H92		14H107.5		14H131.5		14H147		14H168		14H194		16H328		16H342	
13						8H32		10H55		12H72.5		14H92		14H115.5		14H131.5		14H155		14H185					
20						8H35		10H49.5		12H79.5		14H92		14H106		14H126.5		14H147		14H169		16H328		16H328	
21 28						8H35		10H49.5		12H65.5		14H84		14H107.5		14H123.5		14H147		14H161		14H219		14H219	
16-17 24-25								10H55		12H72.5		14H92		14H107.5		14H131.5		14H155		14H168		14H219		14H227	
29						8H35		10H49.5		12H65.5		14H84		14H100		14H123.5		14H147		16H169		16H230		16H238	
30 31						10H55		14H84		14H92		14H123.5		16H151		16H177		16H212		16H238		16H328		16H342	
32						8H35		10H49.5		12H65.5		14H84		14H100		14H123.5		14H147		16H169		16H230		16H230	
13c																								12H65.5	

PLATE 5



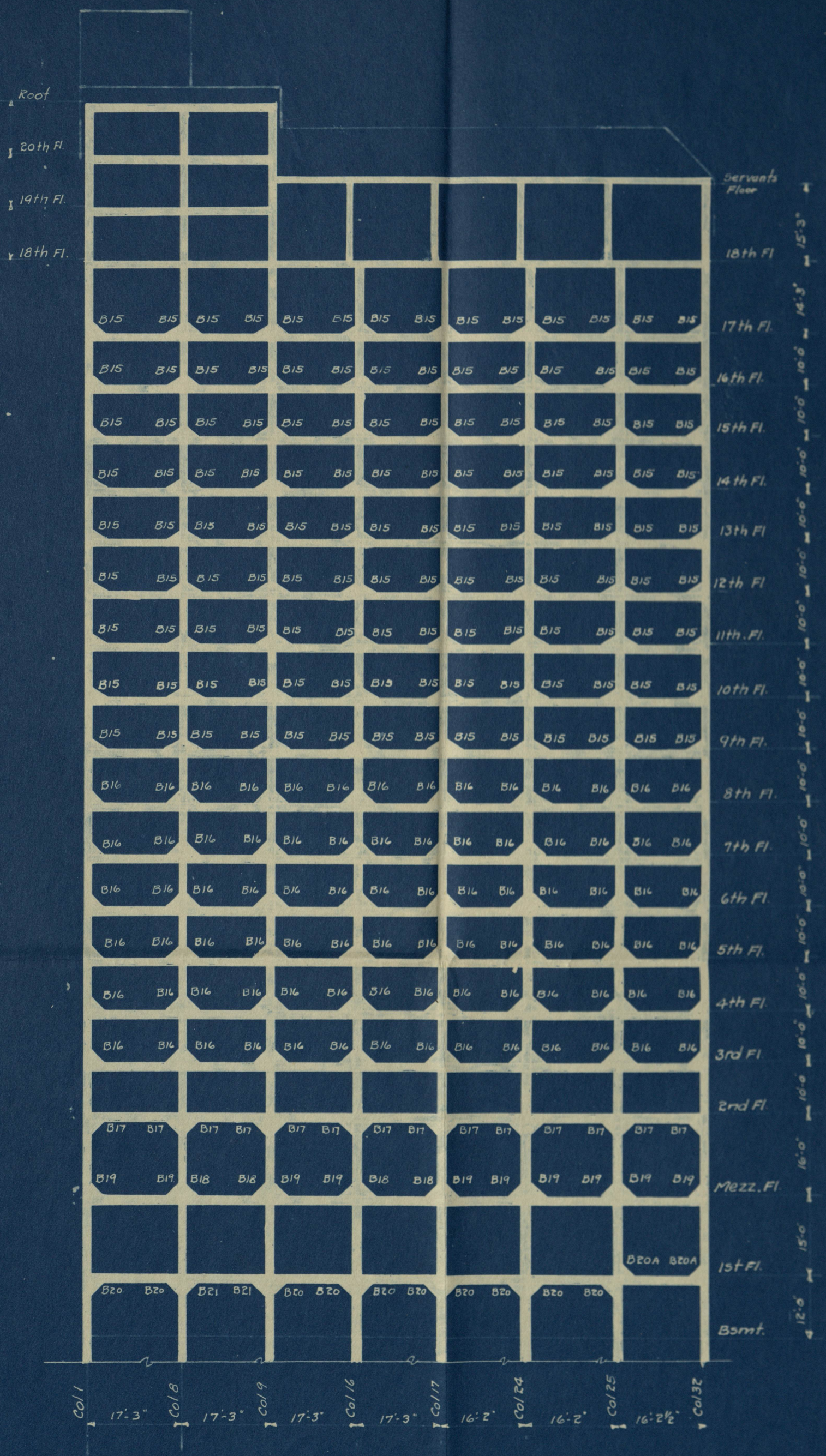
WIND BRACING COL 1 to COL 4

PLATE 6



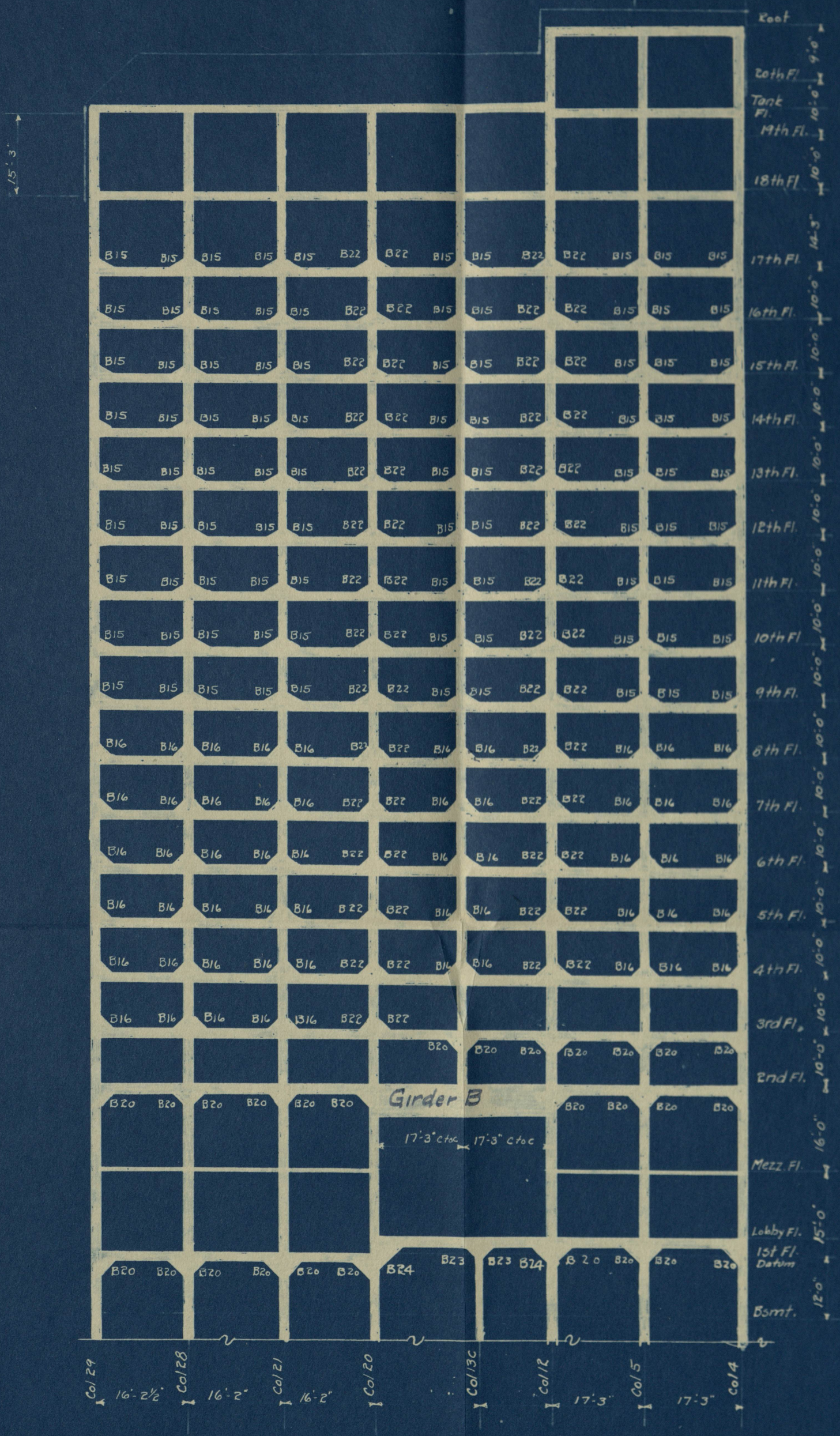
WIND BRACING COL. 29 to 32

PLATE 7



WIND BRACING COL. 1 to COL 32

PLATE 8.



WIND BRACING COL. 4 to COL 29

PLATE 9

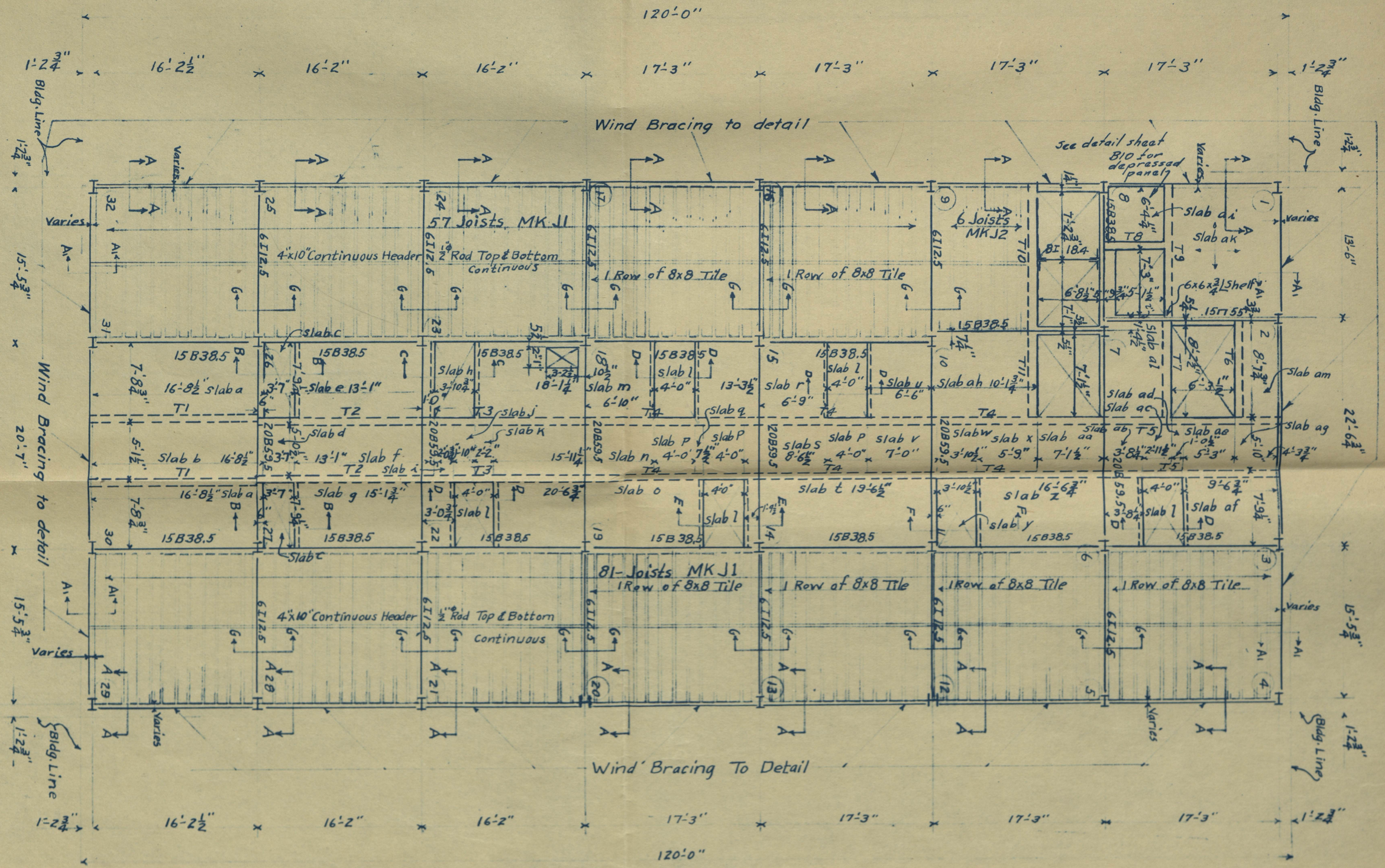
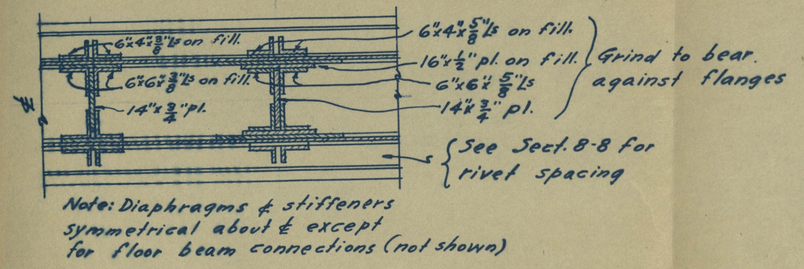
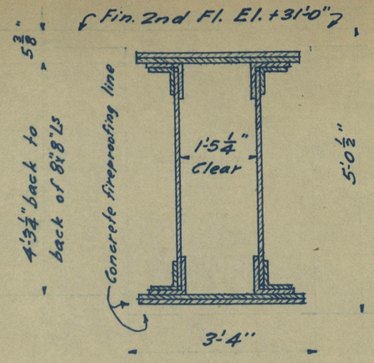


PLATE 10

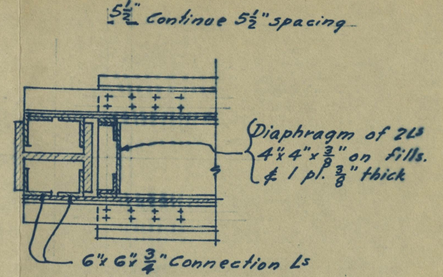
TYPICAL FLOOR FRAMING PLAN - 4th to 17th inclusive -



SECTION 6-6



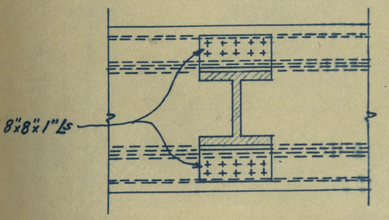
SECTION 7-7



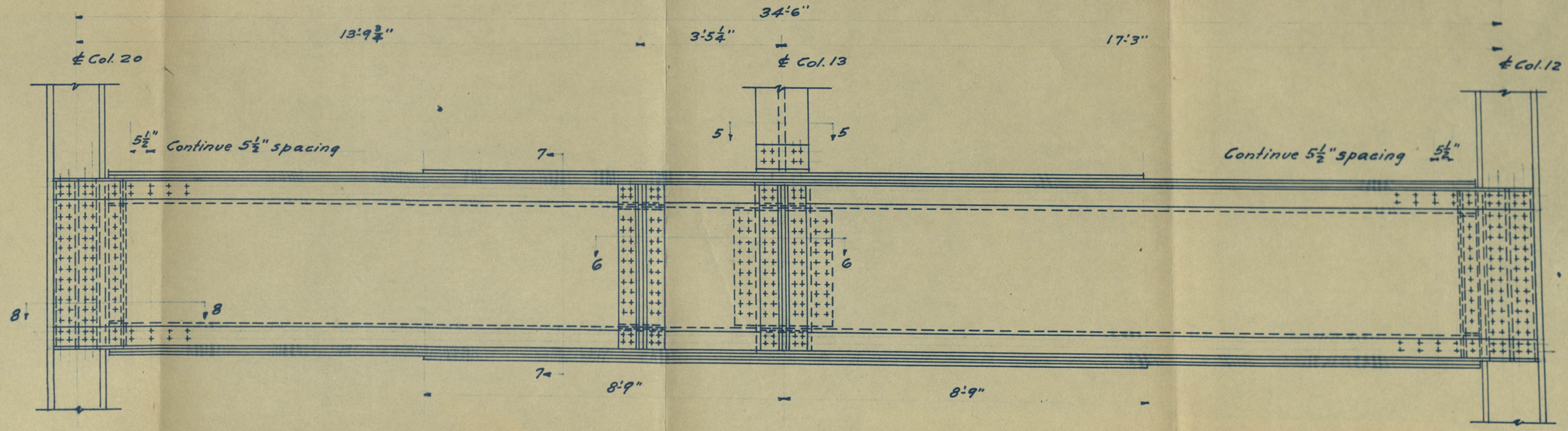
SECTION 8-8

MAIN MATERIAL - GIRDER "B"

2 Web pls. 48"x3/4" } Full Length.
4 Ls 6"x6"x3/4"
4 Ls 8"x8"x1" } Full Length Except
2 Cover pls. 36"x7/8" } Clear Cols. As Shown
2 Cover pls. 36"x1"x17'-6"

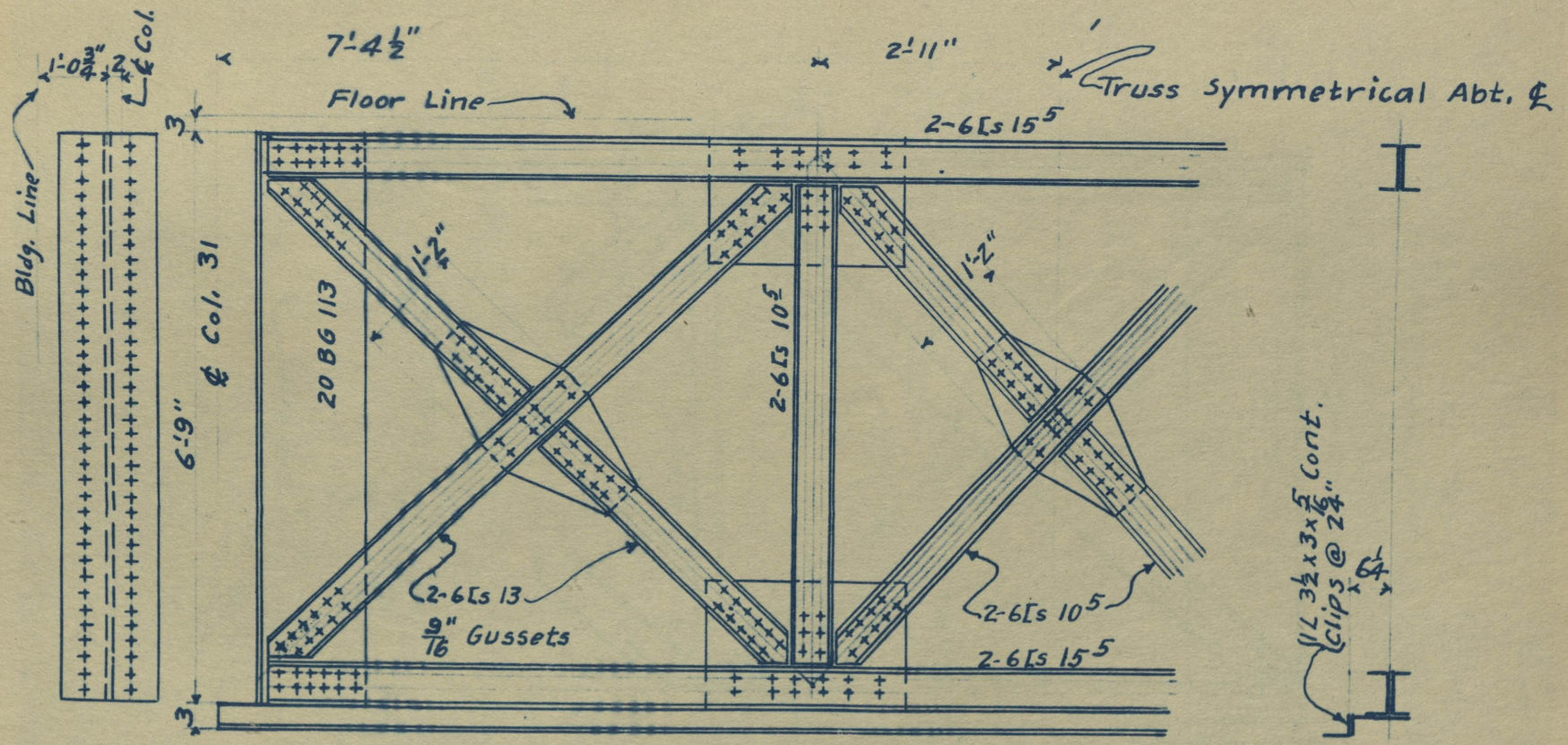


SECTION 5-5



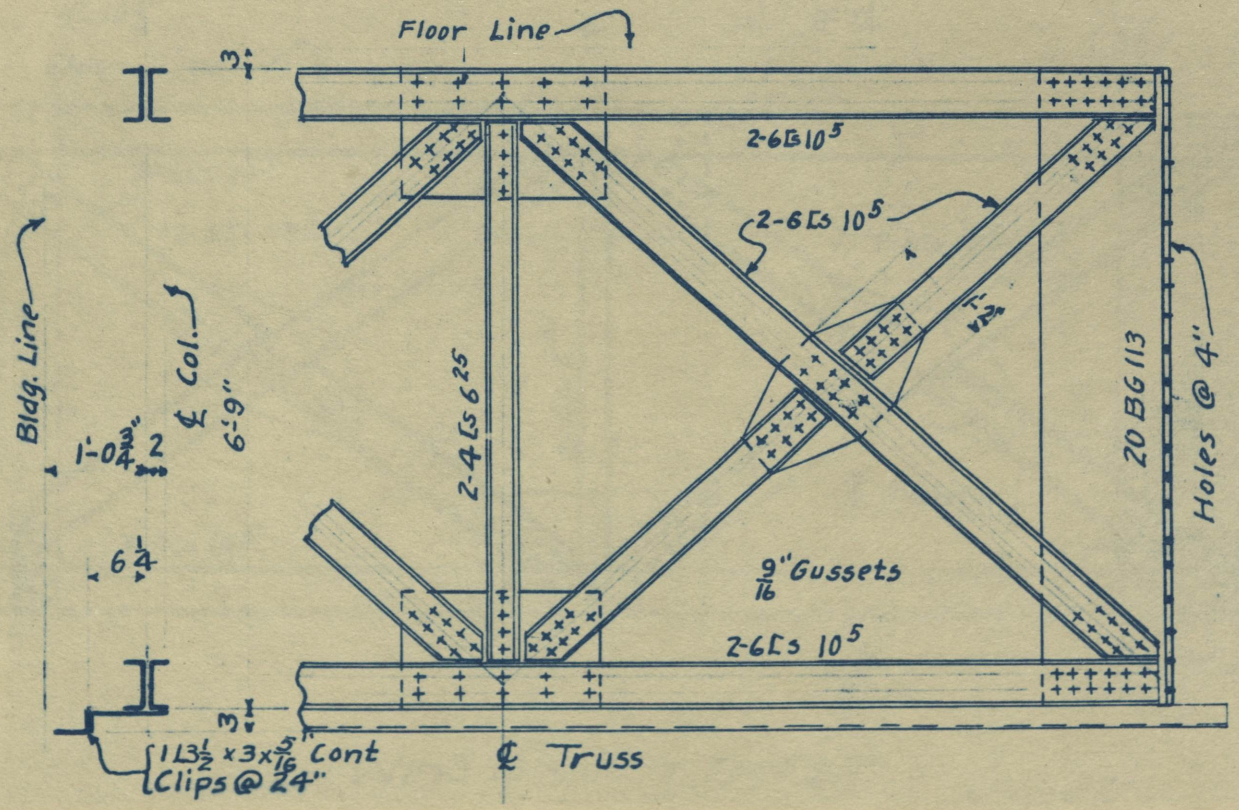
GIRDER "B"

PLATE 11



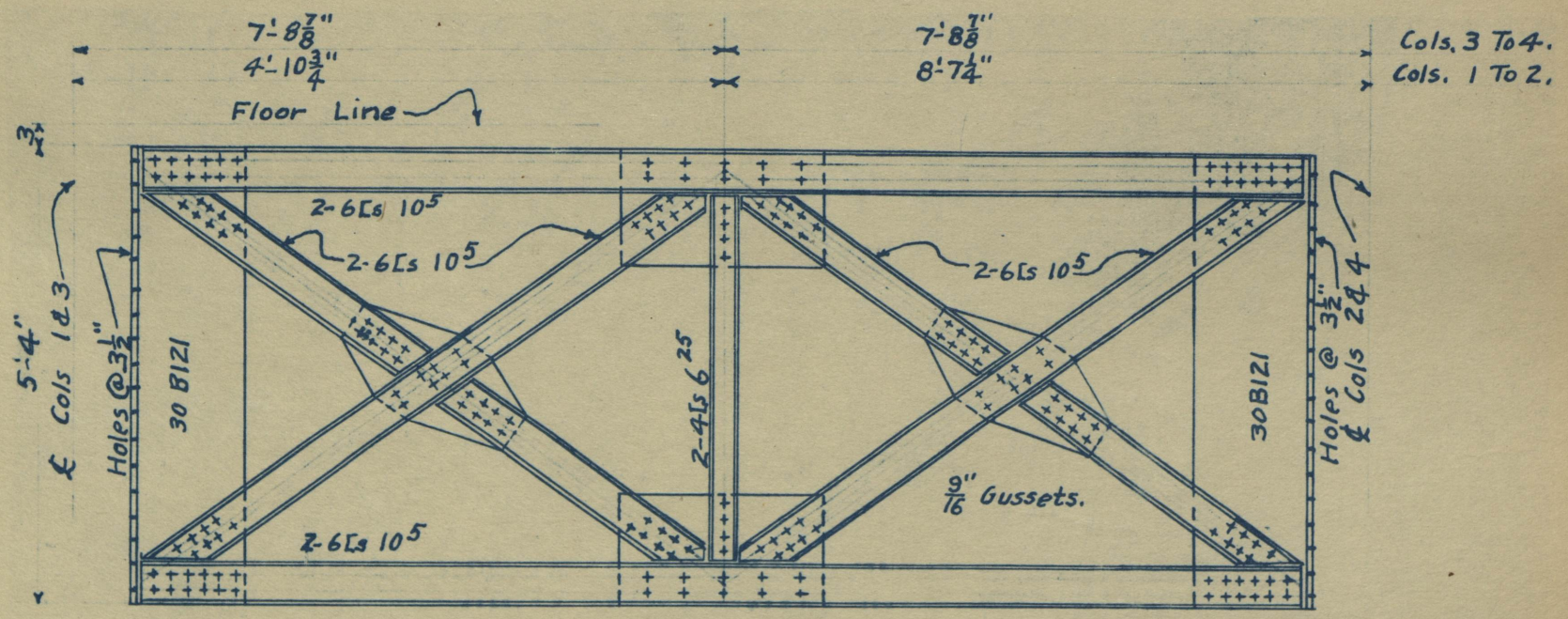
Wind Bracing Truss T-1

PLATE 12



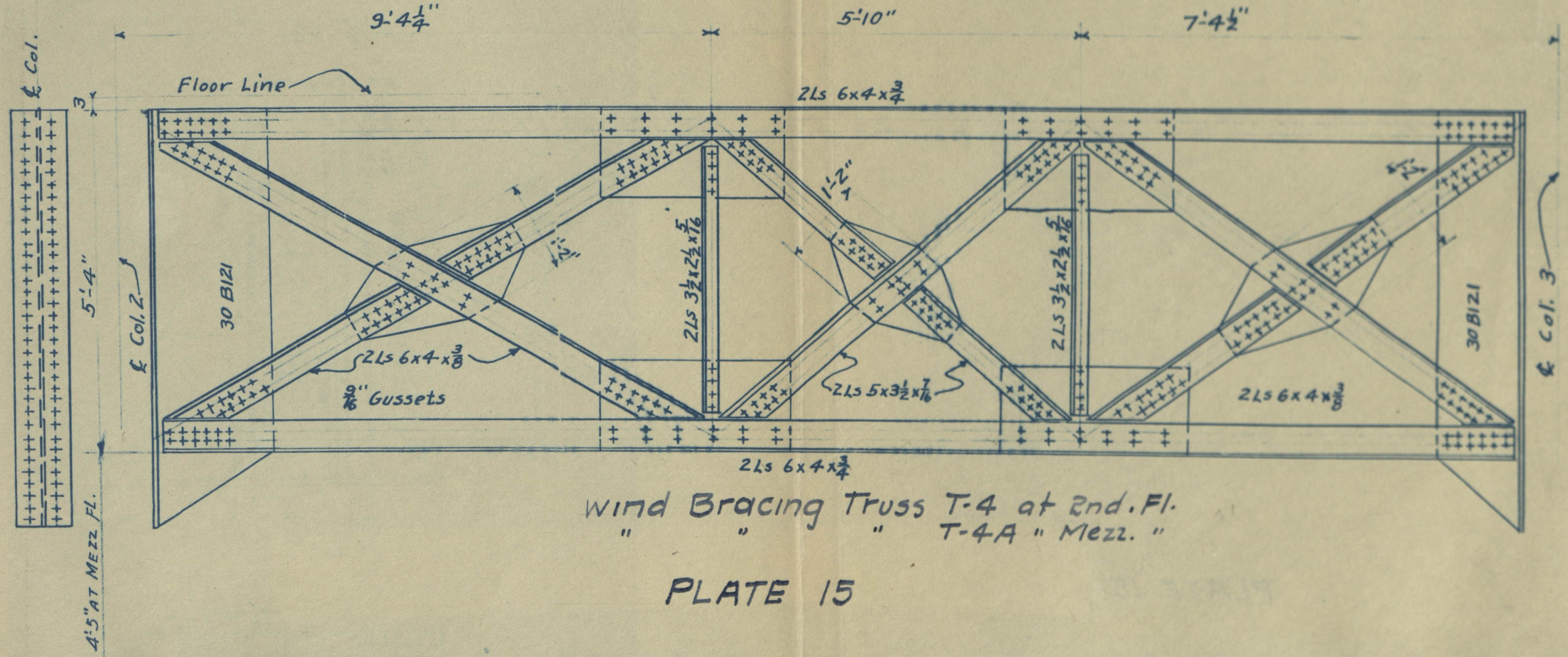
Wind Bracing Truss T2

PLATE 13



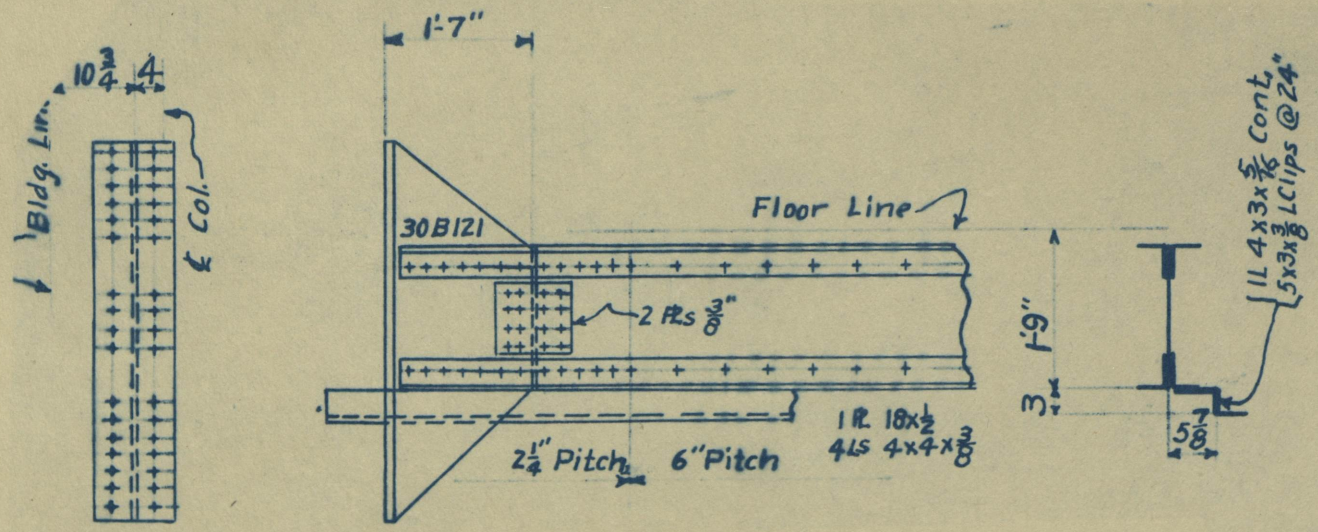
Wind Bracing Truss T-3

PLATE 14



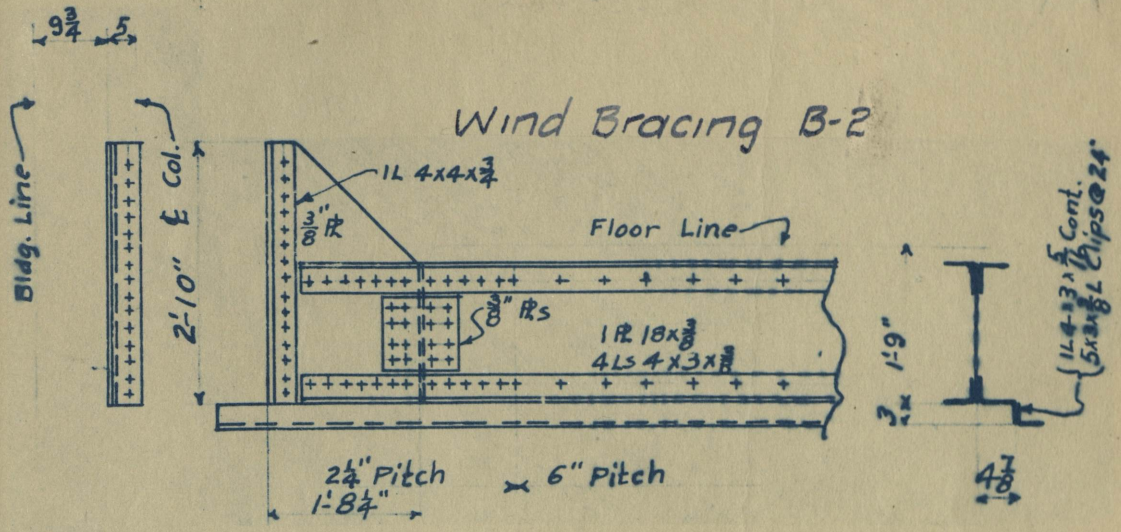
Wind Bracing Truss T-4 at 2nd. Fl.
" " " T-4A " Mezz. "

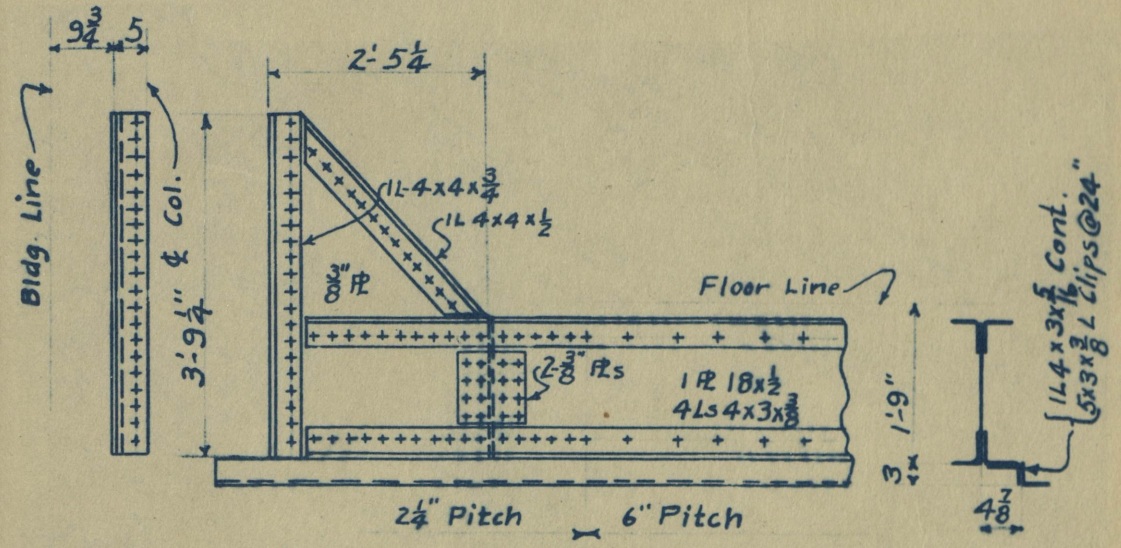
PLATE 15



Wind Bracing B-1

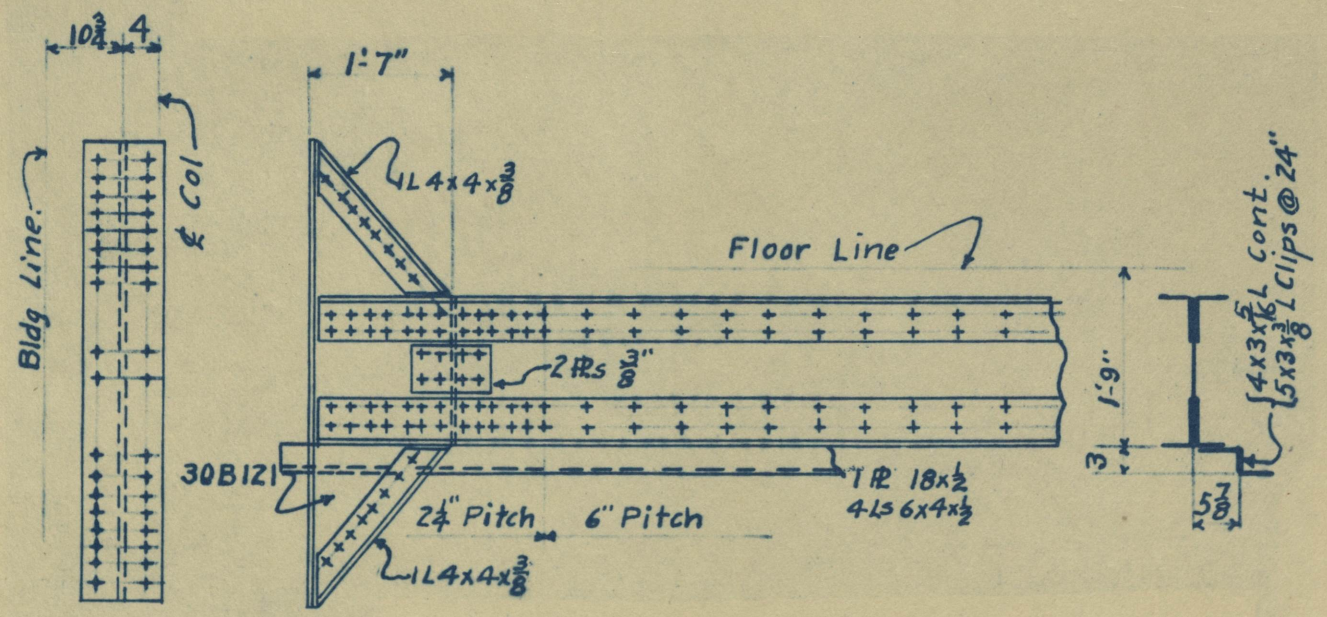
PLATE 16





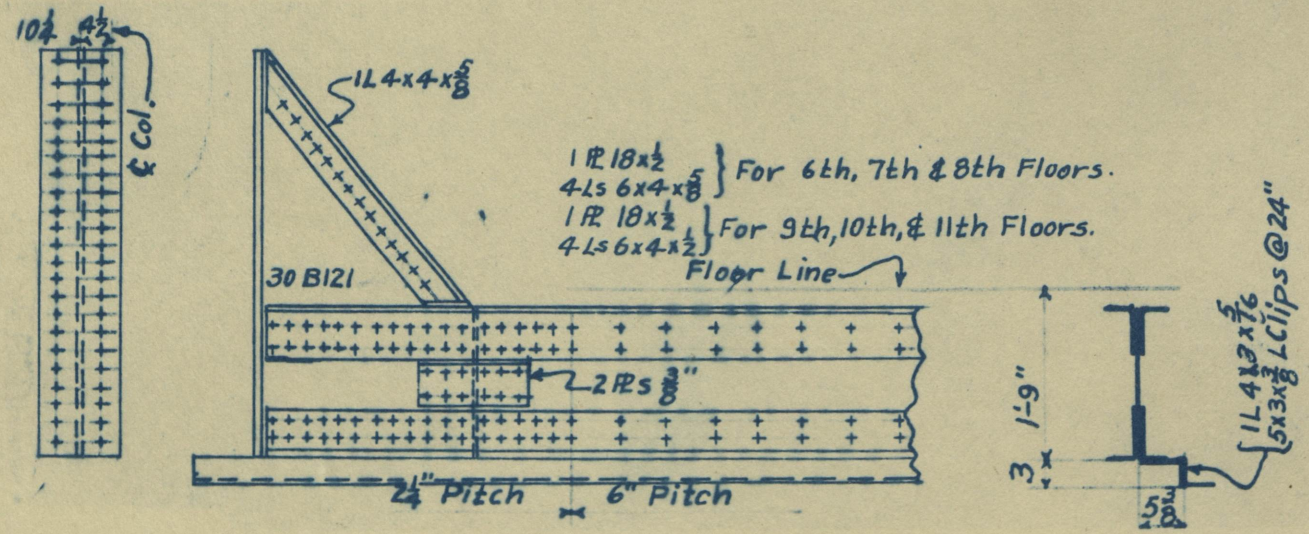
Wind Bracing B-3

PLATE 17



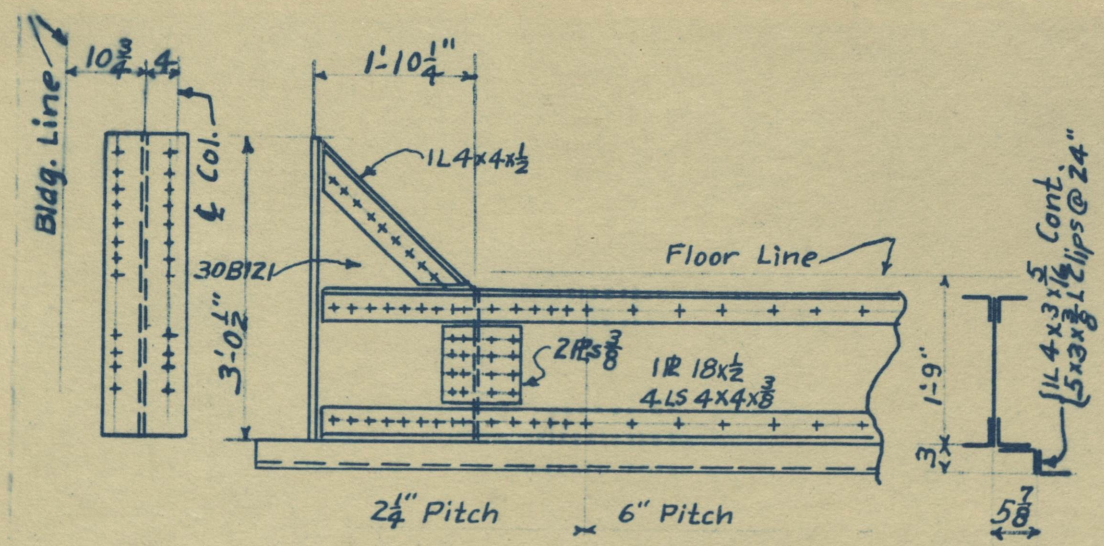
Wind Bracing B4

PLATE 18



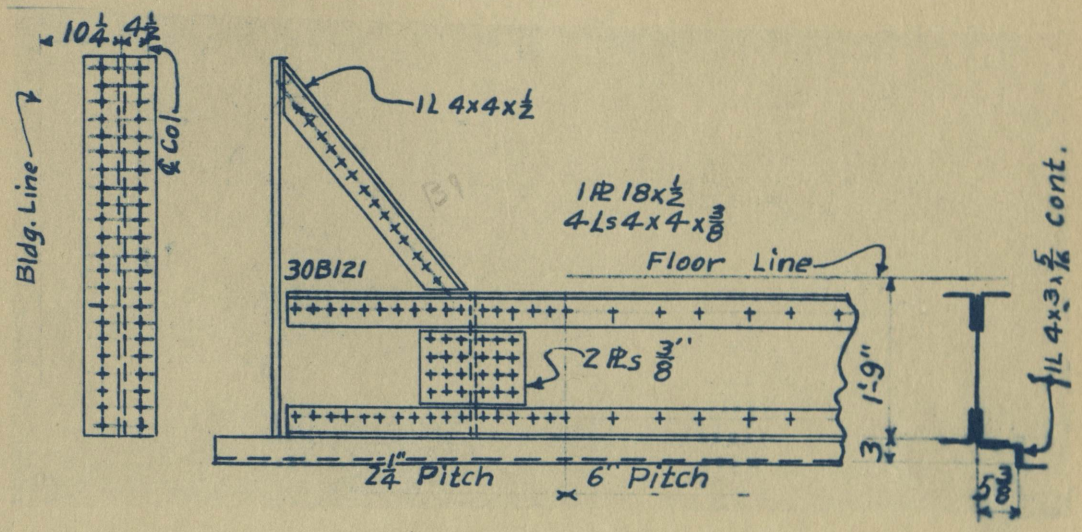
Wind Bracing B5

PLATE 19

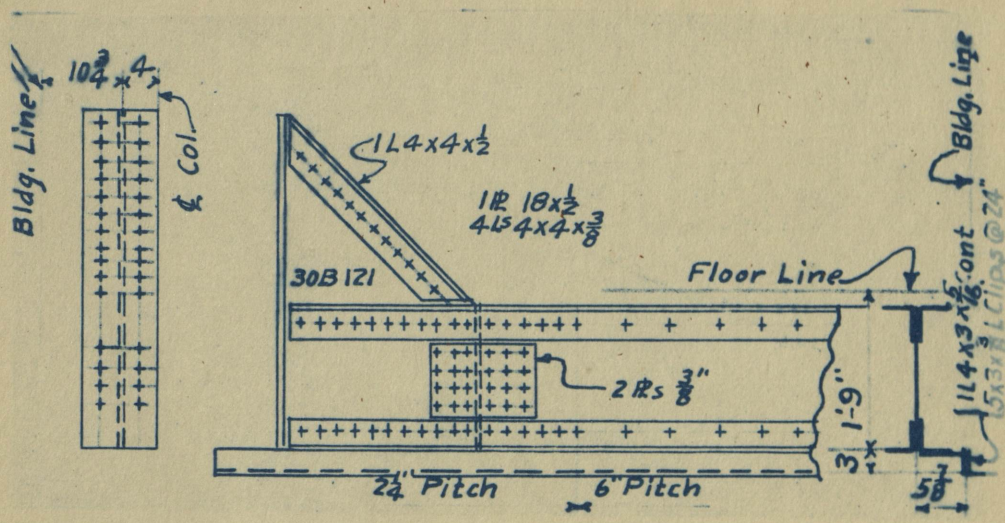


Wind Bracing B 6

PLATE 20

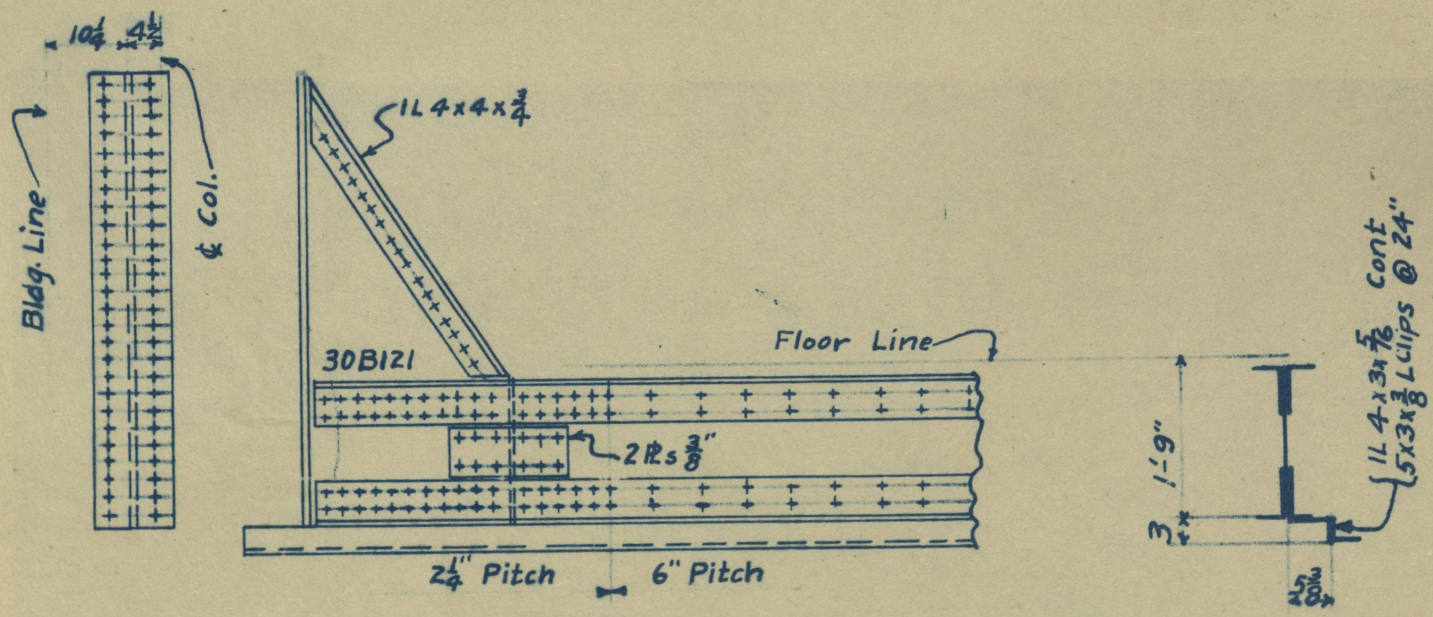


Wind Bracing B9



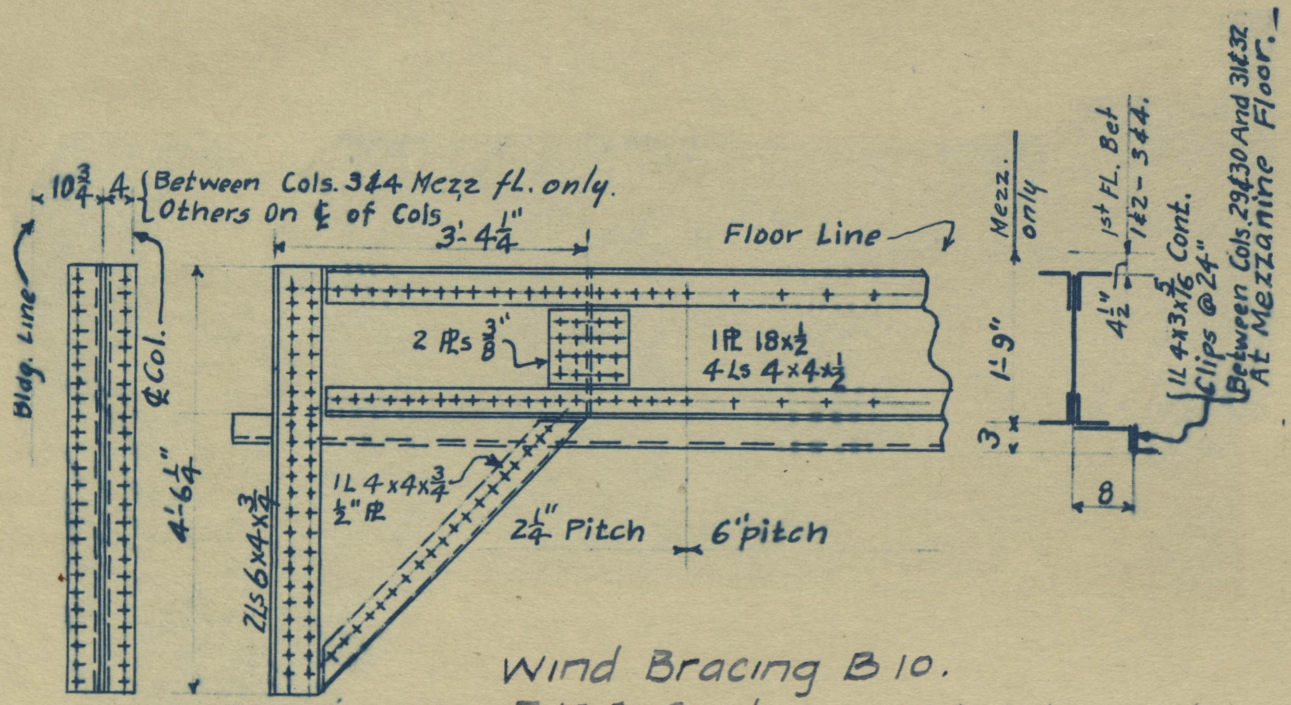
Wind Bracing B7

PLATE 21



Wind Bracing B8

PLATE 22

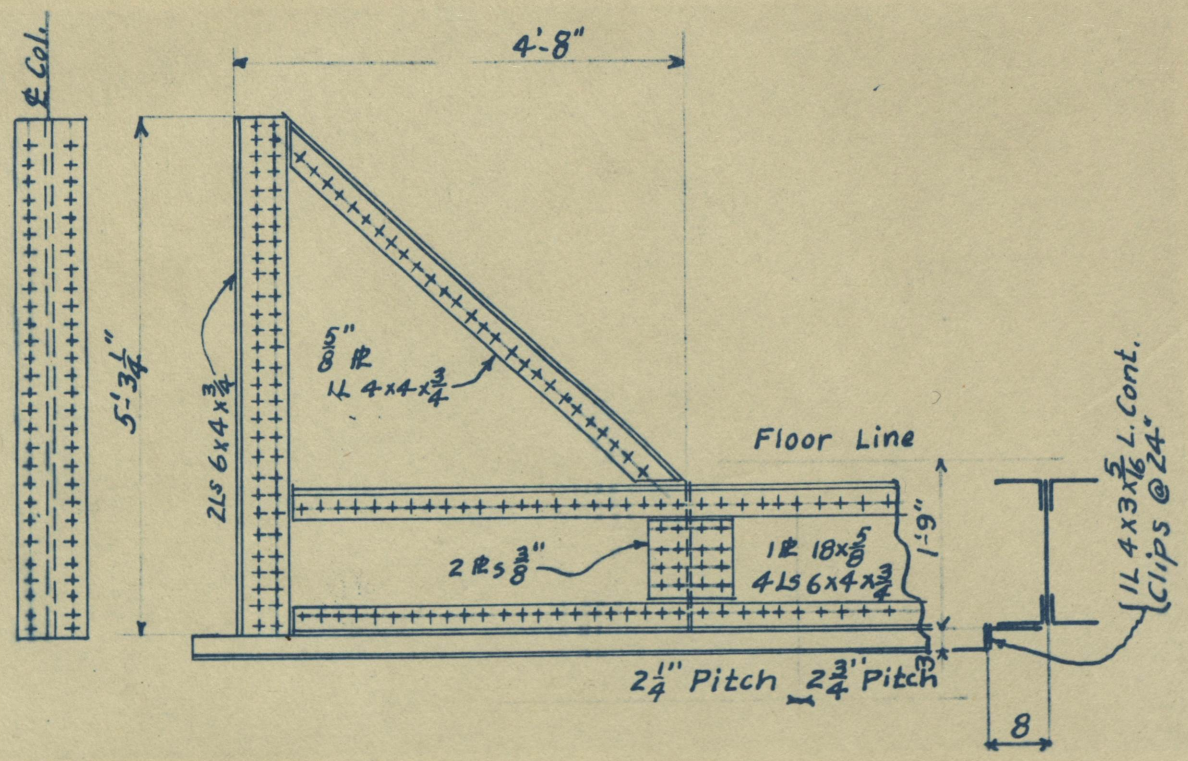


Wind Bracing B 10.

B 10 A Similar except bracket is above

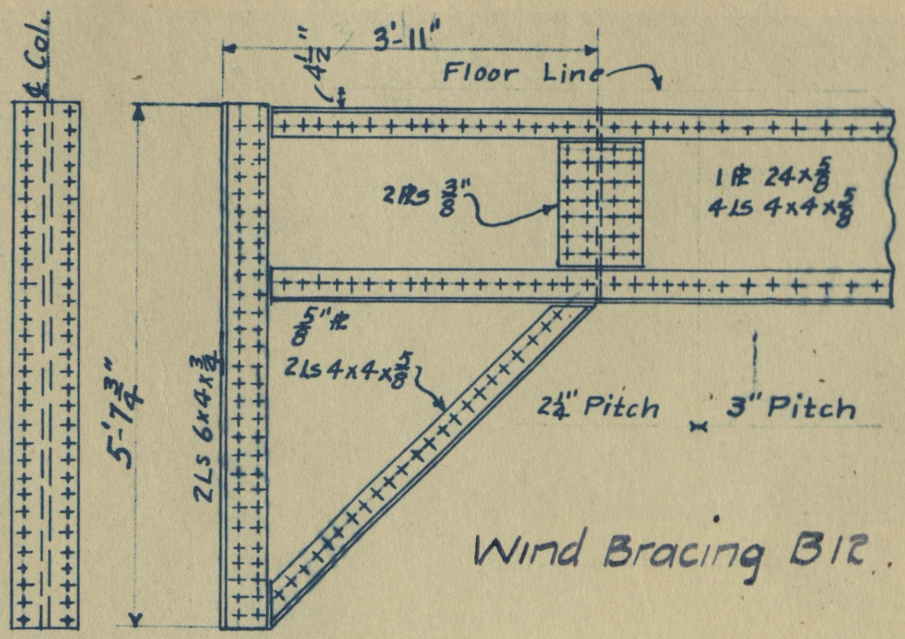
B 10 B " " " " Symmetrical

PLATE 22.A

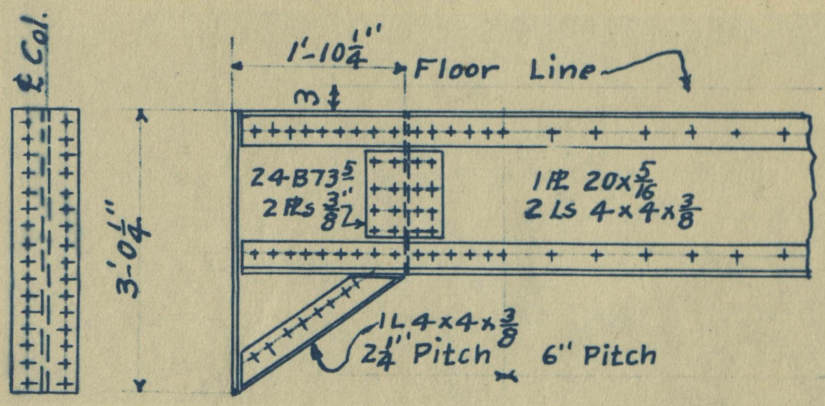


Wind Bracing B11

PLATE 23

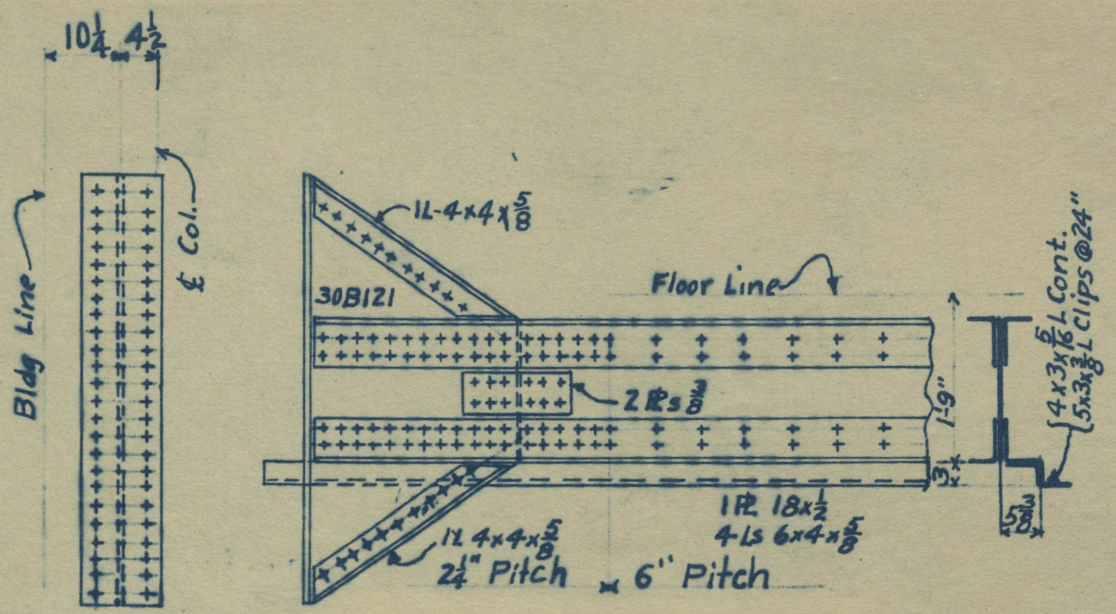


Wind Bracing B12.



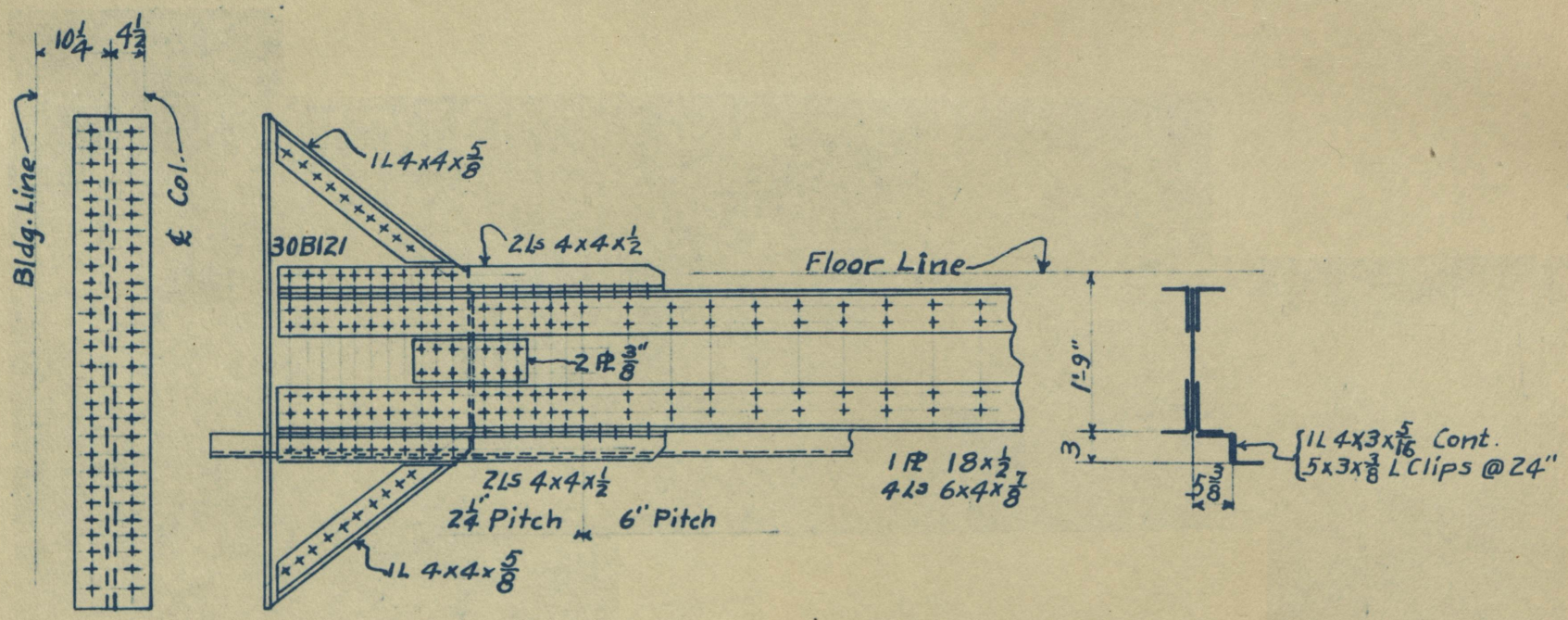
Wind Bracing B17

PLATE 24



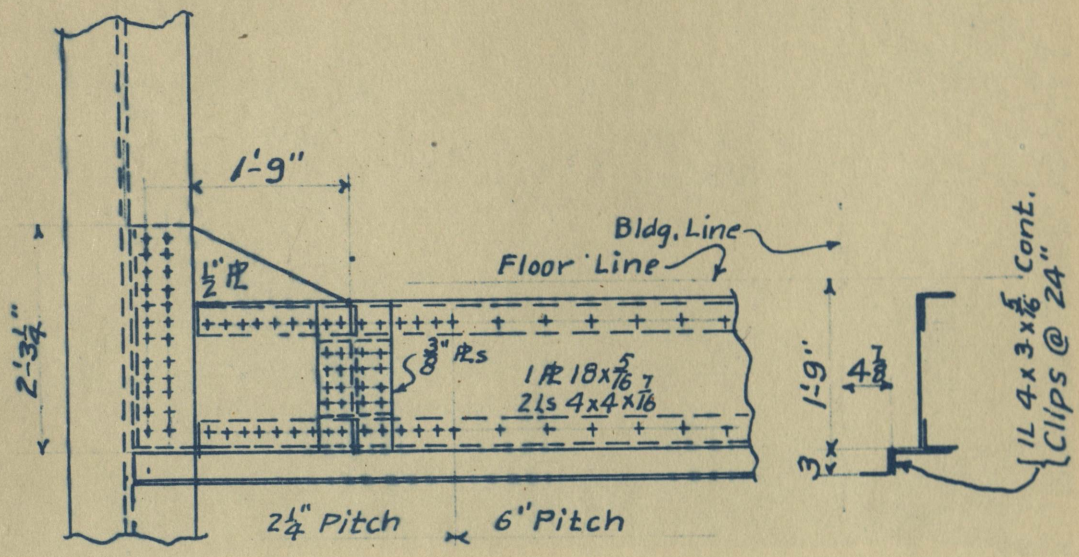
Wind Bracing B13

PLATE 25

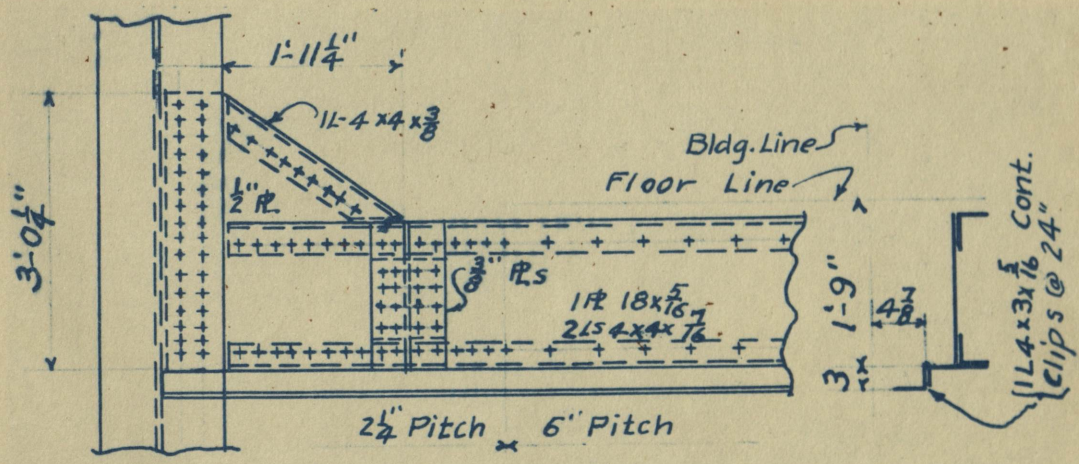


Wind Bracing B14

PLATE 26

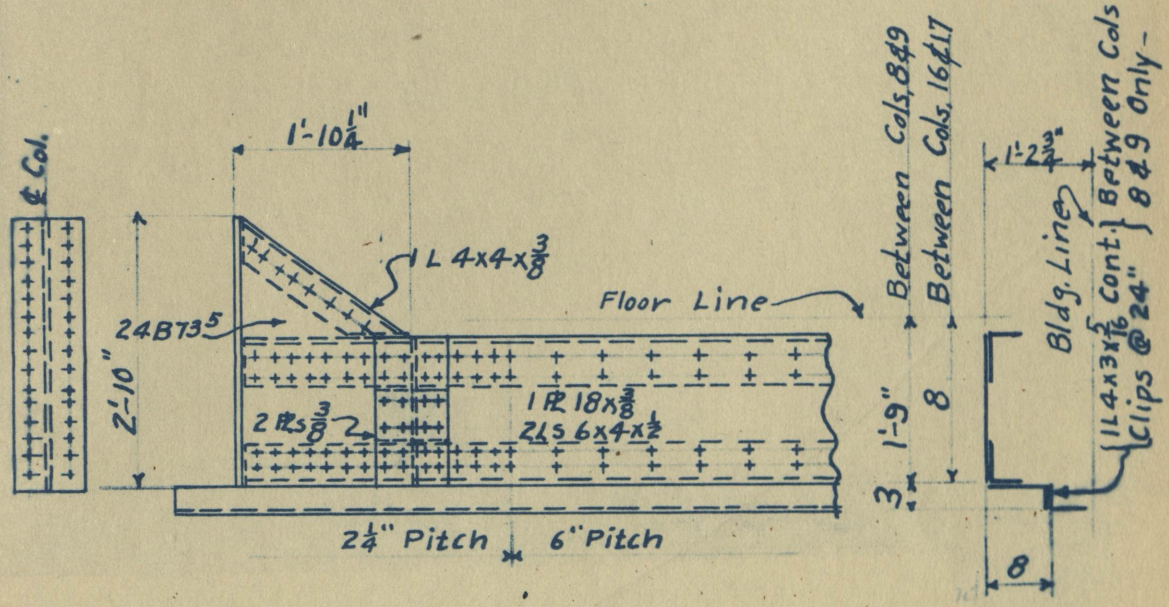


Wind Bracing B15

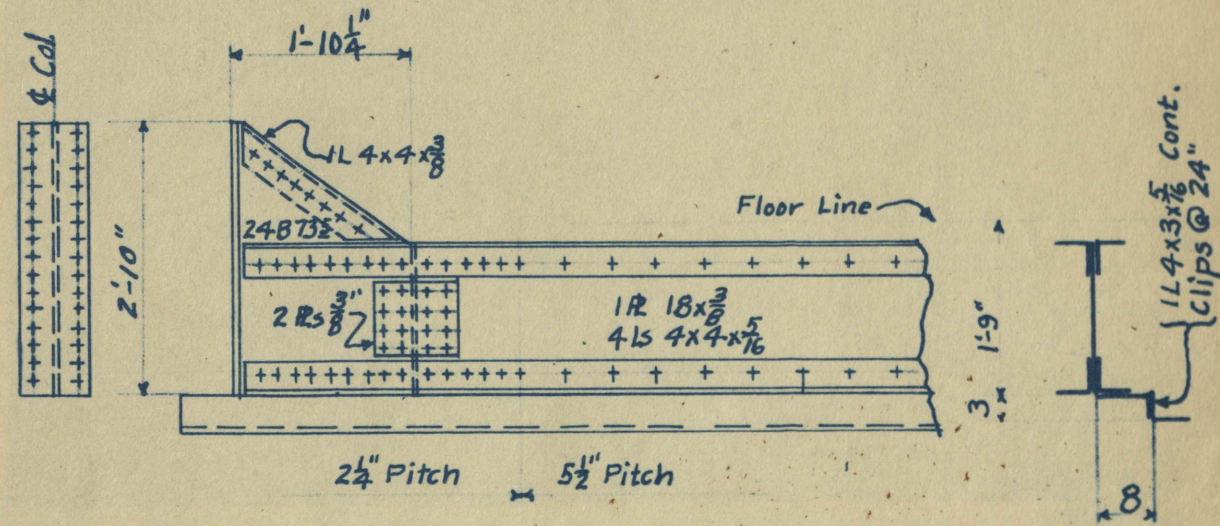


Wind Bracing B16

PLATE 27

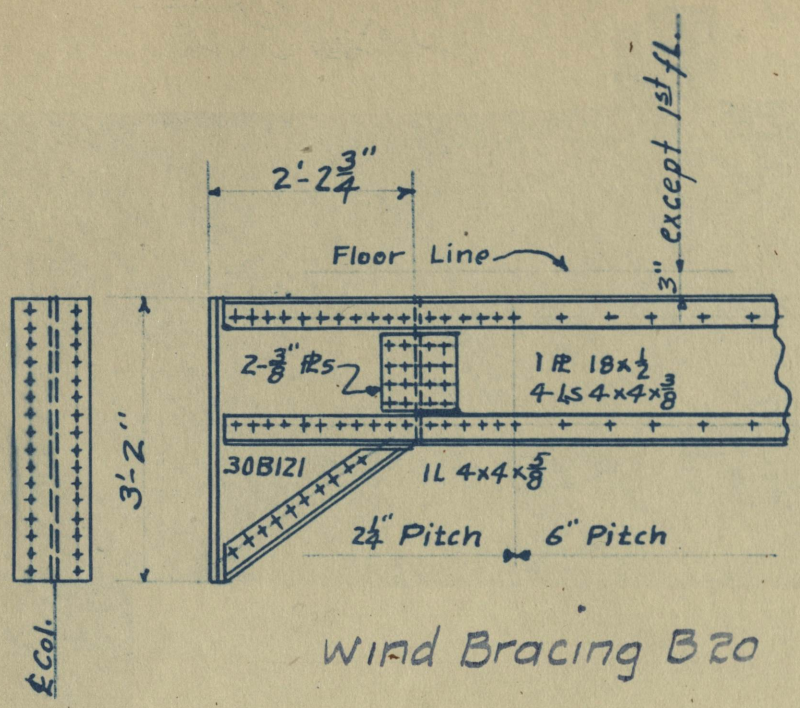


Wind Bracing B18

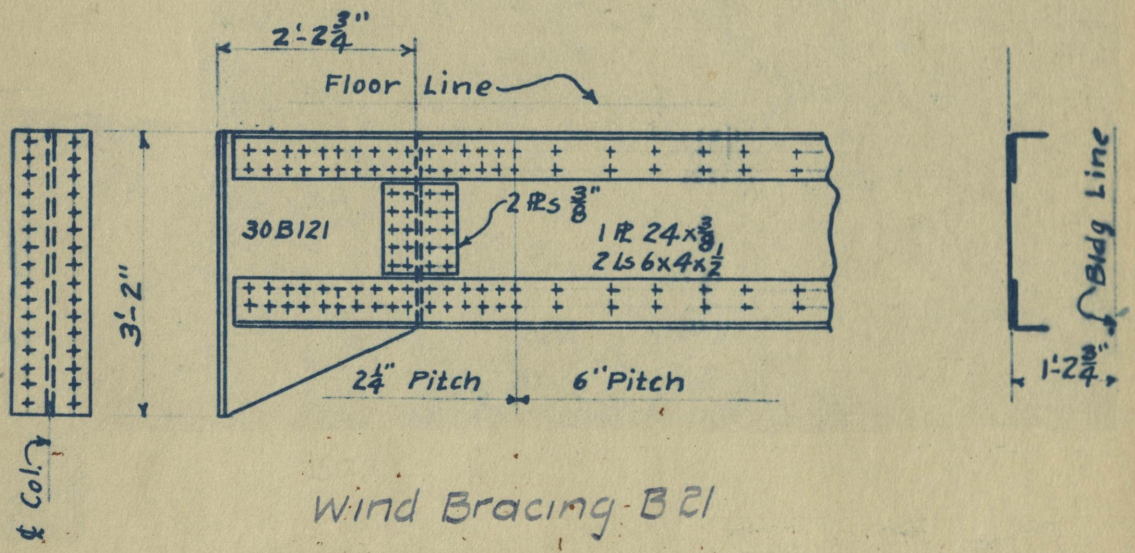


Wind Bracing B19

PLATE 28.

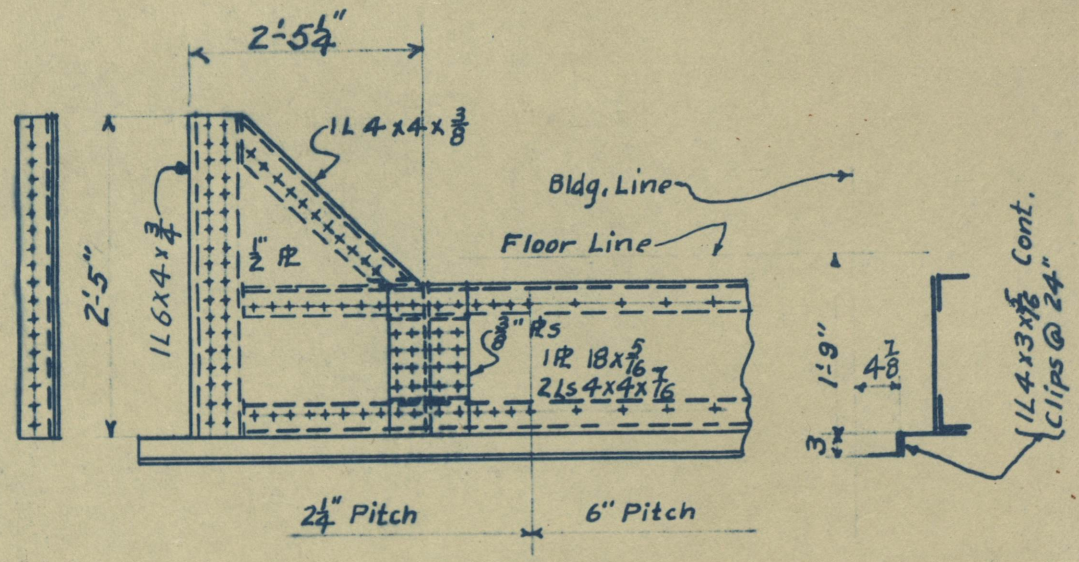


Wind Bracing B20



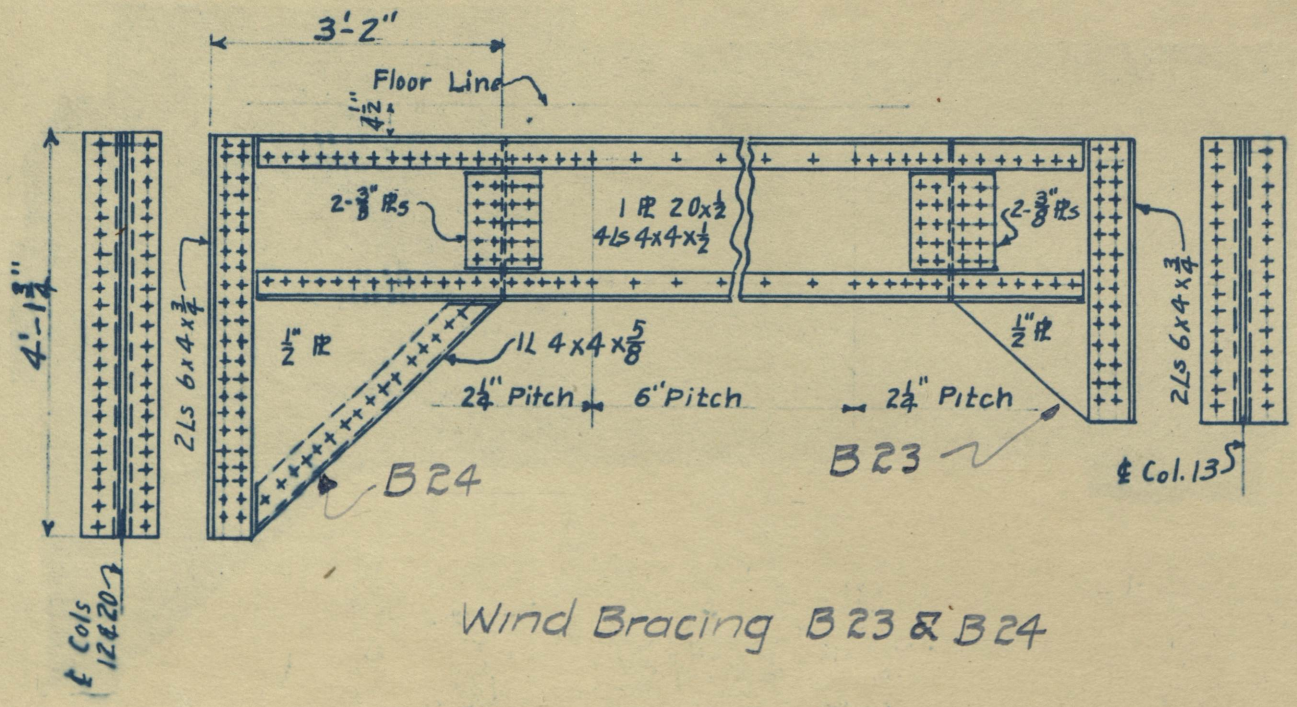
Wind Bracing B21

PLATE 29



Wind Bracing B 22

PLATE 30



Wind Bracing B23 & B24

PLATE 31