STEEL SKELETON OFFICE BUILDING
WITH CAISSON TYPE OF FOUNDATION

BY

L. J. McHugh

ARMOUR INSTITUTE OF TECHNOLOGY

1916
McHugh, L. J.
The practical design of a steel skeleton office

FOR USE IN LIBRARY ONLY
THE PRACTICAL DESIGN OF A STEEL SKELETON OFFICE
BUILDING WITH A DETAILED CONSIDERATION
OF THE CAISSON TYPE OF FOUNDATION.

A THESIS
PRESENTED BY

Lawrence John McHugh.

TO THE

PRESIDENT AND FACULTY
OF

ARMOUR INSTITUTE OF TECHNOLOGY
FOR THE DEGREE OF

BACHELOR OF SCIENCE IN CIVIL ENGINEERING
HAVING COMPLETED THE PRESCRIBED COURSE OF STUDY IN

CIVIL ENGINEERING

APPROVED:

DATE May 19, 1916.
On October 9, 1871 the Great Chicago Fire laid in waste the entire business district and the territory from Harrison Street on the South to Lincoln Park on the North. To-day, and, within a span of fifty years Chicago stands as a great metropolis of over two and one-half million people, with its towering office buildings, its miles of drainage canals, advantageous harbor facilities, and countless factories,
a Chicago sectioned by beautiful driveways and circled by well populated suburbs. It is the result of the tireless, unselfish devotion of our citizens and made possible by the constructive achievements of our builders.

Situated upon a great clay bed, somewhat marshy, originally not more than four or five feet above the average level of the lake and offering no natural advantages, it was necessary to solve the most complicated and carry
on the most difficult problems of the greatest engineering magnitude. These problems have been and surely should be of the greatest interest to every citizen who is interested in the safety and welfare of our city.

It is with this interest that a consideration is made of the problems of the skyscraper towering many feet above the earth and of the soil extending many feet below.
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Plate

A  First floor plan showing layout of stores, corridors, elevators, etc.

B  Typical floor plan showing office arrangement.

C & D  First floor and typical structural framing plans showing arrangement of beams, girders, columns, etc.

E  Showing typical floor construction with a table of arch weights.

F  Plan showing lines of wind bracing
   Diagram of wind pressures acting on building.

G  Moment diagrams for combined loads showing method of combining wind and gravity loads.

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I  Column schedule.

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<td>Standard hard pan caisson table.</td>
</tr>
<tr>
<td>M</td>
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</tr>
</tbody>
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Acknowledgment.

Truly appreciative of the manifest interest and efforts taken in my behalf, I take this opportunity to acknowledge my indebtedness to Mr. John C. Penn, Assistant Professor in Civil Engineering, Armour Institute of Technology, and to express my sincere thankfulness for his valuable assistance.

I am deeply grateful to my father, Mr. John G. McHugh, for his many valuable suggestions and to Professor A. E. Phillips and his associate professors in Civil Engineering, and to our librarian, Mrs. Julia Beveridge for their effective cooperation and ever readiness to serve in my behalf.
PREFACE.

This treatise is divided into two parts.

Part I contains the practical design of a steel skeleton office building. The plates include the necessary architectural and structural floor plans and the requires diagrams and schedules.

Part 2 gives a detailed consideration of the caisson type of foundation together with the historical development of foundations for Chicago's high buildings. It has been the object to make this part a valuable addition to the files of the designer and builder.

May 19, 1916. Lawrence J. McHugh.
PART I

Containing the practical design of a fourteen story steel skeleton office building.
INTRODUCTION

The framework of a steel skeleton office building consists of floor beams and floor girders which carry the floor loads to columns and of the columns which transmit the imposed loads to the foundations, this framework of steel transferring all of the external and internal loads and stresses from the top of the building to the foundations. Special designed wind bracing and the floor girders brace the columns transversely and longitudinally. All connections should be riveted and the entire steel framework protected from fire. In addition to the fireproofing of the steel which may be accomplished with concrete, brick, tile, and terra-cotta, the exposed window areas, door, and other openings are protected and such precautionary steps are taken in the interior that the building may be termed fire-proof.

The building selected for the purpose of illustrating the practical problems of design has been taken because it gives a number of special
conditions in addition to representing a typical case. However, only the typical stringers, girders, etc., have been designed, reference being made to the change that would be required in special cases. The building is designed to be used as an office building. It has fourteen stories above street level and a basement below street level. The basement extends under the sidewalk on two sides of the building.

The building occupies the entire lot, fronting on two streets. Fireproof construction is used throughout. The framework consists of structural steel columns and girders, the floor construction of tile. Partitions in general are four inch hollow tile, plastered on both sides.

Plates C and D give the structural framing plans, and plates A and B a part of the architectural floor plans which are sufficient for this problem; but additional structural and architectural details would be required for making the complete design.
FIREPROOFING

Choice of Material.
All iron or steel shall be protected against the effects of external change of temperature, and of fire by a covering of fireproof material consisting of at least four inches of brick, hollow terra-cotta, concrete, or burnt clay tiles.

For this building concrete is used for fireproofing the steel. It is comparatively easily placed and has many advantages over the other materials. The thicknesses required are

For exterior columns 4"
For interior columns 3"
Spandrels (outside) 4"
Bottom & sides of beams 2"
Around openings 2"

These thicknesses are in compliance with the Chicago Building Ordinance.

LOADS

Classification.
It is of paramount importance that the loads be accurately determined for the
accuracy of the design is dependent upon the accurate determination of the loads.

The structural steel frame of the building is required to support the imposed "dead loads" and "live loads". The "dead load" is taken to include all permanent portions of the buildings, also partitions and permanent fixtures and mechanisms supported by the building. Dead loads are in all cases, gravity loads. The "live load" is taken to mean the temporary and varying loads such as furniture, merchandise, and people. These are dependent upon the type of building: minimum values of the "live load" per square foot of floor area for the different classes of buildings being given in the Chicago Building Ordinance.

Special Loads. In addition to the loads described above additional loads may be caused by elevators, machinery, water in tanks, etc.

Loads on the Building. Unit weights.

To avoid repeating load values in the design and to prevent
misinterpretation in checking, the following table of loads has been made. Average values applicable to ordinary conditions were used.

Table of Weights

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick masonry, hard common, per cu. ft.</td>
<td>125 lb.</td>
</tr>
<tr>
<td>Brick masonry, pressed, per cu. ft.</td>
<td>140 lb.</td>
</tr>
<tr>
<td>Stone concrete, per cu. ft.</td>
<td>144 lb.</td>
</tr>
<tr>
<td>Partition tile, 4 in. thick, per sq. ft.</td>
<td>15 lb.</td>
</tr>
<tr>
<td>Gypsum partition blocks, 4 in. thick</td>
<td>12 lb.</td>
</tr>
<tr>
<td>Plaster on brick, concrete, tile, or gypsum, per sq. ft.</td>
<td>5 lb.</td>
</tr>
<tr>
<td>Steel joists, per sq. ft. of floor</td>
<td>6 lb.</td>
</tr>
<tr>
<td>Steel girders, per sq. ft. of floor</td>
<td>4 lb.</td>
</tr>
</tbody>
</table>

**TYPE OF FLOOR CONSTRUCTION**

In the selection of the type of floor construction a number of items must be considered. The adaptability for the particular requirements of the building; the relative cost of the different types of floors considered; weight and thickness of floor construction; and other like items must be given due consideration.

The flat tile arch between steel I-beam joists
as shown on the following page will be used for this building.

FRAMING SPECIFICATIONS

Arrangement of Girders. Girders are required around the entire perimeter of the building to support the walls. The interior girders are made perpendicular to the long dimension of the building which gives practically duplicate floor panels.

Arrangement of Beams. With the above girder lines established, the beams are spaced as uniformly as practicable. A beam is connected to each column so as to brace it.

DESIGN OF STEEL MEMBERS

Design of Beams. The beams are designed to support the tile arches and floor, a uniformly distributed load. Thus, the procedure results in determining the total load per square foot of floor, the span, and spacing; the product of these factors
Typical Floor Construction.

Table of Arch Weights - Carnegie Handbook.

<table>
<thead>
<tr>
<th>Depth of Arch, inches</th>
<th>Depth of Floor, inches</th>
<th>Approx. Weight, Lb. per Sq. Ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>Steel</td>
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<tr>
<td>8</td>
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<td>8</td>
</tr>
<tr>
<td>10</td>
<td>15</td>
<td>9</td>
</tr>
<tr>
<td>12</td>
<td>17</td>
<td>10</td>
</tr>
</tbody>
</table>
determining the load on the beam.

Typical floor beam.
Span = 17' -11" = 17.92 feet

Load. 10" flat arch ...... 79#/ per sq. ft.
Live Load .......... 50#/ per sq. ft.
Total unit load .... 129#/ per sq. ft.

limiting span is 8'0"
actual span is 4.93 feet

total load on beam = 4.93 x 17.92 x 129 = 11,165#
allowable load on 10" I-22# beam = 13,500#

Therefore use a 10"I-22#.

Design of beams carrying partitions.
weight of tile partition plastered on both sides,
27#/ per square foot.
height of story = 11'8" = 11.67 feet.
total weight of partition on beam = 11.67 x 17.5 x 27 = 5,520#
load on beam = 11,165# plus 5,520# = 16,685#
allowable load on 10" I-35# = 18,360#

Therefore use a 10"I-35# beam.
The women's toilet room is on the 4th floor and the men's toilet room and barber shop are on the 5th floor.

The architectural detail shows a tile floor, marble base, and wainscoat and stalls and furred ceiling in the lady's rest room.

An extra load, therefore, is imposed on the beams and to provide for this additional load the subjected beams are to be 10"I-25# with an allowable load of 15,300#.

Spandrel Beams. (between columns 13 and 18)

These beams carry, in addition to the imposed live and dead loads, the load of the exterior brick curtain wall.

Area of wall supported = 17.5 x 11.67 = 204 sq. ft.
Window area (deducted) = 7.5 x 4.3 = 32 sq. ft.
Area = 172 sq. ft.
weight of wall = 172 x 1.08 x 125 = 21,500#
load from floor = \( \frac{11165}{2} \) = 5,587#

total load = 27,087#

Therefore use a 15"I-35# beam.
Spandrel Beams.  (between columns 2 and 5)

Area of wall supported = 17.5 x 11.67 = 204 sq. ft.
Window area (deducted) = 2(7.5 x 5.75) = 86 sq. ft.

\[
\text{Area} = 118 \text{ sq. ft.}
\]

Weight of wall* = 118 x 1.08 x 125 = 14,750#

Load from floor = \frac{11165}{2} = 5,587#

Total load = 20,337#

Allowable load on 12"I-31.5# = 21,962#

Therefore use a 12"I-31.5# beam.

GIRDERS.

Typical floor

Span of girder - 18 feet.

Concentrated loads at one-third points.

Distance between beams is 6 feet.

Weight per foot of beam = (6x130) plus 25#I = 305#

Load at panel point = 805 x 17.5 = 14,087#

B.M. = 14,087 x 6 x 12 = 1,014,264 inch pounds.

Wt. of partition per foot of girder = 29 x 12 = 348#

Wt. plus weight of 60# girder = 348 plus 60 = 408#

B.M. = \frac{408 x 18 x 18 x 12}{8} = 198,000 inch pounds.
Total Bending Moment = 1,014,264 plus 198,000
= 1,212,264 inch pounds.

\[ M = \frac{I}{G} = \frac{1,212,264}{16,000} = 75.7 \text{ inch}^3 \]

Spandrel Girders. (between columns 1 and 13)

These girders support an additional curtain wall load.

See design of girders on previous page for span and floor load.

B.M. of stringer reactions = \((7045 \times 9 - 7045 \times 3)12\)
= 507,240 inch pounds.

Weight of curtain wall (window area deducted) equals
\((11.7 \times 8) \times (1.08) (125) = 11,700\#\)
resulting B.M. = \(\frac{11700 \times 18 \times 12}{8} = 315,900\# \text{ inch pounds.}\)

Total Bending Moment = \(\frac{507,240\#}{315,900\#}\)
= 823,140 inch pounds.

Section modulus required = \(\frac{823,140}{16,000} = 51.44 \text{ inch}^3.\)

It has been decided to carry the entire wind load to the two ends and carry it through the columns.
and girders 1-13 and 6-18. As this will probably require extra metal in the members the final section will not be determined until the wind stresses have been found.

SPANDBEL GIRDERS. (between columns 6 and 18)

Span of girder - 19.2 feet
Concentrated loads at \( \frac{1}{4} \) points.
Distance between beams = 4.8 feet.
weight per foot of beam = \((4.8 \times 130) + 22\#\) = 645#
load at panel point = \(645 \times \frac{17.5}{2} = 5,645\#\)

B.M. of stringer reactions = \(\frac{3 \times 5645 \times 9.6 - 5645 \times 4.8}{2}\)
= 54,200 foot pounds
= 650,400 inch pounds.

weight of curtain wall (window area deducted) =
\((11.7 \times 9.5 \times 1250 = 13,895\#\)
resulting bending moment = \(\frac{13895 \times 19.2 \times 12}{8} = 400,175\#\)

Total Bending Moment = 650,400 inch pounds
400,175 inch pounds

= 1,050,575 inch pounds.
section modulus required = \frac{1,056,575}{16,000} = 65.56 \text{ inch}^3.

As this girder has been selected to carry the wind load the final section will not be determined until the wind stresses have been found.

**Special Beams.**

Special beams are required around elevators and stairs, and for the support of elevator machinery, stack, pipes and vent, and tanks.

The section between columns 14-17-8-11 which includes three panels contains several special features, that is, a stairwell, three elevator shafts, a stack, and a pipe shaft.

The 8"I-18# support the elevator guys and 6"I beams are sufficient for the framing around the stack and pipe shaft.

The stair load is taken as 50# per square foot for the dead load and 100 pounds per square foot for the live load. The framing will consist of 8"I-beams connecting to the spandrel between columns 16-17.
First Floor Beams.

Load on beams:
- dead load = 80# (10" flat arch) (unit)
- live load = 85# (85% of 100#)
- Total = 165#

load per foot of beam = 165 \times 5 \text{ plus } 30\# \frac{1}{I} = 855\#

\begin{align*}
M &= \text{section modulus required} = \frac{855 \times 17.5 \times 17.5 \times 12}{8} \\
&= 26.09 \text{ inch}^3.
\end{align*}

This requires a 10" I-30# beam.

For beams carrying partitions, the additional weight per foot = 29 \times 12 = 348#

the additional section modulus = \frac{348 \times 17.5 \times 17.5 \times 12}{8}

= 9.98 \text{ inch}^3.

Total section modulus required = 26.8 \text{ plus } 9.98

= 36.78 \text{ inch}^3.

This requires a 12" I-35# beam.

First Floor Beams.

For beams marked 99-100-101 which are of a longer span due to the stair construction

\begin{align*}
\frac{M}{S} &= I = \frac{855 \times 19.3 \times 19.3 \times 12}{8} = 29.8 \text{ inch}^3.
\end{align*}
section modulus of a 10"I-40# beam = 31.7 inch^3.

Therefore use a 10"I-40# beam.

Beams leading from the elevators

total load on beam = (165 x 5.83) + 18#I = 987#

\[ M = \frac{I}{c} = \frac{987 \times 10.6 \times 10.6 \times 12}{8} = 11.0 \text{ inch}^3. \]

section modulus of a 8"I-18# beam = 14.2 inch^3.

Therefore use a 8"I-18# beam.

**First floor girders.**

Span - 18.0 feet.

Concentrated load at \( \frac{1}{4} \) points.

Distance between beams - 4.5 feet.

Load per foot of beam = (165 x 4.5) + 30#I = 775#

Load at panel point = 775 x 17.5 = 13,550#

B.M. of stringer reactions = \( \frac{3 \times 13550 \times 9 - 13550 \times 4.5}{2} \)

= 121,950 foot pounds

= 1,463,400 inch pounds.

**weight of partition and girder per foot of girder,**

\( \frac{20 \times 18}{1} \) plus 65# girder = 587#
resulting bending moment = \( \frac{587 \times 18 \times 18 \times 12}{8} \)

= 285,300 inch pounds.

Total Bending Moment = 1,463,400 inch pounds

285,300 inch pounds

____________________________________

= 1,748,700 inch pounds.

required section modulus = \( \frac{1,748,700}{160000} \) = 109.3 inch^3.

section modulus of a 20"I-65# is 117.0 inch^3.

Therefore use a 20"I-65# beam.

Girders supporting the beams leading from the elevators.

Span - 17.5 feet.

Concentrated loads at one-third points.

Distance between beams - 5.83 feet.

Load at panel point = 987 x 5.83 = 5190# 

B.M. = 5190 x 5.83 x 12 = 363,300 inch pounds.

weight of partition plus portion of floor load plus assumed weight of girder,

\( (29 \times 17.5) \) plus 330 plus 35#I = 872#

resulting bending moment,

\( \frac{872 \times 17.5 \times 17.5 \times 12}{8} \)
resulting bending moment equals 400,300 inch pounds.

Total bending moment = 363,300 inch pounds. 
400,300 inch pounds.

763,600 inch pounds.

section modulus required = 47.7 inch$^2$.

section modulus of a 15"I-36# is 54.1 inch$^2$.

Therefore use a 15"I-36# beam.

Spandrel Girders between columns 1-13 and 6-13.

Span - 20.0 feet

Concentrated loads at $\frac{1}{4}$ points.

Distance between beams - 5.0 feet

Load at panel point - $855 \times 8.5 = 7270\#$

B.M. of stringer reactions-$(3 \times 7270 \times 10 - 7270 \times 5)\frac{12}{2} = 872,400$ inch pounds.

weight of curtain wall (window area deducted),

$(11.7 \times 9.5) \times (1.08) \times (125) \text{ plus } 1700 = 15600\#$

resulting bending moment, \(\frac{15600 \times 20 \times 12}{8} = 463,000''\#\).
Total bending moment = 872,400 inch pounds. 
468,000 inch pounds. 

= 1,340,400 inch pounds.

As these are the girders that will be required to take the wind stresses, the section that will not be determined until the combined effect of the gravity and wind stresses have been found.

WIND BRACING.

In a building such as we are considering it is important that the wind bracing be analyzed with care. The Chicago Building Ordinance requires a design to resist a horizontal wind pressure of 30 pounds per square foot for every square foot of exposed surface.

The combined stresses due to wind loads and dead and live loads should not exceed ordinary stresses by more than 50%. If stresses developed by the wind alone do not exceed 50% of those due to dead and live loads, they may be neglected.

It is assumed that wind pressure acts horizontally and bears uniformly over the entire wind-
ward surface of the building and that it may occur in any direction.

Route of Stress. It has been decided to carry the entire wind load to the two ends and transfer it through the columns and girders between columns 1-13 and columns 6-18. The lines of wind bracing are shown on the following plans.

At each floor then there are horizontal thrusts equal to the product of the story height, one-half the length of the building, and the allowed unit pressure. This amounts to 10,500 pounds. Referring to Table G there are found listed in the third column of the table these horizontal thrusts at each floor. In the fourth column of the table these horizontal thrusts are summarized from the top down to each floor level, giving the total Horizontal Thrusts. Each tier of bracing must transmit a vertical shear equal to the difference in flange stress between a point midway in the story above the tier in question, and a point midway in the story below.
Moment Diagrams for Combined Loads showing method of combining wind and gravity loads.

Lawrence J. McHugh
Plan showing lines of Wind Bracing.

Wind pressures acting on building.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>10,000</td>
</tr>
<tr>
<td>14th Fl</td>
<td>10,500</td>
</tr>
<tr>
<td>13th Fl</td>
<td>10,500</td>
</tr>
<tr>
<td>12th Fl</td>
<td>2,700</td>
</tr>
<tr>
<td>11th Fl</td>
<td>3,750</td>
</tr>
<tr>
<td>10th Fl</td>
<td>10,500</td>
</tr>
<tr>
<td>9th Fl</td>
<td>4,000</td>
</tr>
<tr>
<td>10th Fl</td>
<td>10,500</td>
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<td>8th Fl</td>
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<td>7th Fl</td>
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<tr>
<td>2nd Fl</td>
<td>1,000</td>
</tr>
<tr>
<td>1st Fl</td>
<td>1,000</td>
</tr>
<tr>
<td>Basement</td>
<td>1,000</td>
</tr>
</tbody>
</table>

W. story height x bldg.wt. = 20

1167 = 20

10,500 = 20

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<table>
<thead>
<tr>
<th>Floor</th>
<th>Lever Arm</th>
<th>Horizontal Thrust at each floor, H</th>
<th>Total horizontal thrusts from roof to each flange, W'</th>
<th>Bending Moment in inches</th>
<th>Combined Bending M from Diagram</th>
<th>Section to be used between Columns 6' 7&quot;</th>
<th>Section to be used between Columns 6' 7&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>12.00</td>
<td>6000</td>
<td>6000</td>
<td>17520</td>
<td>1050.575</td>
<td>15&quot; I 60&quot;</td>
<td>15&quot; I 60&quot;</td>
</tr>
<tr>
<td>1</td>
<td>11.67</td>
<td>10500</td>
<td>16500</td>
<td>48180</td>
<td>32850</td>
<td>1850000</td>
<td>15&quot; I 60&quot;</td>
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<td>10500</td>
<td>27000</td>
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<td>15&quot; I 60&quot;</td>
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<td>15&quot; I 60&quot;</td>
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<td>69000</td>
<td>204400</td>
<td>189070</td>
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</tbody>
</table>

Lawrence J. MCHugh
This difference of flange stress can be found by ascertaining the difference in bending moments between the two points and dividing by the effective depth of the system, and for our calculation the formulas given in Burt's were applied.

The Formulas:

The bending moment in an intermediate column in any story equals the total shear in that story multiplied by the story height, and the product divided by two times the number of panels. That is,

\[ M = \frac{W'H}{2n} \]

The bending moment in an outside column is one-half that in an intermediate column, or

\[ M = \frac{W'H}{4n} \]

The bending moment in a girder is the mean between the bending moments in the column above and below the girder. It is expressed by the formula

\[ M = \frac{1}{4n} (W'aHa + W'bHb). \]
Rivets Connecting Girder to Column. In the design of ordinary girders, end connections have been required to resist only vertical shear. In the design of wind-bracing girders it is important that the connection of the girders to the column resist the bending moment.

Distance center to center of columns = 19.5 feet
Depth of girder = 24\(\frac{1}{2}\) inches (back to back of angles)
Unit stresses used are 50% in excess of those allowed for gravity loads.
The rivets thru the end angles and column webs are field driven, 7/8 in. diameter.

Value of 7/8 rivet, field = 1\(\frac{1}{2}\) x 6000 = 9000#
B.M. (24" girder, 1st.Fl.) = 5,326,000"#

From previous assumptions 43 rivets spaced 3"c.c. were investigated. As in a beam, the unit fiber stress varies from zero at the neutral axis to a maximum at the extreme fiber and by thus determining the stress upon each rivet at its distance from the
neutral axis, it was found that the 45 rivets offered a resistance equal to 5,709,300" which is in excess of that required.

DESIGN OF COLUMNS.

Interior.
The action of the wind loads produce bending moment stresses which are similar to the stresses produced by an eccentric load. The effect of this eccentric loading may, however, be expressed in terms of concentric loading. The formula for the transformation is,

\[ W'_w = W'ec \frac{r}{E} \]

in which \( W'_w \) is the equivalent concentric load
- \( e \) is the moment arm,
- \( r \) is the radius of gyration,
- \( c \) is the distance to outermost fiber,
and \( W' \) is the eccentric load.

If \( W'_w \) is less than 50% of the concentric load, it may be neglected. \( W'_e \) is the moment of force about
the point of contra-flexure. \( W' \) the figured eccentric load must be divided by "n" panels to give the required loading on columns or it may be expressed as

\[
W'e = \frac{e \times M}{n} \text{(the calculated moment)}
\]

Should the loading as figured exceed 50\% of the concentric load a new area would be required. This area is determined by dividing the sum of the concentric and equivalent concentric loads by the allowable fiber stress per square inch times 50\%. This area will determine the section to be used. At each floor there is eccentricity due to the unequal loads from the girders connecting to the outside of the columns. The loads connecting to the webs produce no eccentric effect.

The Calculations.

the floor area tributary to the interior columns is

\[
17.5' \times 18.33' = 329.5 \text{ square feet.}
\]
the floor dead load (2nd to 14th fls.) is

Arch ......................... 79#
Girder ........................ 5#
Fireproofing .............. 2#
4" tile ........................ 2#
Total dead load * =88#/ sq. ft.

the total floor dead load equals

\[ 329.5 \times 88 = 28,995\# \]

For the weight of column and covering an average amount per foot of length is computed. This amount is excessive at the top and too small at the bottom.

the column weight is,

Steel = 150#

Concrete = \( \frac{360\#}{510\#/ft.} \)

the weight per column is,

\[ 510 \times 11.67 = 5950\# \]

Live Load on Columns.
The following table gives the live loads on the columns proportioned according to the Chicago Building Ordinance.
<table>
<thead>
<tr>
<th>Floor.</th>
<th>Load.</th>
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<td>Roof</td>
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<tr>
<td>9</td>
<td>30</td>
<td>1</td>
<td>25#</td>
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</table>

There are two 2000 gal. tanks which add a load of 35000#.

The dead weight of the roof is taken at 40 x 327.5 equaling 13180#.

**Eccentric Effect.**

The formula is \( W'e = \frac{Pec}{r^2} \)

deduct \( \frac{1}{2} \) of the live load = \( P \)

for floor columns \( P = 7000 \)

for roof columns \( P = 4125 \)

\[
W'e \ (\text{roof}) = \frac{4125 \times 5 \times 5}{4.28 \times 4.28} = 5635
\]

the eccentric effect will therefore be 5635-4125 or 1500#

\[
W'e \ (\text{floor}) = \frac{7000 \times 5.5 \times 5}{4.28 \times 4.28} = 10,520
\]

the eccentric effect will therefore be 10,520-7000 or 3520#.

**Exterior Columns.**

At all floors from the second to the roof, one-half of the floor
load is transmitted to the column through the girder. The girder is connected to the web of the columns and the load is considered as concentric. The spandrel beams which transfer the wall loads are connected to the outside of the column and, therefore, causes eccentric loads.

The formula \( W'e = W' \frac{e^2}{r^2} \) is used for the determination of this eccentricity.

The calculations.

the floor area tributary to the exterior columns is

\[
\frac{329.5}{2} = 164.75 \text{ square feet.}
\]

the total floor dead load equals

\[
164.75 \times 98 = 15,495\#
\]

the wall being of terra-cotta, filled and backed with brick, the weight is taken at 120\# per cubic foot, the wall load minus the window area is

\[
(4.25 \times 17.5) \quad (6.0 \times 7.4) \times 1.2 \times 120 - 118.77 \times 132 = 15780\#
\]

the weight per column is taken as given for the interior column = 5950\#
Eccentric effect.  The spandrel load and one-half of the floor dead load is eccentric.

\[
\begin{align*}
\text{spandrel load} &= 15,780\# \\
\frac{1}{2} \text{ floor d.l.} &= 7,250\# \\
\text{eccentric load} &= 23,000\#
\end{align*}
\]

The eccentric effect is, therefore, 
\[
\frac{23000 \times 8 \times 5}{2.60 \times 2.60} = 136,275\#.
\]
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<thead>
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**Interior Columns**

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Plate I

Lawrence J. McHugh
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**Column Section**
- Column No. 8: 10 H 49
- Column No. 9: 10 H 54
- Column No. 10: 10 H 56
- Column No. 11: 10 H 59.5

**Bethlehem Sections**
- 10 H 71.0
- 12 H 91.5
- 14 H 114.5
- 14 H 136.5
- 14 H 146.0
- 14 H 170.5
- 14 H 178.5

**Plate I**
Lawrence J. Higginbotham
PART 2

Containing a detailed consideration of the caisson type of foundation.

Chicago is situated over a great clay bed at one time the bottom of an ancient lake, sometimes called by geologists, Lake Chicago, which, at its highest state, seems to have stood about sixty feet above the present level of Lake Michigan. The upper part of this clay has hardened for a thickness of 9 or 10 feet, the clay below this and extending about 90 feet down to rock (but of varying depth) being of different degrees of softness and in places streaked with quicksand and mixed with sand and gravel. At various stages of ancient Lake Chicago there arose those elevations now known as Blue Island, Stony Island, etc., names which suggest the early condition of these places. The various sandy ridges which here and there seam the plain, represent bars or spits formed by the action of the waters of the ancient lake.
Directly under the hardened crust the bed of soft clay is full of water as was the condition when the ice which covered the Chicago plain had retreated and allowed the waters of the former lake to subside. The clay gets dryer and harder deeper down and at the bottom there is much coarse gravel with many glacial boulders. These conditions exist from the Chicago River south to 12th Street and from the lake shore west to the south branch of the River. This clay deposit, however, varies considerably in thickness over the bed-rock, showing that the land where Chicago now stands was formerly of a broken and undulating character.

It was the problem of the architects and builders of those days to determine the capacity of this uninviting flat of loose spongy soil. The safe load for buildings on the hardened clay crust was from \( \frac{13}{2} \) to 2 tons (net) per square foot of foundation, the universal practice being to build piers of dimension stone masonry. Cast iron shoes on top of the piers supported the columns of the building. With this con-
struction the main or first floor was above the street level being reached by steps, or there was a half-basement below and a first floor above street level. With the coming of the eight and ten story buildings it was realized that the first floor must be level with the street and basement space provided for the mechanical plant and other appurtenances. As the construction was then presented, the basements were practically filled with stone pyramids offering no space for the boilers, engines, and various other machinery.

To obtain and provide this space for the boilers a foundation was devised of a grillage of steel rails embedded in concrete which formed a shallow spread foundation. This steel rail grillage was latter developed into the I-beam grillage plan which has found extensive use everywhere in the United States.

The "floating" foundation gave ample basement area and in principle was to carry the shallow spread foundation on the thin, hard stratum covering the deep bed of soft clay underlying the city, keeping the allowable load within the safe limits and avoiding all disturbance and penetration of the clay crust. The "float-
"floating" type undesirable for the larger classes of buildings in the future.

At the present time the deep foundation system is being used almost exclusively. The rapid structural growth in certain sections increasing the value of building ground and decreasing the building area has led to the demand for sub-basement space where the power plant and other appurtenances may be placed so that the basement may be entirely free for business
purposes. In many of the new department stores this machinery is placed in a second sub-basement leaving the basement and first sub-basement available business and storage purposes. With the heavier loads of these taller buildings there was a desire for greater stability and while the "floated" foundations were sufficient to support buildings of 8 or 10 stories, yet with the increased loads of the taller buildings the crust itself would deflect. This shortage and increased cost of building area and the varying treacherous soils encountered led to the development of the caisson type of foundation.

These caissons are wells, preferably carried down to bed rock, and filled with concrete. The excavation is made by hand and the excavated materials are lifted out in buckets, raised by a windlass operated by hand, electric or steam power. The method used in sinking caissons in and about Chicago is the open air method or as it is called, the Chicago method.
Caisson Foundations.

Method of Excavation. After the caissons have been laid out according to the plans, cofferdams are constructed, or as the phrase, the caissons are topped. An excavation is made 4 feet in depth with a diameter the exact diameter of the caisson and the top row of lagging driven. A second row of lagging is then driven 2 feet (this amount according to the specifications) outside of the inner row, the annular space between the two rows excavated to solid blue clay and filled up to the top of the lagging with freshly excavated blue clay and thoroughly puddled and tamped until solid and watertight.

The lagging is generally maple or any good, sound lumber, properly matched, with beveled edges so that each piece may be considered as a stave with radial joints corresponding to the size of the required circle, either 2 or 3 inches in thickness, and 5 inches in uniform width. The lagging is cut in
lengths not to exceed 5 feet, shorter lengths being provided when soft earth, water, quicksand or other unusual conditions are encountered.

As soon as the cofferdam, which is to prevent the caving in of the first set of lagging due to the uncertainty of the top soils, has been constructed, the earth is excavated to the bottom of the next row of lagging, the lagging and rings put in place, bolted and wedged until each separate piece of lagging has a full and solid bearing against the surrounding soil. The rings used are of heavy wrought steel, each in two perfect semi-circles and are provided in sufficient quantity to resist the pressure of the soil in whatever location they are placed.

The process of excavating a depth equal to the length of the lagging, setting the lagging in place and making secure by rings, is carried on rapidly until the bottom of the caisson is reached.

Precautionary Methods.
The greatest possible care should
be exercised in digging the caissons not to remove any more earth than is absolutely necessary and to carefully maintain the diameter the entire depth of each caisson. The general excavation should be kept well inside the face of the lagging until the required depth has been reached, after which the banks are to be neatly trimmed to the proper radius for the setting of the lagging. These foregoing instructions should be strictly enforced but should any voids or cavities occur back of the lagging or where by any chance the line of excavation has been exceeded, the surface of the earth is to be padded with clay, thoroughly pounded into the bank until it is brought to the proper line.

Where the soil is soft and yielding or inclined to run, shorter lengths of lagging and extra heavy rings are used to prevent any bulging or caving that is liable to occur. Where extreme conditions arise or where the ground is subjected to pressure from adjoining buildings, the caissons are bulkheaded. Double thicknesses of lagging are used and braces cut to fit the radius of the caisson are installed with
the necessary drums and jackscrews to hold the braces tightly in place against the lagging. Two sets of these braces are generally used to each set of lagging.

Method of Concreting. After the shaft has reached rock, the bottom is entirely uncovered and leveled off and cleaned, and in most cases each alternate caisson is drilled to a depth of about 8 feet. The rock is examined by the superintendent, and if satisfactory the caisson is immediately concreted. The concrete is dumped from wheelbarrows or cars as rapidly as it is mixed, and should be dropped perpendicularly down the center of the shaft so as not to strike the sides of the caisson. As soon as the concrete has reached the level of the first ring, workmen descend, remove the ring and level off the concrete. This concreting is continued until the next ring is reached, the ring removed and the concrete leveled off. This process is continued until the caisson is filled with concrete to the proper height. Where soft earth or special conditions arise, the rings are left in and, it is now becoming the practice to leave the rings in
regardless of the nature of the soil. After the stone concrete is finished to the proper level, the concrete cap is put on. This cap is made of a richer concrete and is reinforced. Screeds, that is, strips of wood equal to the diameter of the caisson in length and about 2 inches wide, are placed in this cap, care being taken to have these strips level. The grillage beams, on which the columns supporting the steel frame work of the building are set, are placed on these screeds.

Factors of Cost.
On the following pages are contained plates J, K, L, and M giving the notation used for the standard caissons and tables listing all cost factors. Thus, having figured or estimated the loading, the size of caissons that are required may be found in the table and by taking off the various quantities an estimate of the cost may be obtained.

Plate J gives the notation used and also contains the calculations for the area and volume in terms of the diameter.
Plate K is the table for the standard caissons to bedrock and Plate L is the table for the standard caissons to hardpan.

Plate M gives the quantities which may be useful for the excavation estimate for hardpan caissons.

Preparation of the Tables. The Quantities.

Diameters.
The diameters given in the tables are from 4'–0" which is practically the smallest diameter that can be considered to 9'-9" for the hardpan caissons and from 4'-0" to 14'-0" for the bedrock caissons, increasing from four feet in increments of four inches.

Capacity.
The allowed stresses in pounds per square inch as given in the Chicago Building Ordinance are:
Capacity at top, considering shaft of 1:2:4 concrete, 400# per sq. inch.
Capacity at bottom for hardpan caissons, 15000# per sq. foot.
Bearing under base plate, 800# per sq. inch.

Rules For Figuring Quantities. The values used in the compliance of the tables are in accordance with the rules given in the handbook of the Builder's Association of Chicago, edition of 1912. These rules were formulated by a joint committee acting from the Western Society of Engineers, the Illinois Architect's Association, and the Chicago Bricklayer's and Stone Masons Union.

Excavation. Owing to grillage in caissons being left at different heights in the same building unit prices for caissons will be computed on excavated contents including necessary wood lagging and rings for same. Cubic contents of excavation of caissons to be computed from top of first set of lagging to bottom of caissons and from outside to outside of lagging. If steel or other special casing is required same is to be paid for additional at special unit
rice per pound.

Bells. The area of bottom of bell is multiplied by the height of the bell to neck for cubic contents.

Cubic Contents. For caissons 7'-0" or more in diameter estimate actual contents from outside to outside of lagging. For other diameters observe the following:

Dia. 7'-0" to 6'-6" inclus. add 5% to actual contents.
" 6'-6" to 6'-0" " " 15% " " "
" 6'-0" to 5'-6" " " 25% " " "
" 5'-6" to 5'-0" " " 35% " " "
" under 5'-0" " " 50% " " "

No deduction is to be made for cubic contents of metal embedded in concrete.

Concrete Filling. Concrete for filling of caissons is to be computed on actual contents per cubic foot of concrete, but no deduction to be made for any metal embedded in same.
Weight of Hooping Steel. Square deformed bars are used for the hooping steel.

Hoops are lapped 3'-0". The weight of $\frac{3}{4}$" square bars is 1.913 pounds per foot.

Lagging. For lagging figure length of shaft from top of lagging to neck of bell. Lagging is generally 2" or 3" thick, 6" wide, and from 4 feet to 6 feet in length.

To determine the amount of lagging, board measure per foot of caisson, multiply the thickness by the circumference of the caisson and add one-fifth of the product to provide for the tongue and groove.

Cost Estimate.

Caisson Excavation. The excavating for caissons is dependent to a great extent upon the size and depth of the well and upon the kind of digging that is to be encountered. In most cases a test well will be driven and the estimate based on the
test well representing average conditions but it should be noted that test borings do not give thorough reliable results, and only the actual excavation discloses the conditions.

The hardpan stratum is reached at an average depth of 60 feet and bedrock at the average depth of 90 to 100 feet.

The price per cubic yard for small wells is relatively higher because of the crowded quarters in which the men have to work. Thus the number of men required on each caisson will depend upon the size of the well and upon the method of handling the excavated materials. The average number, however, is from 5 to 6 men to each caisson apportioned as follows: two men or "diggers" in the well, one man on top at the "niggerhead" and one man dumping the bucket, the excavated material being removed to the wagon by a fifth man.

Many conditions influence the yardage excavated per 8 hour shift but it may be assumed that two diggers in the hole will average from 6 to 8 cubic yards
per 8 hour shift, this average being considerable higher thru the soft clay stratum and somewhat lower in the hardpan and harder soils where grubbing is necessary. The estimate will also depend greatly upon whether or not water is to be encountered and as it is generally costly when encountered it is best to make some provision for this contingency.

The wages of caisson laborers based on the 1916 Chicago Union wage scale are as follows;

Men working in hole, or diggers,... 62½¢ per hr.
Man at top, niggerhead men,......... 55 ¢ per hr.
Common laborers handling and removing the excavated earth,............42½¢ per hr.

The teaming concerns specializing in earth disposal charge from 50 to 75 cents per cubic yard for hauling excavated earth from the premises, a like sum being charged by the Tunnel Company.

Lagging.
The wood lagging, 2" or 3" by 6" by five foot lengths, is worth $19.00 to $20.00
per 1,000 feet board measure.

The labor cost of placing lagging averages from $9.00 to $12.00 per 1,000 feet board measure, with $10.00 a fair price, making a total cost of $30.00 per 1,000 feet of lagging in place.

Iron Rings For Caissons. The iron rings used in caisson work are made from 3/4 by 3 inch iron and are worth from 6 to 7 cents per pound.

Approximate Cost of Caissons. For our example we will take a 5'-6" caisson going to a depth of 7 feet six inches per 8 hour shift, or removing approximately 8 cubic yards:

16 hours "diggers" at 62½ ¢ ............ $10.00
8 hours "niggerhead" man at 55 ¢ ...... 4.40
24 hours common laborers at 42½ ¢ ...... 10.20
Overhead, foreman, etc. ................. 2.00
Removing earth from premises, 8 cu. yds.

at $.50 .................................. 4.00

Making a total labor cost of $ 30.60
Approximately $3.85 per cubic yard for the excavating only.

The lagging for the above would be worth about $6.00 for the material and $3.00 for labor, or $9.00 in place.

The concrete will cost about $3.75 for material and $1.50 for labor or $5.25 per cubic yard making a total of $42.00.

The total cost of the caisson amounts to about $85.00, or approximately 40¢ per cubic foot.

An average price for caissons complete, including excavating, lagging, and concrete is 45 cents per cubic foot.
# STANDARD CONCRETE PIERS TO BEDROCK

**Concrete - 1:2:4**  
**Lagging - 3"x4.0" lg.**  
**Hoops Lapped - 3"**

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<th>Diameter of Short</th>
<th>Capacity in 1000s</th>
<th>Diameter of Base plate</th>
<th>Cross-section</th>
<th>12' conc. per ft. of pier in short length</th>
<th>Depth of cap</th>
<th>Vol. of cap in cu ft</th>
<th>No. and size of hooping of bars</th>
<th>Weight of hooping steel - lbs</th>
<th>17' G.M. per ft.</th>
<th>4 J. rings</th>
<th>No. of Center</th>
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Lawrence J. McHugh
## STANDARD CONCRETE PIERS TO HARD PAN

**Concrete** ~ 1:2:4  
**Lagging** ~ 3' x 4'-0" Lg.

### Hoops Lapped 3' 0"  
**Vert. Spacers** ~ 4'-0" C.C.

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<th>Diameter of Short</th>
<th>Capacity in lb at gage</th>
<th>Diameter of boss plate</th>
<th>Cross-section Square Sq. Feet</th>
<th>Bottom area in Sq. Feet</th>
<th>Capacity in 5000 @ 2000</th>
<th>Bottom area in Sq. Feet</th>
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<th>Slope of vertical of bell ext.</th>
<th>Length of bottom</th>
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## Standard Concrete Piers to Hard Pan

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| M | Lawrance J. McHugh |
The illustration shown on Plate N is a record of borings and caissons, giving a representation of the soil supporting the high buildings of Chicago. It will be seen that, for a depth of 75 feet below street level, the soil underlying the business section shows no great variation. Below this, however, and down to bed rock, great variations are found. South of the business section and extending as far 12th Street, the really treacherous soils occur, running sand and silt being encountered.

Referring to the plat:— Boring No. I gives a representation of the soil in the vicinity of Wabash Ave. and Peck Court; Boring No. 2 is typical of the soil at the Heisen Building, corner of Harrison and Dearborn Streets; Boring No. 3 was taken at the Monroe Building, corner of Monroe St. and Michigan Ave.; Boring No. 4 was taken at the North American Building, State and Monroe Streets; Boring No. 5 shows the soil at the Montgomery and Ward Building, Chicago Ave. and the Chicago River.

The upper stratum extending to about 50 to 60
feet below street level, is of soft blue clay. This blue clay is soft and yielding, containing sand and gravel pockets, and is the best digging encountered. It gradually becomes, and finally merges, into the tough blue clay stratum. The latter extends about 10 feet, where it merges into a hard pan stratum. The hard pan consists of loosely cemented dry clay and small gravel. This stratum is very hard, and in many instances the foundations are carried only to this soil. Below the hard pan the stratification varies; but in the business section it overlies beds of flaky clay and gravel. Water is generally encountered before the rock is reached, and in many cases rises so as to put the work in danger. Boulders may be found upon the rock and often the so-called shell rock is run upon. Shell rock, as the name implies, is but a shell of rock overlying beds of gravel and clay. The bed rock stratum is so irregular that the determination of its depth without actual testing is guess work. In many instances a ten foot variation in depth occurs within a radius of a few feet.
CAISSONS

SHOWING NATURE OF SOIL

DRAWN BY C. F. MORSE 18
"FIRST FLOOR FRAMING PLAN"

"DESIGN OF A 14-STORY OFFICE BUILDING.

HOMOSAIC INSTITUTE OF TECHNOLOGY

(Stamp)
- Typical Floor Framing Plan -

Design of a 14-Story Office Building

Part of thesis presented for the degree of Bachelor of Science in Civil Engineering

Submitted by [Signature]

Approved by [Signature]