NOTES ON TALL BUILDING CONSTRUCTION.

BY

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THESIS.
FOR THE DEGREE OF

IN CIVIL ENGINEERING.

University of Illinois.

1892.
Notice on Tall Building Construction.

The object of this thesis is to present some features of current practice in designing the metal-skeleton type of buildings.

The matter will be presented under the following heads:

I. Determination of the loads,
II. Footing courses,
III. Base-plates or stubs,
IV. Columns
V. Wind-tracing,
VI. Office Detail.
I. Determination of the Loads.

In tall buildings on compressible soils not all the live load for which the floor beams are designed is taken as the load on the footing course, or on the clay beneath. Though each floor is likely sometime to be loaded to its full capacity, it is unlikely that all the floors will be so loaded simultaneously. When Richardson, the famous architect, designed Marshall Field's mammoth warehouse in Chicago, he calculated the footing course for the full assumed live load, and the result was the wall having footings proportioned for loads actually on them settled much more than the interior of the building where the footings were loaded with only a portion of the live load for which they were proportioned. Possibly this error was due to the fact that Richardson was a New York City architect and was accustomed to designing under the following clause of the building law of that city:

"Every column, post, or other vertical support shall be of sufficient strength to bear safely the weight of the portion of each and every floor dependent upon it, in addition to the weight required as above to be supported"
safely upon said portion of said floors. In New York City the founda-

tion rests on rock, and there the principle that the area of the foundations
should be in direct ratio to the loads may be somewhat disregarded;
but in Chicago with its compressible soil this principle is of vital im-
portance.

The following table gives the live and dead loads
for each floor in The Fair building, Chicago.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Live load</th>
<th>Dead load</th>
<th>Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>120</td>
<td>93</td>
<td>Store</td>
</tr>
<tr>
<td>2nd</td>
<td>220</td>
<td>95</td>
<td>Heavy storage</td>
</tr>
<tr>
<td>3rd</td>
<td>120</td>
<td>93</td>
<td>Storage</td>
</tr>
<tr>
<td>4th</td>
<td>75</td>
<td>92</td>
<td>Light Storage</td>
</tr>
<tr>
<td>5th and above</td>
<td>75</td>
<td>92</td>
<td>Offices</td>
</tr>
<tr>
<td>Roof</td>
<td>40</td>
<td>50</td>
<td></td>
</tr>
</tbody>
</table>
The following per cents of live load were employed in proportioning the floor beams, girders, columns, and footing beams. Floor beams calculated for full dead load + full live load.

Floor girders
Columns
Footing courses
Load on clay

+ %
+ %
+ %
only,

These percentages would vary with the use for which a building is designed. Partitions, vaults, etc. are included in the dead load.

The table on the next page gives the loads in detail for each floor in The Fair. It will be seen that the percentage of live load added in the table for each floor is such that the total live load added from the top down to the floor in question is supposed to be greater than the average of any probable loading on these floors.
<table>
<thead>
<tr>
<th>Story</th>
<th>Load on floor beam</th>
<th>Total live load on floor for the area supported</th>
<th>Percentage of live load on columns</th>
<th>Live load of each floor transmitted to columns in lbs. per sq. ft. of area supported</th>
<th>Total live load on cols. at each story in lbs. per sq. ft. of area supported</th>
<th>Average live load which cols would support in each sq. ft. of area supported</th>
<th>Use of each floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Attic</td>
<td>10</td>
<td>20</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>Roof</td>
</tr>
<tr>
<td>16th</td>
<td>75</td>
<td>115</td>
<td>87</td>
<td>60</td>
<td>100</td>
<td>8</td>
<td>Office floor</td>
</tr>
<tr>
<td>15th</td>
<td>75</td>
<td>190</td>
<td>81 1/2</td>
<td>55</td>
<td>155</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>14th</td>
<td>75</td>
<td>265</td>
<td>78</td>
<td>52</td>
<td>207</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>13th</td>
<td>75</td>
<td>340</td>
<td>76</td>
<td>48</td>
<td>265</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>12th</td>
<td>75</td>
<td>415</td>
<td>72</td>
<td>44</td>
<td>299</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>11th</td>
<td>75</td>
<td>490</td>
<td>69</td>
<td>40</td>
<td>339</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>10th</td>
<td>75</td>
<td>565</td>
<td>66 1/2</td>
<td>37</td>
<td>376</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>9th</td>
<td>75</td>
<td>640</td>
<td>64</td>
<td>32</td>
<td>408</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>8th</td>
<td>75</td>
<td>715</td>
<td>61</td>
<td>29</td>
<td>437</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>7th</td>
<td>75</td>
<td>790</td>
<td>58 1/2</td>
<td>26</td>
<td>462</td>
<td>8</td>
<td>Firestairage</td>
</tr>
<tr>
<td>6th</td>
<td>75</td>
<td>865</td>
<td>56</td>
<td>22</td>
<td>484</td>
<td>70</td>
<td>Storage</td>
</tr>
<tr>
<td>5th</td>
<td>130</td>
<td>995</td>
<td>83 1/2</td>
<td>47</td>
<td>531</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>4th</td>
<td>100</td>
<td>1195</td>
<td>80 1/2</td>
<td>70</td>
<td>606</td>
<td>140</td>
<td>Reception</td>
</tr>
<tr>
<td>3rd</td>
<td>130</td>
<td>1325</td>
<td>68</td>
<td>33</td>
<td>688</td>
<td>76</td>
<td>Store</td>
</tr>
<tr>
<td>2nd</td>
<td>130</td>
<td>1455</td>
<td>66 1/2</td>
<td>26</td>
<td>664</td>
<td>76</td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>130</td>
<td>1585</td>
<td>63</td>
<td>19</td>
<td>683</td>
<td>76</td>
<td></td>
</tr>
<tr>
<td>Basement</td>
<td>130</td>
<td>1715</td>
<td>44 1/2</td>
<td>31</td>
<td>714</td>
<td>76</td>
<td></td>
</tr>
</tbody>
</table>
Notice that the total live load which the floor beams are capable of supporting is 1715 lbf. per sq. ft. of the tributary area of the building, while the columns in the basement are proportioned to support only 714 lbf. per sq. ft. of the tributary area of the building, or only 41 1/2 per cent. of the load which the floor beams would safely support.

II. Footing Courses.

Owing to the compressible soil and to the many tall buildings which have been erected there, the design of foundations has been very fully developed in Chicago. It is universally conceded that in Chicago the designing of isolated piers or separate footings for each stack of columns is the only means of keeping the floor lines level. This method of designing foundations is aptly termed the Chicago Construction. That this construction serves the purpose was amply demonstrated in the first building of the kind erected in Chicago, the Home Insurance, in which the greatest inequality in settlement was 3/16 in. in a total settlement of 3 in.
Formerly cut-stone footing courses were employed, but owing to their expense and more especially to the amount of space occupied in the basement, steel rails were substituted instead. Owing to a subsequent decrease in price of steel beams, these have been substituted to a considerable extent in the place of the rails. As there has been some dispute about the proper method of making the calculations in the design of steel beam footing, we will present two solutions of the problem.

1. Energetic Method.

The following method is the one employed by Mr. Jenney, architect of The Fair, Chicago, in designing the footings for that building, and was quoted, apparently with approval, by Mr. Purdy, of Purdy and Tullidge, engineers for the contractors for the erection of the building, in Engg. News, Aug. 8, 1891, p. 116, Vol. XXXI.

One of the footing courses of The Fair building is shown on p. 8.

1st Course. Let us assume a projection of 5'-0" at opposite side of the steel and also assume that 9'-0" I beams are used and let us compute their weight. The projecting portion is a cantilever and we will assume, for the present at least, that this course is acted upon
by a uniform pressure from below. The total reaction against the bottom of this course is equal to the load upon the steel plus the weight of the course under consideration. The course is $5\times (5' 4" + 5' 6" + 5' 4") = 188.4$ sq. ft. Its thickness is 15" and the content is 98.0 cu. ft. Assuming a cu. ft. of concrete to weigh 150#, the weight of concrete in this course is approximately $150 \times 98.0 = 14,700$. # Assuming, for the moment, that the metal in
this course is as stated in Fig. 1, p. 5, its weight will be 8,460. The total weight of this course then is 14,700 + 8,460 = 23,160. - 116 tons = say 15 tons.
The load at the base of the stool = 680 tons. Therefore the total weight on the bottom of this course is 680 + 12 = 692 tons. The pressure per sq. ft. on the base of this course is 692 / 78.4 = 8.8 tons.

Set \( W \) = the load per sq. ft. on the cantilever, in tons.
\( l \) = the length of the cantilever, in inches.
\( b \) = the breadth " " " " "
\( S \) = the unit-working fiber strain, which we will assume at 5. tons per sq. in. for steel rods and 10 tons per sq. in. for rolled steel. I beam, and 0.1 tons per sq. in. for concrete.
\( I \) = the moment of inertia.
\( c \) = the distance of the most remote fiber from the neutral axis.

Then by the well-known law of the mechanism of material,
\[
\frac{Wb}{2} = \frac{Sl}{c}
\]
Substituting from above,

\[
\frac{3.8 \times (14.)^2 \times 60}{144 \times 2} = \frac{10}{7.5}
\]

from which \( I = 56.3 \), which is the required moment of inertia for the total number of beams. If 9 beams are used, as in Fig. 1, p. 8, each must have a moment of inertia of \( \frac{56.3}{9} = 6.257 \), which is, according to Carnegie Pocket Companion (edition 1892) p. 137, the moment of inertia for a 15° 89° steel beam. This is a reasonable agreement with the results given in Fig. 1, p. 8.

2nd Course. The area of the course is \( 15.5' \times 10.0' = 155.7 \text{ sq. ft.} \). The thickness of the course is not stated, but we will assume it at 0.4', which is approximately the height of a railroad rail weighing 35 lb. per ft. The weight of the concrete in this course then is \( 155.7 \times 0.4 \times 150 = 9402 \text{ lb.} = 4.7 \text{ tons.} \). Assuming, for the moment, that the metal in this course is as stated in Fig. 1, p. 8, its weight is \( 36 \times 25 \times 10 = 9000 \text{ lb.} = 4.5 \text{ tons.} \). The volume of these rails is \( 7.5 \text{ sq. in.} \times 150 \times 36 = 324000 \text{ cu. in.} = 18.8 \text{ cu. ft.} \). The weight of a volume of concrete equal to the volume of the rails = \( 18.8 \times 150 = 2820 \text{ lb.} = 1.4 \text{ tons.} \). The weight of this course then is \( 4.7 + 4.5 - 1.4 = 7.8 \text{ tons.} \). The weight on the
base of the course immediately above was 692 tons. Therefore the weight on the base of the course under consideration is 692 + 78 = 699.8 = say 700 tons.

The pressure on the base of this course is \( \frac{700}{156.7} = 4.5 \text{ tons per sq. ft.} \)

Substituting the values for the 2nd course in eq. 9 on p. 9, we have

\[
\frac{4.5 \times (3.0)^2 \times 185}{14.4 \times 2} = \frac{81}{2.4} = (3)
\]

from which \( I = 791 \), the moment of inertia required for the total number of rails. If 36 rails are used, the moment of inertia of each rail is \( \frac{791}{36} = 22 \). This is a close agreement with the moment of inertia used in obtaining the results given in Fig. 1, p. 8.

3rd Course. The area of the course is 10.0 \( \times \) 31.0 = 310 sq. ft.

Assuming the thickness of this course to be the same as that of the course above, or 0.4, the weight of the concrete in the course is 210 \( \times \) 0.4 \( \times \) 150 = 12,600 = 6.3 tons. Assuming the metal in this course to be as stated in Fig. 1, p. 8, its weight is 50 \( \times \) 25 \( \times \) 21 = 10,500 = 5.3 tons. The volume of the rails is 7.5 \( \times \) 25 \( \times \) 20 = 37.8 cu. m. = 21.8 cu. ft. The weight of a volume of concrete equal to the volume of the rails is 21.8 \( \times \) 150 = 3,270 = 16 tons.
The weight of this course then is \(6.3 + 5.3 - 1.6 = 10\) ton. The weight on the 12 base of the course immediately above was 7.06 ton; therefore the weight on the base of the course under consideration = 7.06 + 10 = 17.06 ton. The pressure on the base of the course is \(\frac{7.10}{14.4} = 0.4\) ton per sq. ft.

Substituting the values for the 3rd course in eq. (1) we have

\[
\frac{3.4 \times (3.2)^2 \times 150}{14.4 \times 2} = \frac{81}{2.4} = 34.6
\]

from which \(I = 4.35\). If 20 nails are used the moment of inertia of each nail is \(\frac{4.35}{20} = 0.217\)

4th Course. The area of the course is 21.6" x 17.10" = 374.4 sq. ft. Assuming the thickness of this course to be the same as that of the course above or 0.4", the weight of the concrete in the course is \(374 \times 0.4 \times 150 = 22,440 = 11.2\) ton. Assuming the metal in this course to be as stated in Fig. 1, p. 8, its weight is \(25 \times 17.13 \times 50 = 2,229 = 11.1\) ton. The volume of the rails is \(7.5 \times 3.14 \times 50 = 80,120\) cu in. = 46.4 cu. ft. The weight of a volume of concrete equal to the volume of the rails = \(46.4 \times 150 = 6,960\) = 3.5 ton. The weight of this course then is 11.2 + 11.1 - 3.5 = 18.8 ton.
The weight on the base of the course immediately above was 710.0 tons; therefore the weight on the base of the course under consideration was 710.0 \times 18.8 = 728.8 tons. The pressure on the base of the course then is \( \frac{728.8}{3744.4} = 1.94 \text{ tone per sq. ft.} \)

Substituting the values for the 4th course in eq. (1), p. 9, we have

\[
\frac{1.94 \times 47^2 \times 252}{1444 \times 2} = \frac{81}{2.44} = 5.35
\]

from which \( I = 11.22 \). If 50 nails are used the moment of inertia of each nail is \( \frac{11.22}{50} = 0.224 \). This agrees closely with the moments of inertia of the nails in the two courses above.

5th Concrete Course. The weight at the base of the course immediately above is 728.8 tons. The area of the concrete is \( 12 \times 18.83 = 414.3 \text{ sq. ft.} \), and the thickness given is 12"; therefore the content is \( 414.3 \text{ cu. ft.} \), and its weight is \( 414.3 \times 150 = 62150 \text{ lb.} = 31.1 \text{ tons.} \) The total weight at the base of the concrete course is 728.8 + 31.1 = 759.9 = say 760 tons; and the pressure per sq. ft. is \( \frac{760}{414.3} = 1.83 \text{ tons.} \)

From Baker's Masonry Construction p. 201, we have the formula,
\[ p = t \sqrt{\frac{P}{41.6}} \] (6)

\( P \) = the pressure in tons per sq. ft. at the bottom of the footing course under consideration;

\( P \) = the modulus of rupture of the material in lbs. per sq. in.;

\( p \) = the greatest possible projection of the footing course, in inches;

\( t \) = the thickness of the footing course, in inches.

Substituting the values for the concrete course, with a factor of safety of 10,

\[ p = 12 \sqrt{\frac{20}{41.6 \times 1.83}} = 12 \times 0.57 = 6.8 \text{ in.} \]

This is a close agreement with the offset, 6 in., given in our example.

2. Correct Method.

The preceding method of solution is correct so far as finding the proper offset for a single footing course, as for example in finding the bending moment of the concrete course about any point on the section CD, Fig. 2. But it is incorrect if applied to find the bending moment of the two bottom courses about any point in the section EF.
The error is due to the fact that in finding the effect of the next to the bottom course the pressure was considered as uniformly distributed between the sections CD and EF, while really it is distributed uniformly between the sections AB and EF; therefore its arm is longer than assumed in the above computation, and hence the preceding method obtained too small a bending moment, and consequently too small a resistance.

He will compute the material required in the section EF, assuming that the area of the concrete has been determined by the safe bearing power of the soil, and that therefore any readjustment must be made in the course of十多年 immediately above the bed of concrete.
The load upon the clay is 1.83 tons per sq ft. The horizontal distance from AB to EF is 4.5" and the dimension perpendicular to the plane of the paper is 21.0".

Making the proper substitution in eq. 1, p. 9, we have

\[
\frac{1.83 \times (53)^2 \times 264}{144 \times 2} = 447.2 \text{ in. ton} = \frac{4472}{4000} \text{ ton}
\]

the second member of which represents the sum of the resisting moments of the concrete course and of the course of rails. The resistance of the concrete course is equal to

\[
\frac{20 \times \frac{1}{2} \times 2.64 \times (12)^3}{2 \times 12} = 126.730 \text{ in. lbs.} = 63.44 \text{ in. ton}
\]

The difference between 447.2 and 63.44, or 444.76 in. ton, is the resisting moment required of the course of rails. The resisting moment of the whole course of rails, the \(\frac{4472}{4000}\) as computed on p. 9, is equal to

\[
\frac{50 \times 6 \times 22}{2.4} = 3667 \text{ in. ton}
\]

which is only 79% of the strength actually required. The safe unit fiber strain on the metal was assumed at 16,000 and the preceding computation show that the strain really is \(\frac{3667}{4472}\) = say 81,100 per sq in., which, though undesirably large, may possibly be safe.

Set us compute the bending moment about the section...
Substituting in eq. (1), p. 8, we have
\[
\frac{1.83 \times (53)}{2.644} = 11.554 \text{ in. tons} = \frac{11}{c}
\]

The resisting moment of the concrete is as before 63.4 in. tons. The resisting moment of the lower course of raile is 3.667 in. tons, but we will assume, for a purpose made apparent farther down, that the lower course of raile has a resistance of 4,649 in. tons, that required by the computations of the preceding paragraph. The resistance required of the upper course under consideration then is 11,554 - (63.4 + 4,649) = 6,842 in. tons. The resisting moment of the 3rd raile in this course is \(\frac{36 \times 8}{2.4} = 2,640\) in. tons, which is only 39% of its required resistance.

As the footing stands in Fig. 2, p. 15, the resistance at the section E F is only 34% of that required. If the deficiency of the lower course of raile is made good, as we have just assumed, then the second course is only 39%, as above, of what it should be.

Set us apply this method of solution to the two remaining courses. Fig. 3 is a section perpendicular to the section in Fig. 2.
Substituting in eq. (1), p. 9, we have for the bending moment about the section PQ:
\[
\frac{1.83 \times (38)^2 \times 226}{144 \times 2} = 2074, \text{ in. tone} = \frac{SI}{C},
\]
the second member of which represents the resisting moment, in this case consisting of the sum of the resistance of the concrete course and the third course of rails. We have already found the resisting moment required of the concrete to be 63.4 in. tons. Therefore 2074 - 63.4 = 2011, in. tone is the resistance required of the third course of rails. The resisting moment of the rails is \(\frac{20 \times 8 \times 22}{2.4} = 1467\) in. tone, which is only 70\% of its required resistance.

He will next compute the bending moment about the section P Q. Substituting in eq. (1), p. 9, we have:
\[
\frac{1.83 \times (102)^2 \times 226}{144 \times 2} = 14940, \text{ in. tone} = \frac{SI}{C^3},
\]
the second member of which represents the resisting moment, in this case consisting of the sum of the resistances of the concrete, the third course of raile, and the course of I beams or top course. We have already found the resisting moment required of the concrete to be 63.4 m. tons, and of the third course to be 201 m. tons; therefore the resisting moment required of the course of I beam is $14,940 - (201 + 63.4) = 12,866$ m. tons. The resisting moment of the I beam is $\frac{9 \times 10 \times 6440}{7.5} = 7,728$ m. tons, which is only 60% of its required strength.

The difference between the preceding method and the one followed by Mr. Saucier (in reviewing Mr. Purdy's presentation of the preceding method) in Eng'g News, Sept. 19, 1891, Vol. XXII, is that we have considered the concrete to be as much a course as the raile and beam, and have dealt with it accordingly; whereas Mr. Saucier calculated the resistance only of the courses of beam and raile. Both Mr. Saucier's method and the one here presented show that the foundation as designed does not possess adequate strength. While we have not the time to compute the resistance due to the friction between the annual courses, it is highly
probable that this friction between the courses does not amount to enough to appreciably affect the strength of the footing.

III. Face-plate or Stool.

The safe load at the base of stool are generally taken as follows:

Granite block——-1,000 lbs. per sq. m.
Brickwork——-200 lbs. 
Concrete——-150 lbs.

The sides may all be of the same thickness, though the diagonal ribs are usually made somewhat thicker than the others. The rule is sometimes followed of the thickness of metal the same throughout. For cast-iron columns the thickness of metal in the stool should be that of the metal in the column. It is good practice to make the height $h$, see Fig. 4 p. 31, equal to the projection of the rib $r$. The top flange $t$ is usually $3$ in. wide. Blow-holes are usually left in the position shown by small circles in Fig. 4. If a stool were cast as shown in Fig. 5, p. 21, only one blow-hole would be left in the middle, the smaller holes shown being used for grouting.
The latest practice in casting stools is to avoid unequal effects of shrinkage by means of the rib A, Fig. 11. Some have advocated casting flanges as at C, Fig. 11 around the edge of the plate for this purpose. This is not often done, though it undoubtedly serves the purpose of keeping the edge straight during cooling. The ribs A are made of the same thickness as the other ribs, and may either slope off as drawn in Fig. 11 or be continued straight across to the diagonal ribs. To save coring, the part A, where the diagonal and side rib meet, is cast solid instead of hollow as between the two side ribs.

There is considerable dispute about the proper mode of computing the strength of stools, some considering that the weakest section is A B, Fig. 11, others that the weakest part is the diagonal rib.

1st Method. The following is the work of computing the strength of a stool after its dimensions have been fixed by estimation.

Assuming the section A B, Fig. 11, p. 213, to be the weakest section, if we find that the safe resisting moment is equal to the bending moment, the stool has sufficient strength. In making this investigation for simplicity only half the section is taken, as is shown in
Fig. VI.

Half of Section on A.B.

Scale \( \frac{1}{4}" = 1' \)
the sectional elevation. From the laws of the resistance of materials,

\[ \frac{M}{S} = \frac{I}{c} \]  (7)

or \[ M = \frac{I}{c} S \]  (8)

\( M \) = the bending moment in inch-lbs.

\( I \) = the moment of inertia.

\( c \) = the distance of the most remote fiber from the neutral axis.

\( S \) = the modulus of rupture in lbe. per sq. in.

Let us take the modulus of rupture at 36,000 lbe. per sq. in., giving a factor of safety of about 10.

\[ M = 300,000 \times \frac{\frac{14}{40} \times \frac{20}{40} \times \frac{14}{2}}{2} = 367,500 \text{ in. lbs.} \]

in which 300,000 is the total load on the stool; \( \frac{14}{40} \times \frac{20}{40} \) is that portion of the load which must be supported on the indicated area, \( 14 \times 20 \); and \( \frac{14}{2} \) is the lever arm.

\[ \frac{I}{c} = \frac{3600 \times 1908.2}{10.25} = 670,500 \text{ in. lbs.} \]

in which 3600 is one tenth of the modulus of rupture; 10.25 is the distance of the most remote fiber from the neutral axis; 1908.2 is the moment of inertia of the section as found below.
I = \frac{1}{12} \left\{ 3.5 \times 10.25^3 + 20 \times 4.75^3 - (20 - 3.5) \times 2.25^3 \right\}

Reducing,
I = \frac{1}{12} (3,769.1 + 2,143.4 - 187.9) = 1908.2

It was the intention of the designer of the stool to have the maximum safe resisting moment exceed the bending moment, but through an error in the computations the moment of inertia here found to be 1908.2, was taken at about half of this, and hence the safe resisting moment 670, 200 is nearly twice as large as 367, 700, the maximum bending moment; or in other words the stool was made twice as large as need be. The stool was cast as drawn.

1st Method. Those who think the weakest section is the diagonal rib, take the two ribs diagonally opposite each other as a continuation of each other, and compute their strength as a simple beam. This supposes the stool to be supported only at the corners. Half the column load is taken as the load on the beam. Formula (D) p. 24 is applicable to this case.

In the application of this formula by a prominent architect of Chicago I was assumed to be equal to \frac{4d^3}{12}, which is equivalent to assuming that the resisting section is a rectangle, whereas it really is of this form \frac{1}{12}.
in which the lower flange is a portion of the bottom plate. Apparently the above expression for the moment of inertia neglects the bottom plate. In the application of this method it would be impossible to determine with any degree of certainty the portion of the bottom plate to be included in the resisting section. Therefore we will not further consider this method of solution.

IV. Columnn.

Following is a brief summary of the arguments for and against different forms of steel column:

1. Phoenix Columnn.
   For,
   (a) A large radius of gyration.
   (b) A symmetrical section, hence practically but one radius of gyration for each column.
   (c) No metal at centre of section.

Against,

(a) Connections not easily made in the field.
(b) Weakness of channel connections.
(c) Objection to guest plate.
do not admit of repainting on inside.

2. I-bar Columns.

For, (a) Means of applying the loads closely to the axis of the column.

(b) Convenience for connection.

(c) Ordinarily only two lines of riveting.

(d) So many pieces that if one happens to fail they cannot all yield.

Against, (a) Metal at centre of section.

(b) Different radii of gyration about axes at right angles to each other.

(c) Not an advantageous section for fire-proofing, particularly if large.

3. Tapered Columns.

For, (a) Only one line of riveting

(b) Convenience for connection.
Economical for a light load.

Against: (a) Inconvenient method of increasing section when a large column is required.

(b) Being composed of but two pieces, a definite fire has proportionally a very great effect on the strength of the column.

4. Columns composed of plates and angles.

For: (a) Convenient for detailing.

(b) Soade applied near the axis.

Against: (a) Eight lines of riveting.

V. Wind-bracing.

Engineers write in declaring wind-bracing to be an essential feature of tall building construction. Many tall buildings, however, have no wind-bracing; others might as well have none; a few only have bracing adequate to resist high winds.

Wind pressure is generally assumed at 20 lbs. per sq. ft. on the surface exposed.

Wind-bracing may be one of three types, as follows:
1. Diagonal tracing rods with turn-bolts are frequently used, and many say they should be universally.

2. Knees or brackets as in ship-building are sometimes used to give the necessary stiffness against wind pressure.

3. In some buildings solid masonry walls run across the structure from top to bottom, and in a few buildings the hollow tile wall is used without any calculations whatever having been made for it.

Most engineers prefer diagonal tracing and struts composed either of floor beams at the floor level or of struts distinct from the floor system. The critical component from the tension rod is the "wind load" on the column. In the Fair the unit working stress for the wind load on the columns was 20,000 lbs for lengths less than 90 radii of gyration, and (25,000 - 57 + lbs. per sq in. for lengths more than 90 radii of gyration.

In the German Theatre, Chicago, where the wind-tracing consists of two walls as well as two lines of diagonal tracing, the wind pressure was taken at 50 lbs. per sq. ft. In determining the
thickness of the two walls a pressure of 200, lbe per sq in. was allowed at the bottom. The walls are about 9 ft thick and laid in cement. In the lines of diagonal bracing I beams in the floor system are placed together for struts. Two vertical lines of wind bracing are between pairs of columns opposite each other, and in the basement the two lines, which are in the same vertical plane with an embayed space between them alone the basement, are connected with each other by diagonal tie rods.

In the Chicago Title and Trust building the only wind-bracing is thru hollow tile walls. The walls are of 8-in. tile built between columns from top to bottom. No calculations were made of the resistance of these walls.

In the Venetian building diagonal bracing is used and each set of bracing was figured to resist a wind force on its tributary area equal to 40, lbe. per sq ft. Of the calculated horizontal when, however, only 70% was taken for the steel bracing, leaving the remainder for the partitions. Four sets of bracing were used and all columns thus traced were made continuous through the basement and
First story. The allowable unit stress on the rods was taken at 5000 lbs.
per sq. in., with no rod less than 7/8 sq. in. square.

VI. Office Detail.

Office calculations, especially for shop work, should invariably be checked and "O.K." before being used. A system of checking is always followed in the offices of the larger iron-contractors. This work can never be done too carefully, and even then it will be found on erecting the material that owing to errors in the office or the ignorance of the men, pieces can not be readily or properly fitted, and a certain amount of punching, drilling, etc., must be done on the building. If the iron-contractor finds it desirable for any reason to make a change, he must before ordering it done secure the superintendent's "O.K."

The method of detailing I beams is practically as follows. The floor-planes as they come from the architect's office must first have all beams and columns designated, the columns on each floor being numbered from unity, and the beams lettered and numbered according to floor and span. It is better to number the beams systematically...
so that having found a particular member on the drawings others can be traced from it. Thus a 30" I beam is commonly lettered A; a 15" I beam, B; a 12" I beam, C; a 10" I beam, D; a 9" I beam, E; an 8" I beam, F; a 7" I beam, G; a 6" I beam, H; and a 5" I beam, I; and a 4" I beam, K.

(Note that the letter I is omitted; the reason is obvious.) A particular 9' 2½" I beam on the first floor might be designated thus: 1 E 9' 2½".

The shop details may be made in various ways:

1. They may be sketched on detail paper and traced, from which, after being checked, they are blue-printed for the machinist and "copier"; 2. they may be sketched in ink direct on tracing cloth, having an I beam drawn in blank on heavy paper beneath, the necessary data having for convenience been set down from the floor-planes; 3. these methods, however, are quite primitive in comparison with the one now in use in most offices. This is as follows: the beams are printed in blank on bond paper, it then being necessary to fill in only the details. The blue-print is of course made direct from this. The conventional signs in building are boxed.

Note that columns are numbered for each story separately.