CARTWRIGHT

Design of a Reinforced Concrete Service Building

Civil Engineering

B. S.

1914
DESIGN OF A REINFORCED CONCRETE SERVICE BUILDING

BY

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THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1914
UNIVERSITY OF ILLINOIS
COLLEGE OF ENGINEERING

May 25, 1914.

I hereby recommend that the thesis prepared under my direction by CHARLES FINDLAY CARTWRIGHT entitled DESIGN OF A REINFORCED CONCRETE SERVICE BUILDING be accepted as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

[Signature]
Assistant Professor of
Civil Engineering.

Recommendation concurred in:

[Signature]
Ira O. Baker
Head of the Department of
Civil Engineering.
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DESIGN OF A REINFORCED CONCRETE SERVICE BUILDING.

INTRODUCTION.

In choosing this subject for a thesis the writer had two objects in view. First, being particularly interested in the subject of reinforced concrete building construction, he desired to become more thoroughly familiar with the details of such construction than it is possible for a student who merely avails himself of the courses offered to an undergraduate, to become. This object he has attempted to attain by obtaining the architect's plans for a four story building, and, imagining himself in the position of the designing engineer, making a complete design of the building and furnishing the engineer's plans necessary for its erection. While the building was completely designed, lack of time has prevented the writer from making a full set of the detailed drawings which would be necessary for its erection, and has forced him to content himself with furnishing only a typical drawing for each of several parts. Second, the writer desired to compare the amount of steel required in a building designed by the Turner Mushroom System with the amount required by a more conservative design. In order to attain this object the writer has made his design along the general lines of the Turner Mushroom System, and, for the purpose of comparison, has obtained the detailed drawings of the same building made by an eminent consulting engineer.
DESCRIPTION OF BUILDING.

The building chosen is a service building which has been erected for the use of the Ford Motor Company at Columbus, Ohio. This building was designed and erected under the supervision of Mr. T. L. Condron, Consulting Engineer, of Chicago, Ill.

The plan of the building is 205 feet 6 inches by 106 feet, as shown by Drawing No. 4. The east and south sides face on public streets. On the north side is a spur track at an elevation of 3.16 feet below the first floor level. On the west side is a vacant lot. Four concrete inclines run from the first floor level to the street level, two on the south side and two on the west side.

The building is four stories high, with a full basement below the street level. The story heights are shown on Drawing No. 2. Two elevators 10 feet 6 inches by 17 feet, one running to the roof and the other to the fourth floor are located as shown on Drawing No. 6. The location of the smoke-stack, 108 feet in height, and of the boiler room, is shown on Drawing No. 4. There are two stairways, as shown on the floor plans, the west stairway running through to the roof and being covered by a skylight, the east stairway stopping at the fourth floor. The entire building is constructed of reinforced concrete. The floor slabs are of the girderless type.

SYSTEM OF DESIGN.

As stated above, the system which the writer has used is this design is essentially the Turner Mushroom system.
The writer has not, however, hesitated to deviate from the Turner system in minor details where it has appeared to him that conditions justified him in so doing as is shown by the following examples.

He has used the Turner umbrella column top, but has also dropped his slab around the column to take care of excessive circumferential shear. The Turner system does not include the dropped slab.

A stirrup with a rod projecting into the slab, as shown on Drawing No. 9, has been used on the spandrel beams, the idea being to partially relieve the stress and to distribute any cracks which might occur due to the negative bending moment at the center of the span along the wall beams.

The straight column rods are butted together and gas pipe sleeves are used as splices. Dowel rods are necessarily used on the column rods which are bent out into the slabs.

The Turner four-way system of reinforcement was used in the floor slabs. The reinforcing rods run from column to column around the sides of the panel in belts approximately seven-sixteenths of the panel length, and two additional belts of the same width run diagonally across the panel. The general method of design in this system consists in providing for the positive bending moment at the center of the panel and in bringing the rods up over the umbrella column top and lapping them to care for the negative bending moment around the support. In computing the positive bending moment at the center of the panel, Mr. Turner recommends the use of \( \frac{WL}{50} \) with a stress of 13,000 lbs. per sq. in. in the steel. This
is equivalent to about $WL/42.5$ with a unit stress of 16,000 lbs. per sq. in. in the steel. The writer, in his design, has reduced this coefficient to $WL/40$ with 16,000 lbs. per sq. in. in the steel. He wishes to state, however, that he fully realizes that the use of any coefficient less than $1/25$ for the bending moment at the center of the panel is not advocated at the present time by the leading engineers of this country.

Art. 1. WORKING STRESSES.

The working stresses used in this design are, as follows:

**Bearing power of soil**.......................... 3 tons per sq. ft.

**1:2:4 Concrete:**

<table>
<thead>
<tr>
<th>Stress Type</th>
<th>Stress Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ratio of moduli of elasticity</td>
<td>1.5</td>
</tr>
<tr>
<td>Compression-extreme fiber-bending</td>
<td>600 lbs. per sq. in.</td>
</tr>
<tr>
<td>Direct compression</td>
<td>500 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>Axial compression (columns, 1% to 4% of vertical steel, 1% of spiral)</td>
<td>650 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>Maximum axial compression (average of concrete and steel) not to exceed</td>
<td>1000 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>Diagonal tension or shearing:</td>
<td></td>
</tr>
<tr>
<td>Without web reinforcement</td>
<td>40 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>With &quot;</td>
<td>100 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>Bond, plain round bars</td>
<td>75 &quot; &quot; &quot; &quot;</td>
</tr>
</tbody>
</table>

**1:1-1/2:3 Concrete:**

<table>
<thead>
<tr>
<th>Stress Type</th>
<th>Stress Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial compression (columns 1½ to 4½ vertical steel, 1½ spiral)</td>
<td>750 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>On exterior columns</td>
<td>650 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>Maximum axial compression (average of concrete and steel) not to exceed</td>
<td>1300 &quot; &quot; &quot; &quot;</td>
</tr>
<tr>
<td>On exterior columns</td>
<td>1000 &quot; &quot; &quot; &quot;</td>
</tr>
</tbody>
</table>
Art. 2. NOTATION.

In the computations to follow the notation given below will be used.

As denotes area of steel.

A " area of concrete enclosed by spiral.
as " area of steel in a given belt.
M " bending moment.
fc " allowable unit compressive stress on concrete.
fs " " stress in steel.
n " ratio between the moduli of elasticity of steel and concrete.

Art. 3. DESIGN OF FLOOR SLABS.

As an illustration of the design of a floor slab, the computations for the design of the panel between columns 13, 14, 23 and 24 on the first floor will be given, as this is a typical interior panel.

The panel is 25 feet by 27 feet.
The live load is 150 lbs. per sq. ft.
A 9 in. concrete base and a 1-3/4 in. cement finish is to be used, the finish to be an integral part of the slab.

Weight of slab equals 1.0 x .9 x 150 equals 135 lbs. sq.ft.

Total load " 150 plus 135 " 285 " " "
Panel " 27 x25 x 285 " 192,000 lbs.
M " 192,000 x 27 40 " 130,000 ft.lbs.
As " 130,000 x 12 16,000 x 7/8 x 9 " 12.32 sq. in.
as equals $12.32/4$ equals $3.08$ sq. in.

Will use $3/8$ in. round rods.

Number of rods required in each belt equals $3.08/0.1104$ equals $28$
Spacing of rods equals $4.7$ in. O.C.

Width of belt equals $7/16 \times 25$ equals $11$ ft.

Design of Slab Around Column.

22 in. column was chosen as this is the smallest diameter of column that occurs under this loading.

For explanation of notation used see Figure 1, page 7.

d equals $192,000/3.1416 \times 10.75^" \times 80$ equals 72 in.
Slab was dropped 4 inches.

d_1 equals $192,000/3.1416 \times 14.75 \times 80$ " 52 "
T_1 " $192,000/3.1416 \times 22 \times 80$ " 32 "

The distance $T_1$ was made 3 feet below bottom of floor slab.

Design of Cantilever Slab Between Columns 10, 27, 28, and 45.

To illustrate this design the computations for the cantilever slab between columns 9 and 10 will be given. The steel from the adjacent panel, running normally and diagonally to the line connecting the center line of columns 9 and 10, will be bent up into the top of the slab after passing the center line of the columns. The band running parallel to the line connecting the center line of the two columns will remain in the bottom of the slab, and the column tops will be extended to give it support, as shown in Figure 2. It is believed that in this way the live load and the weight of the slab may be safely neglected in designing the cantilever. Hence, it is sufficient to add steel in the top of the slab
All belts crossing Cols. 9 and 10 to be raised to top of slab at supports and continued in Top of slab to West Wall.

Note: The only exception to the above is the belt parallel to the West Wall marked "Bottom of Slab."
to carry the weight of the brick wall shown by shaded portion in Figure No. 2. The steel for this portion of the slab was computed as for a true cantilever, with a load per lineal foot equal to the weight of the wall per lineal foot, and a moment arm of 3 feet 6 inches.

Calculations.

Distance between belts perpendicular to wall equals 25 minus 11 equals......... 14.0 ft.

Weight of wall per lineal foot equals 1670 lbs.

M equals 1670 x 3.5 " ...5630 ft. lbs.

As " 5630 x 12/16,000 x 7/8 x 9 " ...0.556 sq. in.

Use 1/2 in. rounds, 4 inches O. C.

The amount of additional steel necessary to carry the weight of this cantilever was computed for the band running between columns 9 and 10 parallel to the wall.

Cantilever Panel in Roof Slab.

While this building is of the girderless type, it was decided to use beams around elevator shafts and stair wells. In these cases the light is necessarily shut off in any type of construction, and the use of beams around the openings was found to be cheaper than the girderless construction. In the roof slab, however, it was found that, on account of the narrow space between the smoke-stack and the west elevator, and the excessive load of the pent-house, 27 feet in height, which had to be carried, the use of beams would result in a narrow, deep, ill-proportioned member. Hence, a cantilever panel with a wide, shallow beam was resorted to. For the location of the pent-house walls contributing the loads, and
Figure 3
beams and bands of steel in the slab, see Figure 3, page 9. 

The loads from the pent house walls are, as follows:

Wall "A" equals. ...
... 72,400 lbs.

" " "B" " ...
... 106,900 "

" " "C" " ...
... 64,500 "

" " "D" " ...
... 54,600 "

Computations.

Beam "B".

Live load equals. ...
... 100 lbs. sq. ft.

Dead " " 1.0 x .8 x 150. ...
... 125 " " "

Total " " ...
... 225 " " "

Panel " " (27 x 25 x 225)/5 ...
... 32,000 lbs.

Wall "D" " 54,600 x 112/274 ...
... 22,300 "

Half of Wall "D" is concentrated at a 
point 8 feet from left support.
The other half is distributed over 19 
feet of beam from right support.......

Wall "B" equals. ...
... 106,900 "

" "B" is distributed over 19 feet 
of beam from right support.......

Weight of Beam "B" equals. ...
... 17,500 "

Total load equals. ...
... 178,700 "

M equals 0.8 of M for simple beam equals 615,000 ft lbs.

As " (615,000 x 12)/16,000 x 7/8 x 30 equals 17.60 sq.in

Use 18/ 1-1/8 in. round rods. Section: 84 in. x 31-3/4 in.

Beam "C".

Panel load equals 25 x 25 x 225)/5 equals.28,100 lbs.

Wall "C" " ...
... 64,500 "

Weight of Beam "C" equals. ...
... 16,200 "

Total load " ...
... 108,800 "

M equals. ...
... 292,000 ft. lbs.
As equals \((292,000 \times 12)/(16,000 \times 7/8 \times 30)\) equals \(6.35\) sq. in.

Use 14.7/8 in. round rods. Section: 84 in. x 31-3/4 in.

Steel Band "A".

Panel load equals \((27 \times 25 \times 225)/5\) equals 32,000 lbs.

" " due to dead load of deepened slab, equals \(85,000\) "

Wall "A" " .......................... 72,400 "

Total load equals .................. 189,400 "

M equals \((189,000 \times 27)/10\) equals .......... 510,000 ft. lbs.

As " \((510,000 \times 12)/(16,000 \times 7/8 \times 20)\) equals .................. 22.0 sq. in.

Use 34, 7/8 in. round rods. Width of band equals 9 feet, 10 inches.

Steel Band "D".

Panel load equals \((27 \times 25 \times 225)/5\) equals 32,000 lbs.

" " due to deal load of deepened slab equals .................. 49,000 "

Wall "D" equals 54,600 x 252/274 equals .... 32,300 "

Total load equals .................. 113,300 "

M equals \((113,000 \times 25)/10\) equals .......... 282,000 ft. lbs.

As equals \((282,000 \times 12)/(16,000 \times 7/8 \times 20)\) equals .................. 12.10 sq. in.

Use 28- 3/4 in. round rods. Width of band equals 8 feet, 6 inches.

The proportion of the weight of Wall "D" supported by the slab on each side of the elevator-door opening, was determined by moments, as shown above.
Steel Band "E".

The slab is considered as cantilevered from a point 5 feet 6 inches from the line connecting the center lines of columns 25 and 26, carrying a uniform load of 360 lbs. per square foot; and the proportion of wall "D" assigned to this part of the slab, as above, is concentrated at the end of the cantilever.

Then:

Weight of wall "D" equals \( \frac{32,300}{6} \) equals 5,400 lbs. lin. ft.

M equals 5,400 x 4.83 plus \((4.83)^2/2\) equals 30,300 ft. lbs.

As " \((30,300 \times 12)/16,000 \times 7/8 \times 20 \) " 1.30 sq. in.

Use 5/8 in. round rods, 3 inches O. C.

Width of band equals 6 feet.

Steel Band "F".

The slab is considered as cantilevered from a point 4 feet from the line connecting the center lines of columns 25 and 30, carrying a uniform load of 360 lbs. per square foot, and a concentrated load of wall "A", placed at a distance of 3 feet from the end of the cantilever.

Then:

Weight of wall "A" equals \( \frac{72,400}{11} \) equals 6,580 lbs. lin. ft.

M equals 6850 x 6 plus 360 x \(9^2/2\) " 55,600 ft. lbs.

As equals \((55,600 \times 12)/(16,000 \times 7/8 \times 20)\) equals......................2.38 sq. in.

Use 7/8 In. round rods, 3 inches O. C.

Width of band equals 13.0 feet.
Art. 4. DESIGN OF BEAMS.

The beams of this building are designed by the straight line formula, neglecting tension in the concrete. Web reinforcement consists of stirrups. Tension rods, where not needed at the bottom of the beam to carry the moment, are bent up and carried over the support to take the negative bending moment at that place. They are extended approximately one fifth of the panel length into adjacent beams where possible, in order to reach the point of inflection. The bent-up rods are not considered in designing the web reinforcement, but are left as an additional factor of safety. Where required, compression steel is placed in the top of the beam. Attention is called to Drawing No. 9 for a detail of a typical beam, and for rod-bending details.

Art. 5. DESIGN OF COLUMNS.

The columns are designed by Turneaure and Maurer's formula, $P = \text{fcA} + \text{p(n-1)}$.

The amount of vertical steel is kept between the limits of 2 and 4 per cent of the area of the concrete enclosed by the spiral hooping. The average percentage of vertical steel used is 2. Spirals are used, but are not considered as taking any direct stress. The area of the spiral steel is maintained at 1 per cent of the area of the column core.

The columns are designed to carry the following loads:

Total dead load, plus

Fourth story ................. 100% of live load.

Third " .................. 90% " " "

Second " .................. 85% " " "

P27.
First story.................. 80% of live load.
Basement.................... 75%, ” “ “

A 1:1-1/2:3 mix of concrete is used in all columns, and a unit compressive stress of 750 lbs. per sq. in. is used on interior columns. This stress is reduced to 650 lbs. per sq. in. in wall columns in order to allow for wind pressure and eccentricity of loading.

The column rods are bent out into the footing to distribute the punching shear, and, on interior columns, are bent out into each floor slab to carry the column top. The straight rods are butted together at each floor and spliced with a gas pipe sleeve. For column schedule, see Drawing No. 1. For column details, see Drawings Nos. 2, and 3, P. 21.

Art. 6. DESIGN OF CURTAIN AND RETAINING WALLS.

The reinforcement in the curtain and retaining walls is arbitrarily assigned by the specifications, as 1/2 in. round rods, 12 in. O. C., horizontally and vertically, in alternate faces of the walls. This steel was put in according to the specifications, but was first checked up, a wind pressure of 30 lbs. per sq. ft. being used in computing the steel for the curtain walls, and an equivalent fluid pressure of 30 lbs. per cu. ft. being used in computing the retaining walls. The steel required by the specifications was found to be sufficient. Drawings Nos. 8 and 9 show typical wall sections.
Art. 7. DESIGN OF FOOTINGS.

In designing the footings, the bearing power of the soil was assumed to be 3 tons per sq. ft. That column which has the highest percentage of live load was chosen, and the area of the required footing determined on the basis of this bearing power of the soil. The unit pressure, due to dead load alone, over this area was then determined, and the area of the remaining footings computed by dividing the total dead load of the column by this unit pressure. The footings were then designed as inverted slabs, carrying a uniform load and supported in the center of the area by a circular column.

The detail of a typical footing is shown on Drawing No. 5. It will be noted that the footing is stepped. Four-way reinforcement is used in the bottom tier, and two-way in the top tier. The steel in the top tier was not computed, but 1/2 in. round rods in the smaller, and 5/8 in. round rods in the larger footings, are arbitrarily placed 6 inches C. C. to assist in distributing the load from the columns. The unit bond stress was slightly excessive, so the rods will be anchored by means of hooks. No web reinforcement is used. A schedule of square footings, showing width, depth and steel required in each tier, is given on Drawing No. 5, page 23.
CONCLUSION.

The writer was unable to obtain a complete set of the original designer's drawings of this building; hence, he is unable to make the best possible comparison of the relative economy of the two designs, based on the total amounts of steel and concrete required by each design. He is, however, able to compare the quantities of steel and concrete required by each design for a floor slab panel, and for two similarly located columns.

The panel between columns 13, 14, 23, and 24, on the first floor, is chosen for this comparison. The original design required for this panel, 3000 lbs. of steel and 22.95 cu. yds. of concrete. The writer's design requires 1733 lbs. of steel and 22.95 cu. yds. of concrete, giving a saving of 42 percent in steel, the concrete being the same for both designs.

Of the columns, column No. 20 is chosen for a similar comparison. Table No. 1 shows the results of this comparison.

Figuring on a basis of relative cost of steel to concrete per unit volume of 80 (a fair estimate figure), the relative cost of the original design to the writer's design of a single panel floor slab is as 1.23 to 1.00, and of column No. 20, as 1.11 to 1.00. The amount of steel used in the walls and footings is about the same in the two designs.

The comparison as to floor slabs made above, shows, as was to be expected, that the Turner system of design is the more economical of the two in the use of steel. In general, the writer believes that it is more economical as to total cost. The comparison made between the two designs of column No. 20...
### TABLE No.1

Showing pounds of steel and cubic yards of concrete required in Col. No. 20 by the original design and by the writer’s design.

| Story | Original Design | | | | | | Writer’s Design | | | |
|-------|-----------------|---|---|---|---|---|---|-----------------|---|---|---|
| 4     | 8            | 13-9” | 5”  | 1.04   | 1140      | 113      | 5.30            | 4            | 19-2” | 1”   | 2.67    | 2050      | 18        | 092            |
|       | 15prl. 13-3” | 5”  | 9.40 | 1250    | 20        | 113      |                 | 4            | 5-0”  | 1”   | 2.67    | 530       | 18        |                |
|       |               |      | 5.30 |         |           |          |                 | 4            | 5-0”  | 1”   | 2.67    | 530       | 18        |                |
|       |               |      |      |         |           |          |                 |              | 5.30  | 1    | 2.67    | 530       | 18        |                |
|       |               |      |      |         |           |          |                 |              | 7.00  | 1    | 2.67    | 530       | 18        |                |
|       |               |      |      |         |           |          |                 |              | 10.0  | 1    | 2.67    | 530       | 18        |                |
|       |               |      |      |         |           |          |                 |              | 12.0  | 1    | 2.67    | 530       | 18        |                |
|       |               |      |      |         |           |          |                 |              | 13-4” | 1”   | 2.67    | 530       | 18        |                |
|       |               |      |      |         |           |          |                 |              | 1 Col. Top | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 |
|       |               |      |      |         |           |          |                 |              | Total | 3390 | 547.0 |      | 547.0 |      |      |      |      |
| 3     | 18            | 15-3” | 5”  | 2.08   | 5700      | 150      | 25              | 8            | 18-2” | 1”   | 2.67    | 388.0     | 150       | 25              |
|       | 15prl. 12-3” | 5”  | 16.00| 196      | 24        | 151      |                 | 8            | 5-0”  | 1”   | 2.67    | 102.0     | 150       | 25              |
|       |               |      |      |         |           |          |                 | 15prl. 12-4” | 5”  | 12.10 | 150      | 25              |
|       |               |      |      |         |           |          |                 | 1 Col. Top   | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 |
|       |               |      |      |         |           |          |                 | Total        | 760.0 | 857.0 |      |      |      |      |      |      |      |
| 2     | 21            | 16-0” | 15”  | 3.38   | 1350      | 115      | 25              | 8            | 18-2” | 15”  | 3.38    | 492.0     | 115       | 25              |
|       | 15prl. 12-3” | 5”  | 19.20| 235      | 28        | 205      |                 | 8            | 5-0”  | 15”  | 3.38    | 1350      | 115       | 25              |
|       |               |      |      |         |           |          |                 | 15prl. 12-4” | 5”  | 22.00 | 272      | 31              |
|       |               |      |      |         |           |          |                 | 1 Col. Top   | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 |
|       |               |      |      |         |           |          |                 | Total        | 1370.0 | 1116.0 |      |      |      |      |      |      |      |
| 1     | 24            | 20-0” | 15”  | 3.38   | 1620      | 160      | 30              | 12           | 22-2” | 15”  | 3.38    | 850.0     | 160       | 30              |
|       | 15prl. 16-3” | 5”  | 29.30| 475      | 32        | 352      |                 | 12           | 5-0”  | 15”  | 3.38    | 173.0     | 160       | 30              |
|       |               |      |      |         |           |          |                 | 15prl. 16-4” | 5”  | 26.60 | 435      | 35              |
|       |               |      |      |         |           |          |                 | 1 Col. Top   | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 |
|       |               |      |      |         |           |          |                 | Total        | 2095.0 | 1675.0 |      |      |      |      |      |      |      |
| Bsmt. | 25            | 15-3” | 15”  | 413    | 15800     | 3600     | 30              | 14           | 20-11” | 15”  | 338     | 990.0     | 3600      | 30              |
|       | 25            | 6-0”  | 15”  | 413    | 6200      | 36      | 314             | 14           | 5-0”  | 15”  | 338     | 2360      | 36        | 314             |
|       | 15prl. 11-3” | 5”  | 3220 | 3620    | 38        | 346      |                 | 1 Col. Top   | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 | 217.0 |
|       |               |      |      |         |           |          |                 | Total        | 2576.0 | 18050 |      |      |      |      |      |      |      |
|       | TOTAL FOR COLUMN- | 71460 | 1135 | | | | | 60000 | 1274 | | | | | |
is not a fair one. It will be noted that the difference in
cost between the two designs is due to the fact that the
original designer contracted his column cores and used a higher
percentage of steel than did the writer. He may have been
forced to do this in order to obtain as small a column as
possible, a restriction which was not placed upon the writer.
As it is a well-known fact that it is cheaper to carry com-
pressive stresses by concrete than by steel, this restriction
(if it existed) would naturally give the writer the cheaper
column.

In conclusion, the writer wishes to say that the use of
the diagonal belts of rods, of the column tops to carry the
slab steel to the top of the slab over the supports, and the
concentration of metal around the supports where it is most
needed to care for the negative bending moment and shear, and
finally the element of economy, make the Turner Mushroom
system appeal particularly to the writer. He, however, realizes
that this method of reinforcing does not lend itself readily
to an analysis of the stresses occurring in the steel, and
that a majority of the leading engineers of the country do not
advocate the use of a coefficient smaller than 1/25 for the
bending moment at the center of the span.