DEPARTMENT OF CIVIL ENGINEERING
BRIGHAM YOUNG UNIVERSITY

DESIGN OF A CEMENT BIN

By

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A PROJECT SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE MASTER OF CIVIL ENGINEERING

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DESIGN OF A CEMENT BIN

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Master of Civil Engineering

by
Chaiyaporn Punyasiri
August 1972
This project, by Chaiyaporn Punyasiri, is accepted in its present form by the Department of Civil Engineering Science of Brigham Young University as satisfying in part the requirements for the degree of Master of Civil Engineering.

Chairman, Advisory Committee

Member, Advisory Committee

8-25-72

Date

Chairman, Major Department
ACKNOWLEDGMENT

I wish to express my gratitude and deep appreciation to Professor Glen L. Enke, for his careful guidance and encouragement in making this project possible.

I dedicate this work to my parents, Rear Admiral Lakshana Punyasiri and Mrs. Chiaranai Punyasiri who have stood behind me and supported me through many years of school.
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acknowledgment</td>
<td>iii</td>
</tr>
<tr>
<td>List of Tables</td>
<td>v</td>
</tr>
<tr>
<td>List of Figures</td>
<td>vi</td>
</tr>
<tr>
<td>Symbols and Notations</td>
<td>vii</td>
</tr>
<tr>
<td>Introduction</td>
<td>1</td>
</tr>
<tr>
<td>Problem Criteria</td>
<td>2</td>
</tr>
<tr>
<td>Problem Detail and Critical Dimension Required</td>
<td>13</td>
</tr>
<tr>
<td>Roof Unit</td>
<td>14</td>
</tr>
<tr>
<td>Calculation Pressure of Bin Wall</td>
<td>22</td>
</tr>
<tr>
<td>Design of Bin Wall</td>
<td>26</td>
</tr>
<tr>
<td>Design of Hopper Bottom</td>
<td>34</td>
</tr>
<tr>
<td>Analysis of Seismic Force</td>
<td>48</td>
</tr>
<tr>
<td>Design of Girder</td>
<td>56</td>
</tr>
<tr>
<td>Design of Column</td>
<td>61</td>
</tr>
<tr>
<td>Foundation Unit</td>
<td>71</td>
</tr>
<tr>
<td>Bibliography</td>
<td>78</td>
</tr>
</tbody>
</table>
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Bending Moment in Walls Spanning Horizontally and Subjected to Horizontal Pressure of Intensity $P_h$</td>
<td>6</td>
</tr>
<tr>
<td>2. Calculation Pressure of Bin</td>
<td>23</td>
</tr>
<tr>
<td>3. Values of $(1 - e^{-x})$ in Which $x = u'k/R \ast h$ in Janssen's Formula</td>
<td>24</td>
</tr>
<tr>
<td>4. The Reinforcement of Bin Wall</td>
<td>29</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Front View</td>
<td>15</td>
</tr>
<tr>
<td>2. Side View</td>
<td>16</td>
</tr>
<tr>
<td>3. Plan</td>
<td>17</td>
</tr>
<tr>
<td>4. Reinforcement of Roof</td>
<td>21</td>
</tr>
<tr>
<td>5. Area Reinforcement Required for Outer Face of Outer Wall</td>
<td>31</td>
</tr>
<tr>
<td>6. Area Reinforcement Required for Inner Face of Outer Wall and Both Faces of Intermediate Wall</td>
<td>32</td>
</tr>
<tr>
<td>7. Detail of Corner in Square Bin</td>
<td>33</td>
</tr>
<tr>
<td>8. Detail at Intersection of Square Bin</td>
<td>33</td>
</tr>
<tr>
<td>9. Reinforcement of Outer Wall</td>
<td>42</td>
</tr>
<tr>
<td>10. Reinforcement of Intermediate Wall</td>
<td>46</td>
</tr>
<tr>
<td>11. Reinforcement of Girder</td>
<td>60</td>
</tr>
<tr>
<td>12. Reinforcement of Column (Upper Part)</td>
<td>70</td>
</tr>
<tr>
<td>13. Reinforcement of Foundation at Long Direction</td>
<td>76</td>
</tr>
<tr>
<td>14. Reinforcement of Foundation at Short Direction</td>
<td>77</td>
</tr>
</tbody>
</table>
SYMBOLS AND NOTATIONS

A = cross-sectional area of a bin
A_C = core area of reinforced concrete column
A_g = gross area of section
A_S = area of tensile reinforcement of beam or column
A_S' = area of compression reinforcement
A_{st} = total area of longitudinal reinforcement
A_V = total area of web reinforcement in tension within a
distance "s" measured in a direction parallel to the
longitudinal reinforcement
a = center to center spacing of hoops
b = width of rectangular beam, or column dimension
C = numerical coefficient
c = distance from neutral axis to extreme fiber
D = normal diameter of bar
D.L. = dead load
d = effective depth of flexure member
E = earthquake load
e = base of natural logarithms or eccentricity
f_C = allowable design stress in concrete
f_C' = compressive strength of concrete
f_s = allowable reinforcement ste 1 stress
f_v = tensile stress in web reinforcement
f_Y = yield strength of reinforcement
H_Z = clear height of column
h = depth below the top of the bin of the point for which the pressure is being calculated.

ho = the clear, vertical span of wall between supports.

h' = effective length

h" = the permissible maximum length of the longer side of any rectangular hoop

I = moment of inertia

j = ratio of distance between centroid of compression and centroid of tension to the depth d

K = numerical coefficient

k = ratio of distance between extreme fiber and neutral axis to effective depth

L.L. = live load

L" = anchorage length

l = length of span

M = bending moment

\( M_b \) = maximum sum of the moment capacity of the beam framing into the top connection

\( M_c^b \) = moment capacity of the column at bottom connection

\( M_c^T \) = moment capacity of the column at top connection

\( M_u \) = ultimate moment

n = ratio of modulus of elasticity of steel to that of concrete

= sum of perimeter of all effective bars crossing the section on tension side

\( \bar{P} \) = design load
\( P_u \) = ultimate load

\( p \) = ratio of area of tensile reinforcement to effective area of concrete in beams and column

\( P_h \) = intensity of the horizontal pressure on the wall

\( P_h \) = normal component

\( P_t \) = tangential component

\( P_v \) = intensity of the vertical pressure

\( R \) = hydraulic radius of bin or ratio of cross-sectional area of fill material to perimeter of the bin or reduction factor for long column

\( r \) = radius of gyration

\( = \) the value of the restrained end

\( S \) = spacing of stirrup

\( T \) = hoop tension or direct tension

\( t \) = total depth of section

\( U \) = required ultimate load capacity of section

\( u \) = allowable bond stress

\( u_u \) = allowable ultimate bond stress

\( u \) = coefficient of friction filling on filling

\( u' \) = coefficient of friction filling on concrete

\( V \) = total shear at section or total lateral load in the case of seismic force

\( V_u \) = ultimate shear at section

\( v \) = shearing stress
\( v' \) = shearing stress taken by web reinforcement

\( v_c \) = allowable shearing stress for concrete

\( W \) = total dead load including partitions for computed lateral force due to seismic force

\( w \) = density of the material stored in the bin

\( z \) = numerical coefficient dependent upon the zone; for

location in zone no. 1, "\( z \)" shall be equal to 1/4; for
location in no. 2, "\( z \)" shall be equal to 1/2; for
location in no. 3, "\( z \)" shall be equal to 1.
INTRODUCTION

Containers used for the storage of wheat, cement, coal, etc., or any granular material are known as bunkers and silos. The essential difference between silos and bunkers lies in the ratio of their dimension, i.e., ratio of height to diameter which governs the design of these structures. A shallow container whose diameter is large as compared to height is termed a bunker. In such a structure the plane of rupture between the wedge which causes maximum pressure and the remaining fill cuts the top horizontal surface, and it does not cut the opposite side of the bunker. On the other hand, if the height of the container is large as compared to its diameter, so that the plane of rupture cuts the opposite side and it does not cut the top horizontal surface, the container is termed a silo. In the case of bunkers the vertical weight is supported by the bottom which is generally hopper shaped, sloping in one or more directions. In the case of silos, as the plane of rupture does not pass through the top horizontal surface, a portion of vertical weight of stored material is supported by the hopper bottom and the remaining amount of vertical weight is supported by the side walls and the stored material.

The exact analysis of these structures is not possible as there are several unknown factors which affect the pressure exerted. Factors such as impact due to filling,
emptying process, and location of discharging hole have considerable effect on design and hence exact analysis is rather difficult.

PROBLEM CRITERIA

The Pressure on the Walls

In the case of the bunker, the vertical walls are subjected to the force of the retained material. The pressure against the vertical wall is generally calculated by Rankine's formula.

The pressure at a depth 'h' is given by

\[ p = wh \cos \alpha \times \frac{\cos \alpha - \sqrt{\cos^2 \alpha - \cos^2 \phi}}{\cos \alpha + \sqrt{\cos^2 \alpha - \cos^2 \phi}} \]

where;

w = density of filling,
\( \phi \) = angle of internal friction.
\( \alpha \) = angle of surcharge.

The pressure acts in a direction parallel to the surface of retained material. Horizontal component

\[ ph = p \cos \alpha \].

In the case of the silo, the method of dealing with the pressure on the wall of deep bins has been investigated by H. Janssen and by W. Airy. Both methods have been, and still are, used extensively for silos made of reinforced concrete or other materials.
Janssen's formula. The derivation of Janssen's formula is not given below. Only the formula is given here.

\[ p_v = wR / \mu k (1 - e^{-\frac{\mu'kh}{R}}) \]
\[ p_h = kp_v = wR / \mu (1 - e^{-\frac{\mu'kh}{R}}) \]
\[ k = \frac{p_h}{p_v} = 1 - \sin \phi / 1 + \sin \phi \]

\[ \phi = \text{angle of repose of filling.} \]
\[ \phi = \text{angle of friction of the filling on the walls of the bin} \]
\[ e = \text{base of natural logarithms, 2.71828.} \]
\[ w = \text{density of material stored in the bin.} \]
\[ R = \text{the "hydraulic mean depth" of the section.} \]
\[ U = \text{perimeter of the section.} \]
\[ h = \text{depth below the top of the bin of the point for which the pressures are being calculated} \]
\[ \mu' = \tan \phi' \]

W. Airy's formula. By Airy's formula it is possible to calculate horizontal pressure per unit length of periphery and position of plane of rupture. Once the horizontal pressure is known, vertical pressure on vertical walls can be calculated and hence vertical load taken by vertical walls can be found.

Case I. where plane of rupture cuts the top horizontal surface and it does not cut opposite side.
\[
\tan \theta = \mu + \sqrt{\mu \times \frac{1 + \mu}{\mu + \mu'}}
\]

\[
P = \frac{wh^2}{2} \times \frac{\tan \theta - \mu}{1 - \mu' \mu + (\mu + \mu') \tan \theta}
\]

**Case II. Plane of rupture cuts the opposite side.**

\[
\tan \theta = \sqrt{\frac{zh}{b} \times \frac{1 + \mu'}{\mu + \mu' + \frac{1 + \mu'}{\mu + \mu'}} \times \frac{1 - \mu'}{\mu + \mu'} - \frac{1 - \mu'}{\mu + \mu'}}
\]

\[
P = \frac{wb (zh - b \tan \theta)}{2} \times \frac{\tan \theta - \mu}{1 - \mu' \mu + (\mu + \mu') \tan \theta}
\]

In these,

- \(w\) = weight per cubic foot of the material.
- \(h\) = depth of grain in the bin.
- \(b\) = breadth of bin.
- \(\mu\) = coefficient of friction of grain on grain.
- \(\mu'\) = coefficient of friction of grain on the wall of the bin.
- \(\theta\) = the angle between the plane of separation of the wedge-shaped mass causing maximum pressure and the horizontal.
- \(P\) = the horizontal pressure against the side of the bin per foot run of its circumference.

**Design of the Wall**

The walls of bunkers and silos are designed to resist bending moments and tensions caused by pressure of the contained material. If the wall spans horizontally, it is designed for the bending moments and direct tension. If the wall spans vertically, horizontal reinforcement is provided to resist the direct tension and vertical reinforcement to resist the bending moments.
For walls spanning horizontally, the bending moments and forces depend upon the number and arrangement of the compartments. For structures with several compartments, the intermediate walls act as ties between the outer walls. The Table 1 is given for the negative bending moments on the outer walls of rectangular bins with various arrangements of intermediate walls or ties.

$$Bending\ Moment = \frac{P_h D^2}{k}$$

An external wall is subject to maximum stresses when the adjacent compartment is filled, since it is then subjected simultaneously to the maximum bending moment and the maximum direct tension. An internal cross wall is subjected to maximum bending moment when the compartment on one side of it is filled, and to maximum direct force (but not bending moment) when the compartments on both sides of the wall are filled.

**Hopper Bottom**

There are two types of bottoms. The first type is continuously supported along two sides only, while the second type is supported all round. At the bottom of the side wall there will be vertical pull in a downward direction due to the weight of the retained material, weight of the sloping bottom and opening. This will give rise to direct tension in the sloping bottom. In addition to designing the hopper bottom for direct tension, it is to be designed as a slab
Table 1

Bending Moments in Walls Spanning Horizontally and Subjected to Horizontal Pressure of Intensity \( p_h \)

<table>
<thead>
<tr>
<th>Form of Container</th>
<th>Formula</th>
<th>Bending Moment Coefficients ( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Values of ( B/D )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>( M_1 )</td>
<td>( M_1 = \frac{P(D+2b)^3}{12(D+2b)} )</td>
<td>16</td>
</tr>
<tr>
<td>( M_2 )</td>
<td>( M_2 = \frac{P(D+2b)^3}{12(D+2b)} )</td>
<td>10.1</td>
</tr>
<tr>
<td>( M_3 )</td>
<td>( M_3 = \frac{P(D+2b)^3}{12(D+2b)} )</td>
<td>19.2</td>
</tr>
<tr>
<td>( M_4 )</td>
<td>( M_4 = \frac{P(3D+5b)^3}{12(3D+5b)} )</td>
<td>11.2</td>
</tr>
<tr>
<td>( M_5 )</td>
<td>( M_5 = \frac{P(3D+5b)^3}{12(3D+5b)} )</td>
<td>18.2</td>
</tr>
<tr>
<td>Form of Container</td>
<td>Formula</td>
<td>Bending Moment Coefficients k</td>
</tr>
<tr>
<td>-------------------</td>
<td>---------</td>
<td>------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Values of B/D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>( M_2 )</td>
<td>( D )</td>
<td>11.4</td>
</tr>
<tr>
<td>( B )</td>
<td>( M_1 )</td>
<td>16</td>
</tr>
<tr>
<td>( M_3 )</td>
<td>( M_2 )</td>
<td>80</td>
</tr>
</tbody>
</table>

avoiding large cracks.

1.3 In computing the vertical wall stresses for straight walls the maximum allowable unit compressive stress on the total net wall area of the structure shall be the lesser of the following:

\[ f_c = 0.225 f'_c \left[ 1 - \left( \frac{h}{40t} \right)^3 \right] \]

\[ f_c = 0.225 f'_c \left[ 1 - \left( \frac{1}{40t} \right)^3 \right] \]

2. Bond shall be computed according to ACI 318-63, Section 1301. The allowable bond stress shall be that allowed for "Bars other than top bars" as called for in Section 1301 (c) (1) and (c) (2). Splices shall in all cases follow ACI 318-63, Section 805, with the following additional requirements:

2.1 Where the geometry of a bin makes it possible to modify the design splices in the field, the drawings shall call for and the reinforcement shall be detailed to provide a lap equal to 12 in. plus that required in ACI 318-63, Section 805, with a minimum lap of 24 in.

2.2 Where splices can be staggered, they shall be spaced no less than sixty bar diameters from the splices in the adjoining tiers. Where more than half of the bars are spliced within a length of sixty bar diameters and where splices are made at points of maximum stress, the laps should be further increased by 20 percent.

Wall Loads

1. The basic lateral load on a wall due to the action of the filling material shall be obtained using the following formula:

\[ p_n = wR/M' \left( 1 - e^{-\mu kh/R} \right) \]
compression or at point of inflection.

2. Where a section of wall may be loaded from either side, reinforcement shall be provided so that each side can be loaded independently.

3. The clear distance between parallel horizontal bars in walls shall be not less than two times the maximum size of coarse aggregate nor less than two inches. The maximum spacing shall not exceed two times the wall thickness or twelve inches. The minimum bar size shall be #3.

4. Vertical reinforcement is rarely used in bin wall construction except as additional reinforcement around openings or as shrinkage and temperature reinforcement.

5. Vertical shrinkage or temperature reinforcement shall be placed in exterior walls. The minimum amount of such reinforcement shall be 0.0015 of the gross cross-sectional area of concrete. Jackrods in slipform construction may be considered as part of this reinforcement as limited by the connection or splice provided. The maximum spacing between vertical shrinkage or temperature reinforcement shall not exceed three times the wall thickness. The placing of vertical shrinkage or temperature reinforcement in interior walls is not required and might interfere with construction procedures.

6. The bearing wall thickness of straight walls shall be not less than the smaller of:

\[
\begin{align*}
t \text{ (in.)} &= \frac{12h}{25} \text{ (ft.)} \\
t \text{ (in.)} &= \frac{12}{25} \text{ (ft.)}
\end{align*}
\]

with a minimum of four inches.
7. Walls which act as hanger beam to support bin bottom and slabs shall be designed as deep beams. The hanger bars shall extend at least one-half the effective depth of the beam plus the amount necessary to develop bond into the bottom of the wall.

PROBLEM DETAILS AND CRITICAL DIMENSIONS REQUIRED

The elevated square bins have the height of 22.5 ft. (clearance from ground surface to bottom girder). Clearance for operating a railroad car of 16 ft. aside and 22 ft. high for transportation under the bins running between the column throughout the line of the bins must be provided (see fig.).

The required storage of the bin is 1,000,000 barrels (4,000,000 cubic feet) of cement with the unit weight of 94 lbs/cu. ft. and the angle of repose of cement is 40°.

The meet the requirement, the bins are computed to have a capacity of 100,000 cubic feet per one bin, the total being 40 square bins lined up in a row. The bottom of the bin is a hopper with four sides sloped (50° with horizontal plane), suitable for gravity discharge through an opening of 1 ft. 6 in. square (rotary valve). The roof is a two-way slope, making 40° with the horizontal plane with the height of 16.8 ft.

The Frost line is 4 ft. below the grade. Maximum
foundation pressure is 4800 psf. for dead load and bin load only. Use 6000 psf for dead load + bin load + seismic (use zone 3 for seismic).

After calculations with the trial and error method, the suitable dimension of the bins are computed to be as follow:

The width of the storage is 44 ft. (including 2 ft. thickness of the bin wall).

The height of the hopper bottom is 23 ft, and it is 15 inches thick.

The height of the roof is 16.8 ft, and 9 inches thick.

The height of the storage from the bottom of the hopper to the top of the roof is 90.47 ft.

The bottom girder is 22.5 ft. above the ground surface.

The total height from the ground to the roof is 116.47 ft.

Footing is 5 ft. below the ground surface and 1 ft. below the frost line.

ROOF UNIT
Minimum thickness = \( \frac{L}{35} \)

Length of span = 26.1 ft.

Minimum thickness = 26.1/35 x 12 = 8.95 say 9 in.

D.L. = 9/12 x 150 = 112.5 psf.

L.L. = 12 psf.

Total vertical load (p) 112.5 + 124.5 psf

Then the component of load normal to the slab is

The tangential component is \( P_t = 124.5 \cos 50 = 80.1 \text{ psf} \)

The two types of action to be examined are the transverse movement and the longitudinal moment.

Transverse bonding moment at corner \( a \)

\[ = .95.3 \times 26.1^2 / 12 = 5,410 \text{ lb-ft/ft} \]

Use \( f' = 4000 \text{ psi} \)

\[ f_c = 0.45 \times 4000 = 1800 \text{ psi} \]

\[ f_s = 20000 \text{ psi} \]

\[ k = 0.419, j = 0.860 \]

\[ d = 9 - 2.5 = 6.5 \text{ in} \]

\[ A_s = 5410 \times 12/20,000 \times 5.5 \times 0.860 \]

\[ = 0.581 \text{ in}^2 \]
Use No. 5 bars spacing 6 in. center to center.

Transverse bending moment at mid span =

$95.3 \times 26.1^{2} \times \frac{1}{24} = 2.705 \text{ lb.-ft./ft.}$

$A_s = 2.705 \times 12 / 20,000 \times 6.5 \times .860 = 0.291 \text{ in.}^2$

Use No. 5 bars spacing 12 in. center to center.

Temperature Reinforcement

\[ 0.0020 \text{ bt} \]

\[ = 0.0020 \times 12 \times 9 = 0.216 \text{ in.}^2/\text{ft.} \]

Use No. 3 bar spacing 6 in. center to center.

---

Beam moment in the longitudinal direction.

The tangential loading per foot long,

\[ w = 80.1 \times 26.1 = 2,089 \text{ lb/ft} \]

Bending moment = \[ 2,089 \times 40^2 / 8 = 417,800 \text{ lb-ft.} \]

The plate must now be treated as a long slender beam whose compression edge is laterally supported.

\[ d = 26.1 \text{ ft.} \]

\[ A_g = 417,800 \times 12 / 20,000 \times 0.860 \times 26.1 \times 12 \]

\[ = 0.931 \text{ in.}^2 \]

Use 2 - #8 bar.
not less than

1) 1/6 of clear span = 1/6 x 26.1 = 4.5 ft.

2) the effective depth of the member = 6.5 in.

Use 4.5 ft.

\[ \text{Total top reinforcement} = 0.2133 \times 26.1 + 4.5 = 10 \text{ ft.} \]

**CALCULATION PRESSURE OF BIN WALL**

The material is cement; the angle of repose is $40^\circ$

Density = 94 lb. per cubic foot

Coefficient of friction

\[
(\mu) \quad \text{filling on filling} \quad 0.316
\]

\[
(\mu') \quad \text{filling on concrete} \quad 0.700
\]

A square silo may be taken to have the same pressure as a circular silo.

\[
R = \frac{A}{U} = \frac{\text{area enclosed by a horizontal section through the bin} (\text{ft}^2)}{\text{perimeter of the same section} (\text{ft})} = \frac{l^2/4}{l/4} = \frac{l}{4} = \frac{40}{4} = 10 \text{ ft.}
\]

\[
k = 1 - \sin\phi / l + \sin\phi = 1 - \sin 40 / l + \sin 40 = 0.357 / 1.643 = 0.217
\]

Find intensity of the horizontal and vertical pressures on the walls by Janssen's formula.
Table 2

Calculation Pressure of Bin

<table>
<thead>
<tr>
<th>Depth h (ft) from top</th>
<th>$\frac{X}{R}$</th>
<th>$1-e^{-X}$</th>
<th>Horizontal Pressure $Ph$ lb/ft$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.076</td>
<td>0.074</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>0.15</td>
<td>0.139</td>
<td>187</td>
</tr>
<tr>
<td>15</td>
<td>0.23</td>
<td>0.205</td>
<td>275</td>
</tr>
<tr>
<td>20</td>
<td>0.30</td>
<td>0.259</td>
<td>348</td>
</tr>
<tr>
<td>25</td>
<td>0.38</td>
<td>0.316</td>
<td>424</td>
</tr>
<tr>
<td>30</td>
<td>0.46</td>
<td>0.369</td>
<td>496</td>
</tr>
<tr>
<td>35</td>
<td>0.53</td>
<td>0.411</td>
<td>552</td>
</tr>
<tr>
<td>40</td>
<td>0.61</td>
<td>0.457</td>
<td>614</td>
</tr>
<tr>
<td>45</td>
<td>0.68</td>
<td>0.493</td>
<td>662</td>
</tr>
<tr>
<td>50</td>
<td>0.76</td>
<td>0.532</td>
<td>714</td>
</tr>
</tbody>
</table>
Table 3

Values of \((1 - e^{-x})\) in Which \(x = \frac{\kappa}{R} \cdot h\)
in Janssen's Formula*

<table>
<thead>
<tr>
<th>(x)</th>
<th>(1 - e^{-x})</th>
<th>(x)</th>
<th>(1 - e^{-x})</th>
<th>(x)</th>
<th>(1 - e^{-x})</th>
<th>(x)</th>
<th>(1 - e^{-x})</th>
</tr>
</thead>
<tbody>
<tr>
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DESIGN OF BIN WALL

At bottom (50 ft. from top wall).

Horizontal pressure \( = 714 \text{ psf.} \)

Moment at mid-span \( = \frac{wL^2}{24} = 714 \times 40^2 \times 12/24 = 571,200 \text{ lb.-in.} \)

Moment at supports \( = \frac{wL^2}{12} = 714 \times 40^2 \times 12/12 = 1,142,400 \text{ lb.-in.} \)

Direct tension \( T = \frac{F_d}{A} = 714 \times 40/2 = 14280 \text{ lb.} \)

Dividing the direct tension at mid span equally between top and bottom steel.

The wall thickness of straight walls shall not be less than the smaller of equation,

\[ t_{(i)} = \frac{12h_o}{25} \quad (i) \]

\[ t_{(i)} = \frac{16L}{25} \quad (ii) \]

with a minimum of 4 in.

\[ t \text{ in inch} \quad h_o \text{ and } L \text{ in ft.} \]

\[ \therefore \text{ Minimum thickness } t = 12 \times \frac{50}{25} = 24 \text{ in.} \]

Allowing 3.0 in. from the center of reinforcement to outer fiber.

\[ \therefore \text{ Effective depth } 24 - 3 = 21.0 \text{ in.} \]

In computing the vertical wall stresses for straight walls, the maximum allowable unit compressive stress on the total net wall area of the structure shall be the lesser of the following:
\[ f_c = 0.225 f_c' \left[ 1 - (h/40t)^3 \right] \]
\[ = 0.225 \times 4,000 \left[ 1 - \frac{50}{40} \times 24 \right]^3 \]
\[ = 900 (1-0.000125) \approx 900 \text{ psi} \]
\[ f_c = 0.225 f_c' \left[ 1 - (L/40t)^3 \right] \]
\[ = 900 \left[ 1 - 0.000072 \right] \approx 900 \text{ psi} \]

or

\[ n = 8, \quad k = 1/1 + \frac{f_s}{nf_c} = 1/3,777 = 0.266 \]
\[ j = 1 - 1/3 k = 1 - 0.266/3 = 0.912 \]

At mid span,

\[ A_s \text{ for } M = 571,200/20,000 \times .912 \times 21.0 = 1.490 \text{ in}^2 \]
\[ A_s \text{ for tension} = 14,280/2 \times 20,000 = 0.357 \text{ in}^2 \]

Total reinforcement required at outer face = 1.847 in\(^2\)

Use #8 bars @ 5 in. actual area 1.90 in.\(^2\)

At support section

\[ M = 1,142,400 \text{ lb.-in.} \]
\[ \text{Tension} = 14,280 \text{ lb.-in.} \]

Transfer the tension 9.5 in. to the line of the tensile steel.

Modified M = 1,142,400 - 14,280 x (12-30)
\[ = 1,014,000 \text{ lb.-in.} \]

\[ A_s \text{ for } M = 1,014,000/20,000 \times .912 \times 21 = 2.640 \text{ in}^2 \]

\[ A_s \text{ for tension} = 14,280/20000 = 0.714 \text{ in}^2 \]

Total reinforcement required at inner face = 3.354 in\(^2\)
Use #11 bars @ 5.5 in. actual area 3.40 in.\(^2\)

Check shear

\[ V = 714 \times 1 \times 20 = 14,280 \text{ lb} \]

\[ v = \frac{V}{bd} \]

\[ = \frac{14,280}{12 \times 21} \]

\[ = 55.3 \text{ psi} < 126 \text{ psi O.K.} \]
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<th>Direct Tension 1bs</th>
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Total horizontal reinforcement not less than 0.0025bt = 0.0025bt = 0.0025 x 12 x 24 = .72 in²/ft.
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<td>50</td>
<td>135,660</td>
<td>1,006,740</td>
</tr>
</tbody>
</table>
Figure 5
Shown Reinforcement of Outer Face of Outer Wall
Figure 6

Shown Reinforcement at Inner Face of Outer Wall and Both Faces of Intermediate Wall
Figure 7
Detail of Corner in Square Bin

Figure 8
Detail at Intersection of Square Bin
DESIGN OF HOPPER BOTTOM

Design of Sloping Bottom

Capacity of a silo:

40 x 40 x 50 = 80,000 cu ft

1/3 x 16.75 x 40 x 40 = 8,940 cu ft

23/3 [(40 x 40) + (1/2 x 1 1/2) + \sqrt{1600 + 9/4}] = 12,760
Total 101,700 cu ft = 12,760 cu ft
Weight of cement = 94 x 101,700 = 9,560,000 lb

or 4,780 ton

The sloping bottom acts as a slab.

Design the pyramid bottom to resist bending in which the loads are:

1. The normal component, \( p_h \), due to the weight of the cement.

2. The normal component of the weight of the slab.

The slab may be considered as spanning horizontally between the intersections of adjacent sloping faces. In which case it will be noted that, working from the bottom upwards, as the pressure heads decrease, the spans increase.

There is an average depth of 70 ft. below the surface of the cement.

The span of this slab is 20 ft.

\[
p_h = \frac{wR}{\mu} \left(1 - e^{-\frac{\mu k h}{R}}\right) \quad \text{Janssen's formula}
\]

\[
= 1.342 \left(1 - e^{-0.0152h}\right)
\]

\[
h = 70 \quad p_h = 1.342 \left(1 - e^{1.063}\right) = 1,342 (0.654) = 878 \text{ lb per sq ft}
\]

\[
p_v = \frac{p_h}{k} = \frac{878}{0.217} = 4,050 \text{ psf}
\]
\[
\tan 50^\circ = \frac{y}{x} = 1.192
\]

\[
y = 1.192 \times x
\]

from general formula of ellipse \(\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1\)
\(x^2/4050^2 + (1.192x)^2/878^2 = 1\)
\(x^2/16400,000 + 1.42x^2/770,000 = 1\)
\(0.00000061x^2 + 0.000001844x^2 = 1\)
\(0.00001905 \ x^2 = 1\)
\(x^2 = 525,000\)
\(x = 725\)
\(y = 1.192x = 865\)

The average normal pressure from the cement is
\[p_n = \sqrt{x^2 + y^2} = \sqrt{725^2 + 865^2}\]
\[= \sqrt{525,000 + 747,000}\]
\[= 1,127 \text{ psf.}\]

The normal pressure due to the weight of the concrete is (assume a 15" slab) 188 x cos 50° = 121 psf

Total normal load 1,248 psf
Shear 1,248 x 20/2 = 12480 lb
Bending Moment (1248 x 20 x 20)/12 = 41,600 lb-ft

Minimum \(d = \sqrt{2M/f_c b j k}\)
\(f_c = 0.45 \ f'_c = 1800 \text{ psi} = j = 0.860, \ k = 0.419\)
\(d = \sqrt{(2 \times 41,600)/(1,800 \times 0.860 \times 419)} = 11.3 \text{ in}\)

Actual \(d = 15 - 1.4 - .7 - 1.5 = 11.4 \text{ in} \) (assume #11 bars)

\(A_s = (41,600 \times 12)/(20,000 \times 0.860 \times 11.4) = 2.55 \text{ in}^2\)

Use #11 bar @ 7 in. actual area 2.67 in. ^2
Flat Bottom

The total depth from apex of cement to bottom of hopper is 89.8 ft.

From Janssen's formula,

Horizontal pressure at flat bottom \( P_h \)

\[ P_h = 1342 \left( 1 - e^{-0.0152 \times 89.8} \right) \]

\[ = 1342 \times 0.746 \]

\[ = 1000 \text{ psf} \]

Direct tension \[ = 1.5/2 \times 8/12 \times 1,000 = 500 \text{ lb} \]

Required steel \[ 500/20,000 = 0.025 \text{ in}^2 \]

Provide 4 - #4 bar and #2 stirrup at 8 in.
At the bottom of the side wall, there is a vertical pull in a downward direction due to the weight of cement in the bin, the weight of the sloping bottom, and the gate.

The direct tension on one side is made up as follows:

\[
\frac{1}{4} \text{ weight of cement in bin } = \frac{1}{4} \times 9,560,000 = 2,390,000 \text{ lb.}
\]

\[
\frac{1}{4} \text{ weight of sloping bottom } = \frac{(40 + 1.5)}{2} \times 30 \times 15/12 \times 150 = 90,000 \text{ lb.}
\]

\[
\frac{1}{4} \text{ weight of flat bottom } = 19.6/12 \times 8/12 \times 3.13 \times 150 = 510 \text{ lb.}
\]

\[
\frac{1}{4} \text{ weight of gate (assume gate } = 100 \text{ lb) } = 25 \text{ lb.}
\]

Total \[2,480,535 \text{ lb.}\]

Tension per foot run \[2,480,535/40 = 62,013.40 \text{ lb.}\]

At the top of the sloping bottom slab, we must provide for a direct tension per foot run along the slab of \[62013.4 \times 1.305 = 81,100 \text{ lb}\]

This requires an area of steel per foot run of \[81,100/20000 = 4.05 \text{ in}^2\]

Use \#11 bar @ 4.5 in center to center.

\[
\#11 \text{ bar bond stress } (4.8 \sqrt{f_{C'}})/D = (4.8 \sqrt{4000})/1.41 = 216 \text{ psi}
\]

Use 216 psi.

\[.\text{ Anchorage length } L'' = A_s f_{s} / u \zeta \circ = (1.56 \times 20,000)/ (216 \times 4.430) = 32.6 \text{ in}\]

The hanger bars should be extended at least one-half the effective depth of the beam plus the amount necessary to develop bond into the bottom of the wall.
Provided anchorage length = \((51.25 \times 12)/2 + 32.6 = 340\) in. use 29 ft.

**Design the sidewalls to act as girders.** Ensure that they are sufficiently thick to resist the shear, where cement is stored. The thickness already found for the wall slab may not be sufficient owing to the high shears which exist. The span of the side walls, acting as girders, will be taken as 40 ft.

The load on outer girder is made up as follows:

\[ \frac{1}{4} \text{ weight of cement in bin } = \frac{1}{4} \times 9560000 = 2,390,000 \text{ lb.} \]
\[ \frac{3}{4} \text{ weight of sloping bottom } = (40 + 1.5)/2 \times 30 \times 15/12 \times 150 = 90,000 \text{ lb.} \]
\[ \frac{1}{4} \text{ weight of flat bottom } = 19.6/12 \times 8/12 \times 3.13 \times 150 = 510 \text{ lb.} \]
\[ \frac{1}{4} \text{ weight of gate } = 25 \text{ lb.} \]

side wall = \(40 \times 50 \times 2 \times 150 = 600,000 \text{ lb.} \)
top rib = \(40 \times 2.5 \times 2.5 \times 150 = 37,500 \)
Bottom rib = \(40 \times 2.5 \times 2.5 \times 150 = 37,500 \text{ lb.} \)
Roof weight 26.1 x 9/12 x 40 x 150 = 117,500 lb.

**Total** 3,273,035 lb.

\[ M = (3,273,035 \times 40)/12 = 10,920,000 \text{ lb-ft.} \]

The average depth of the girder 51.25 ft.

The steel required is \(\frac{(10,920,000 \times 12)}{(20,000 \times 0.860 \times 51.25 \times 12)} = 12.39 \text{ in.}^2 \)
Figure 9

Shown Reinforcement of Outside Wall
INTERMEDIATE WALL

The intermediate walls act as ties between the outer walls, an intermediate cross wall is subjected to maximum bending moment when the compartment on one side of it is filled, and to maximum direct force (but not bending moment) when the compartments on both sides of the wall are filled. The intermediate wall may be loaded from either side. Reinforcement shall be provided so that each side can be loaded independently. Then, the horizontal reinforcement of the intermediate walls is the same as those of the inner face of the outer walls.

Design the intermediate wall as a girder, to ensure that they are sufficiently thick to resist the shear where cement is stored in both compartments.

The span of the intermediate wall is 40 ft.

The load on intermediate wall is made up as follows:

\[
\frac{1}{2} \text{ weight of cement in bin} = \frac{1}{2} \times 9,560,000 = 4,780,000 \text{ lb}
\]

\[
\frac{1}{2} \text{ weight of sloping bottom} = 2\left[\frac{(40+1.5)}{2} \times 30 \times 15/12 \times 150\right] = 180,000 \text{ lb}
\]

\[
\frac{1}{2} \text{ weight of flat bottom} = 2\left[19.6/12 \times 8/12 \times 3.13 \times 150\right] = 1,020 \text{ lb}
\]

\[
\frac{1}{2} \text{ weight of gate} = 50 \text{ lb}
\]

Roof weight = \(\frac{1}{2} \times 16.8 \times 40 \times 2 \times 150 = 108,000 \text{ lb}\)

Side wall = \(40 \times 50 \times 2 \times 150 = 600,000 \text{ lb}\)

Bottom rib = \(40 \times 2.5 \times 2.5 \times 150 = 37,500 \text{ lb}\)

Total = 5,706,570 lb

Bending moment = \(\frac{(5,706,570 \times 40)}{8} = 28,532,850 \text{ lb-ft}\)

The average depth of the girder 57.0 ft.
The steel required is \( \frac{28,532,850}{(20,000 \times 860 \times 57)} \)
\[ = 29.1 \text{ in}^2 \]

Use 20 \#11 bars actual area \( 31.2 \text{ in}^2 \)

The maximum shear on the girder is \( \frac{5,706,570}{2} = 2,853,285 \text{ lb.} \)
The average shear stress \( (\nu) = \frac{2,853,285}{(24 \times 51.25 \times 12)} \)
\[ = 191 \text{ psi} \]

Allowable shear stress \( (\nu_C) = 1.1 \sqrt{f_C} = 70 \text{ psi} \)

\[ \nu' = 191 - 70 = 121 \text{ psi} \]

Use stirrup or vertical reinforcement \#6 bar 2 layer

\[ A_V = 0.44 \times 2 = 0.88 \text{ in.}^2 \]

\[ S = \frac{A_v \nu'}{\nu' b} = \frac{(0.88 \times 20,000)}{(121 \times 24)} \]
\[ = 6.02 \text{ in} \]

Max. spacing:

1. \( \frac{d}{4} = \frac{51.25}{4} = 12.81 \text{ ft.} \)
Figure 10

Shown Reinforcement of Intermediate Wall
Detail C

#4 @ 12 in.

#11 @ 8 in.

#11 @ 7 in.

#11 @ 4.5 in.

#11 @ 7 in.

#11 @ 5.5 in.

#11 @ 4.5 in.

#6 @ 6 in.

#4 @ 12 in.
2. 16 bars diameter = 16 \times 0.750 = 12 \text{ in.}

3. \sqrt{A/(0.0015 \times b)} = 0.88/(0.0015 \times 24) = 24.5 \text{ in.}

4. or 12 \text{ in.}

or Use #6 bar @ 6 \text{ in.}

Actual area per foot span = (0.88 \times 12)/6 = 1.76 \text{ in.}^2

Minimum of vertical temperature reinforcement 0.0015 bt = 0.432 \text{ in.}^2 < 1.76 \text{ in.}^2

Use #6 bars @ 6 \text{ in.} as vertical reinforcement at the intermediate wall.

**ANALYSIS OF SEISMIC FORCE**

Find the centroid of the filled bin:

Take moment about x axis
roof 2 x 26.1 x 9/12 x 150 = 5,872.5
side wall 2 x 2 x 50 x 150 = 30,000
slope bottom 2 x 30 x 15/12 x 150 = 11,250  -11.5  -129,375
fill material under roof 1/2 x 16.8 x 40 x 94 = 31,584  55.6  1,756,070
fill material between wall 50, x 40 x 94 = 188,000  25  4,700,000
fill material of hopper (1.5 + 40) / 2 x 23 x 94 = 44,861  -7.95  -356,649
Total 311,567.5  7,063,000

Centroid of filled bin 7,063,000 / 311,567.5 = 22.67 foot
from x axis.

Consider frame AB as typical

Assume column size 72" x 72" and beam 2' x 2'
load from bin and cement 5,706,570 x 2 = 11,413,140 lb.
load from column 72/12 x 72/12 x 52.17 x 150 x 2 = 563,000 lb.
load from beam 2 x 2 x 32 x 150 = 19,200 lb
Total load W = 11,995,340 lb.
Total lateral force due to seismic force, V = ZKLW
k = 1.00
Moment of inertia of column \( \frac{1}{12} \times 6 \times 6^3 = 108 \text{ ft}^4 \)

Moment of inertia of girder \( \frac{1}{12} \times 2 \times 2^3 = 1.33 \text{ ft}^3 \)

\[
K_{AB} = K_{BA} = \frac{108}{23.67} = 4.56
\]

\[
K_{BC} = K_{CB} = \frac{108}{28.5} = 3.79
\]

\[
K_{BE} = K_{EB} = \frac{1.32}{41.5} = 0.032
\]

There are 2 degrees of freedom with two sideway analyses.
DESIGN OF GIRDER

Assume live load on girder = 500 lb/ft.
dead load 2 x 2 x 150 = 600 lb/ft

Live load moment,

at center span 1/16 x 0.5 x 41.5^2 = 53.8 k-ft
at end 1/12 x 0.5 x 41.5^2 = 71.8 k-ft

Dead load moment,

at center span 1/16 x 0.6 x 41.5^2 = 64.6 k-ft
at end .1/12 x 0.6 x 41.5^2 = 86.2 k-ft

Moment due to seismic force 302.3 k-ft

Moment at mid span 1.5 x 64.6 + 1.8 x 53.8 = 194 k-ft

Negative moment at end 1.40 (71.8 + 86.3 + 302.3) = 460.4 k-ft
or 0.90 x 71.8 + 1.25 x 302.3 = 442.6 k-ft

Use design moment at end = 460.4 k-ft

Positive moment at end 1.40 (302.3 - 71.8 - 86.2) = 144 k-ft
or 1.25 x 302.3 - 0.90 x 71.8 = 313.4 k-ft

Code requires min. 50% of negative moment, therefore use
313.4 k-ft as the design positive moment at end.

At the top of the girder,

\[ M_u = 460.4 \text{ k-ft} \]

\[ M_u = 460.4/0.9 = 516 \text{ k-ft} \]

\[ f'_c = 4,000, f_y = 60,000 \text{ psi}, \bar{R} = 1041, b = 24 \text{ in} \]

\[ d = \sqrt{(516 \times 12,000)/(24 \times 1041)} = 15.8 \]

min. \( t = 15.8 + 1.5 + 0.38 + .7 = 18.38 \text{ in} \)

actual \( d = 24 - 1.5 - 0.38 - .7 = 21.42 \text{ in} \)
\( a/d = 0.378 \quad a = 0.378 \times 21.42 = 8.1 \text{ in.} \)

Try \( a = 3.9 \text{ in.} \)

\[
\bar{M} = A_s f_y j_d
\]

\( A_s = \frac{516 \times 12}{60 \times 19.47} = 5.3 \text{ in}^2 \)

\( a = \frac{5.3 \times 60}{0.85 \times 4 \times 24} = 3.9 \text{ in} \approx \text{ assumed} \)

\( A_s = 5.3 \text{ in}^2 \)

Use 4 - #11 actual area 6.24 in\(^2\)

At the bottom of end girder,

Moment 313.4 k-ft

\[
\bar{M}_u = 313.4 / 0.9 = 348 \text{ k-ft}
\]

Try \( a = 2.54 \text{ in} \)

\( j_d = 21.42 - 1.27 = 20.15 \text{ in} \)

\( A_s = \frac{348 \times 12}{60 \times 20.15} = 3.46 \text{ in}^2 \)

\( a = \frac{3.46 \times 60}{0.85 \times 4 \times 24} = 2.54 \text{ in} \)

\( A_s = 3.46 \text{ in}^2 \)

Use 4 - #8 actual area 4 in\(^2\)

At the mid span,

Moment 194 k-ft

\[
\bar{M}_u = 194 / 0.9 = 216 \text{ k-ft}
\]

Try \( a = 1.54 \text{ in} \)

\( j_d = 21.42 - .77 = 20.65 \text{ in} \)
\[ A_s = \frac{(216 \times 12)}{(60 \times 20.65)} = 2.09 \text{ in}^2 \]
\[ a = \frac{(2.09 \times 60)}{(0.85 \times 4 \times 24)} = 1.538 \text{ in.} \]
\[ \therefore A_s = 2.09 \text{ in}^2 \]

Use 4 - #8 the same as the bottom of end bar.

Anchorage length of #11 bar

Bond stress \[ u_u = \frac{(6.7\sqrt{f_{c'}})}{D} = \frac{(6.7\sqrt{4,000})}{1,410} = 300 \text{ psi} \]
\[ = \frac{(1.56 \times 60,000)}{(4,430 \times 300)} = 70.5 \text{ in. use 72 in \ for \ #8 \ bar} \]

Bond stress \[ u_u = \frac{(6.7\sqrt{4,000})}{1} = 424 \text{ psi} \]
\[ L'' = \frac{(1.79 \times 60,000)}{(3.142 \times 424)} = 35.5 \text{ in \ use \ 36 \ in} \]

Check the permissible percentage of reinforcement:

Top \[ p = \frac{A_s}{bd} = \frac{6.24}{(24 \times 21.42)} = 0.0122 < 0.025 \text{ max.} \]
Bottom \[ p' = \frac{A_s'}{bd} = \frac{4}{(24 \times 21.42)} = 0.0078 < 0.025 \text{ max.} \]

Furthermore,
\[ q_u = \frac{p f_y}{0.70 f_{c'}} = \frac{(0.0122 \times 60)}{(0.70 \times 4)} = 0.262 \]
\[ q_{u'} = \frac{p' f_y}{0.70 f_{c'}} = \frac{(0.0078 \times 60)}{(0.70 \times 4)} = 0.167 \]
Hence, \[ q_u - q_{u'} = 0.262 - 0.167 = 0.095 < 0.25 \text{ max.} \]

The maximum possible shear that can be induced will be determined on the flexural capacity of the two ends of the girder. The ultimate capacity of the section as given by equation:
\[ M_u = A_s f_y d \left(1 - 0.4 q_u\right) \]
\[ \text{or} \]
\[ M_u = (A_s f_y - A_s' f_{y'}) d [1 - 0.4 (q_u - q_{u'})] + A_s' f_{y'} (d-d') \]
from the first equation
\[ M_u = 6.24 \times 60 \times 21.42/12 (1 - 0.4 \times 0.262) \]
\[ = 600 \text{ k-ft} \]
from the second

\[ M_u = [(6.24 - 4) \times 21.42/12 (1 - 0.4 \times 0.095) + 4 \times (21.42 - 2.5)/12] \times 60 = 608 \text{ k-ft} \]

and the ratio of positive to negative capacity is

\[ \frac{600}{608} = 0.987 > 0.75 \text{ min.} \]

With the dead and live load equal to 1.8 k per foot,
The total shear due to vertical loads is \((1.8 \times 33)/2 = 29.7 \text{ k}\)

The maximum shear that can be induced is therefore

\[ V_u = 29.7 + (600 + 608)/33 = 56.4 \text{ k} \]

The shear capacity of the beam without reinforcement is

\[ 1.9 \times 24 \times 21.42 \times \sqrt{4,000} = 61,800 \text{ lb.} \]

Since the shear resistance is adequate, the amount of web reinforcement within a distance equals to four times the effective depth from the end of the beam not less than

\[ A_V \times d/s = 0.15 A_s \]

Use stirrup No. 3 , \( A_V = 2 \times 0.11 = 0.22 \text{ in}^2 \)

\[ S = (0.22 \times 21.42)/(0.15 \times 4) = 7.86 \text{ in.} \]

Provide stirrup No. 3 bar at 7.5 in. on center for a distance equal to 8 feet from each end of the girder and stirrup No. 3 bar at 10 in. throughout the girder.
DESIGN OF COLUMN

Design Column
(Lower Part)

The axial load on lower part of column is made up as follows:

\[ \frac{1}{2} \text{ weight of cement in bin } \frac{1}{2} \times 9,560,000 = 4,780,000 \text{ lb.} \]

\[ \frac{1}{2} \text{ weight of sloping bottom } 2\left[\frac{(40 + 1.5)/2 \times 30 \times 15/12 \times 150}{2}\right] = 180,000 \text{ lb.} \]

\[ \frac{1}{2} \text{ weight of flat bottom } 2\left[\frac{19.6/12 \times 8/12 \times 3.13 \times 150}{2}\right] = 1,020 \text{ lb.} \]

\[ \frac{1}{2} \text{ weight of gate } 50 \text{ lb.} \]

Roof weight \[ 108,000 + 117,500 = 225,500 \text{ lb.} \]

Side wall \[ (40 \times 50 \times 2 \times 150)2 = 1,200,000 \text{ lb.} \]

Bottom rib \[ (40 \times 2.5 \times 2.5 \times 150)1.5 = 56,250 \text{ lb.} \]

Top rib \[ 40 \times 2.5 \times 2.5 \times 150 = 37,500 \text{ lb.} \]

Weight of the column \[ 6 \times 6 \times 52.17 \times 150 = 281,000 \text{ lb.} \]

\[ \frac{1}{2} \text{ weight of girder } 2 \times 2 \times 41.5/2 \times 150 = 12,450 \text{ lb.} \]

Total dead load = 6,773,770 lb.

or \[ 6,773.77 \text{ k} \]
Axial load due to seismic force 30,200/41.5 = 728 k

Moment due to seismic force 17,040 k = ft

From Uniform Building Code: All lateral load resisting frame member shall be design by the ultimate strength design method.

\[ U = 1.4(6773 + 728) = 10,500 \text{ k} \]

or \[ U = 0.90 \times 6,717 + 1.25 \times 728 = 6,951 \text{ k} \]

Ultimate axial load \( (P_u) \) 10,500 k

Ultimate moment \( (M_u) \) \( 1.4 \times 17040 = 23,850 \text{ k-ft} \)

eccentricity \( (e) = 23,850/10,500 = 2.25 \text{ ft or} \)

27.2 in.

ACI code required min. 0.1 t = 0.1 x 6 = 0.6 ft

Use eccentricity 27.5 in. in the following design

\[ h' = h(0.78 + 0.22r') \]

\[ r' = 1.5 \]

\[ h' = 28.5 \times (0.78 + 0.22 \times 1.5) = 31.6 \text{ ft} \]

\[ R = 1.18 - 0.009 \frac{h'}{r} \leq 1.0 \]

\[ = 1.18 - (0.009 \times 31.6)/(0.3 \times 6) = 1.02 \quad \text{use } R = 1 \]

From Whitney expressed, \( \bar{P} \)

\[ \bar{P} = A_s f_y/(e/(d-d') + 0.5) + b t f_c'/(3te/d^2 + 1.18) \]
\[ \bar{P} = \frac{10,500}{0.7} = 15,000 \text{ k} \]

For design the column the value of \( f_y \) use 60,000 psi

Column size 72" x 72".

\[ 15,000 = A_s' \times 60 / (27.2/64 + 0.5) \]
\[ + (72 \times 72 \times 4) / (3 \times 72 \times 27.2/68^2) + 1.18 \]

\[ 15,000 = 64.8 \times A_s' + 8,460 \]

\[ A_s' = \frac{6,540}{64.8} = 100.8 \text{ in}^2 \]

\[ A_{st} = 2 \times A_s' = 201.6 \text{ in}^2 \]

From Uniform Building Code: The reinforcement ratio "p" in tied column shall be not less than .01 nor greater than 0.06 for seismic zones No. 2 and No. 3.

\[ p = \frac{201.6}{(72 \times 72)} = 0.0389 > 0.01 > 0.06 \text{ O.K.} \]

Use 52 - #18 reinforcing bars actual area 208 in\(^2\)

Check for spiral transverse reinforcement

Special transverse reinforcement is not required if

\[ \frac{P}{A_g f'_c} \leq 0.12 f'_c \]

\[ \frac{P}{A_g f'_c} = \frac{7,501}{(72 \times 72 \times 4)} = 0.362 \geq 0.12 \]

\[ \therefore \text{ required special transverse reinforcement} \]

\[ p'' = 0.45 \left( \frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_y''} \]

\[ = 0.45 \left[ (72/64)^2 - 1 \right] \frac{4,000}{60,000} \]

\[ = 0.00787 \]

Which is less than the 0.008 specified for cold-drawn wire. Use the required minimum \( p'' = 0.008 \)

With \( h'' = 64 \text{ in.} \quad a = 3 \text{ in.} \)
The size required = No. 8 at the face of connection.

The length of the column from the connection that required special transverse reinforcement, 72" in.

The bar size of hoop reinforcement can be reduced at 24 in. from the bottom of girder to No. 7 and at 48 in. to No. 5 as shown in fig. , and also the same as the bottom of column.

**Design Hoop Reinforcement**

If it is assumed that all the steel is distributed equally on four sides, but on the sides perpendicular to the axis of bending it is ineffective then

\[ P_t^m = \frac{(208 - 52) / (72 \times 72)}{60,000 / (0.85 \times 4,000)} \]

\[ = 0.530 \]

\[ P_u / b t f'_{c'} = \frac{10,500 / (72 \times 72 \times 4)}{0.504} \]

\[ t = 72 \text{ in} \quad d = 69 \text{ in} \]

\[ d / t = 68/72 = 0.945 \]

From graphic solution for equation (A9) and equation (A10) of Appendix to ACI code,

\[ M_u / b t^2 f'_{c'} = 0.30 \]

If it is assumed that all the steel is concentrated at the two faces, then
$$p_t^m = \frac{208}{(72 \times 72) \times 60,000/(0.85 \times 4,000)}$$

$$= 0.708$$

with $P_u/\beta f'_{c'} = 0.504$ gives

$$M_u/\beta t^2 f'_{c'} = 0.37$$

Since the side reinforcement will be effective to some degree, it is reasonable to assume that the actual value will be about midway between the extreme conditions. On this basis,

$$M_u/\beta t^2 f'_{c'} = (0.37 \times 0.30)/2 = 0.335$$

and consequently

$$M_u = 0.335 \times 72^3 \times 4 = 500,000 \text{ k-in. or 41700 k-ft}$$

$$M_b = 600 + 608 = 1,208 \text{ k-ft}$$

The ultimate shear:

$$V_u = (M_c + \frac{1}{2}M_b)/H' = (41,700 + 604)/27.5 = 1540 \text{ k}$$

The shear capacity of the column without reinforcement has been given in the equation,

$$V = 1.9b'd/\sqrt{f'_{c'}} (1 + P/16A_{tr}/\sqrt{f'_{c'}})$$

which equals uncracked transformed area of total column cross-section.

$$A_{tr} = A_c + (n - 1) A_g$$

$$= 5184 + 7 \times 200 = 6,584 \text{ in}^2$$

$$V_c = 1.9 \times 72 \times 68/\sqrt{4,000} \left(1 + \frac{(10,500,000)}{16 \times 6,584 \times \sqrt{4,000}}\right)$$
= 1,530 k
\[ A_{fy}d/s = 1,540 - 1,530 = 10 \, k \]
\[ A_v/S = 10/68 \times 60 \]
\[ = 0.000245 \]
Considering that No. 4 ties will be used throughout the central portion of the column beyond the confined end,
\[ A_v = 4 \times .20 = 0.80 \, in^2 \]
\[ S = 0.80 \, in^2 / 0.000245 = 3225 \, in \]
maximum spacing for tied column.

1. 16 bar diameter = 16 x 2.257 = 36.1 in
2. 48 tie diameter = 48 x 0.50 = 24 in
3. Column width = 72 in
Provide no. 4 ties spacing 24 in, center to center.

**Design Column**

(Upper Part)

Dead load = 6,773,770 - 12,450 - 154,000 = 6,607,320 lb
or 6,607 k

Axial load due to seismic force 30,200/41.5 = 728 k

Moment due to seismic force 13,990 k-ft

\[ U = 1.4 \times (6,607 + 728) = 10,250 \, k \]

or \[ U = .90 \times 6,607 + 1.25 \times 728 = 6,861 \, k \]

ultimate axial load (\( P_u \)) = 10,250 k

ultimate moment (\( M_u \)) = 1.4 x 13,990 = 19,600 k-ft

Eccentricity (\( e \)) = 19,600/10,250 = 1.91 ft or 23 in.

\[ h' = 23.67 (0.78 + 0.22 \times 1.5) = 26.3 \, ft \]
\[ R = (1.18 - 0.009 \times 26.3)/(0.3 \times 6) = 1.05 \text{ use } R = 1 \]

\[ \bar{P} = 10,250/0.7 = 14,650 \text{ k} \]

\[ \bar{P} = A_s, \frac{f_y}{(e/(d-d')) + 0.5} + b f_c'/(3f_e/d^2 + 1.18) \]

\[ 14,650 = (A_s' \times 60)/(23/64 + 0.5) + (72 \times 72 \times 4)/[(3 \times 72 \times 23)/68^2 + 1.18] \]

\[ 14,650 = 69.8 A_s' + 9,220 \]

\[ A_s = 5,430/69.8 = 77.8 \text{ in}^2 \]

\[ A_s = 154.6 \text{ in}^2 \]

\[ p = 154.6/(72 \times 72) = 0.0298 \geq 0.018 < 0.06 \text{ o.k.} \]

Use 40 - #18 reinforcing bars actual area 160 in\(^2\)

Check for special transverse reinforcement.

\[ P/A_{gf_c'} = 7,335/(72 \times 72 \times 4) = 0.354 \geq 0.12 \]

Required special transverse reinforcement

\[ p'' = 0.45 \left( A_g/A_c - 1 \right) f_c'/f_y \]

\[ = 0.45 \left[ (72/64)^2 - 1 \right] 4,000/6,0000 = 0.00787 < 0.008 \text{ min.} \]

Use the required minimum \( p'' = 0.008 \)

with \( h'' = 64 \text{ in.} \) \quad a = 3 \text{ in}

No. 8 is required at the face of connection.

The length of the column from the connection that required special transverse reinforcement, 72 in.
Figure 12

Shown Reinforcement of Column (Upper Part)
FOUNDATION UNIT

It is suitable to use mat foundation because spread footings will also cover almost all of the entire area. Mat is selected because it is difficult to control the differential settlements in the large areas as in this case, and mat is also used as a flexible foundation, advantageously, for bins and silos.

The design will be based on conventional analysis (designed as rigid structure).

Numbers of column = 82
Columns are divided into two rows (long direction) each row = 41 columns

Four columns at both ends carrying 3386 k each (including seismic force = 3750 k each).

The rest carrying 6773.71 k (including seismic force = 7501.77 k).

The distance from center to center of columns in the short direction = 39'. While the distance from center to center of columns in the long direction = 40.5' (at both ends) and 42' (column between). See figure.

Max. soil pressure for dead load + bin load, 4800 psf. Use 6000 psf for dead load + bin load + seismic.

Total load = 78 x 6773.71 + 4 x 3386
= 541,544 k

Area of Mat required = 541,544/4.8 = 112,700 ft²
Try 1705' x 67' with area of 114,200 ft²
Use #14 bar @ 5 in. actual area = 5.40 in²/ft.

At mid span (top bar)

\[ M = \frac{1}{16} \times 5.25 \times 39^2 \]

\[ = 499 \text{ k - ft/ft} \]

\[ A_s = \left(\frac{499 \times 12}{20 \times 0.860 \times 90.461}\right) = 3.86 \text{ in}^2/\text{ft} \]

Use #14 bar @ 7 in actual area = 3.86 in²/ft

Check cantilever beam in long direction at end of mat foundation (bottom bar)

\[ M = 5.25 \times 14 \times 14/2 = 515 \text{ k - ft/ft} \]

\[ A_s \text{ required} = \left(\frac{515 \times 12}{20 \times 0.860 \times 92.29}\right) = 3.90 \text{ in}^2/\text{ft} \]

Then #11 bar @ 4.5 in. actual area 4.16 in²/ft is adequate.

Check cantilever beam in short direction (bottom bar)

\[ M = 5.25 \times 14 \times 14/2 = 515 \text{ k - ft/ft} \]

\[ A_s \text{ required} = \left(\frac{515 \times 12}{20 \times 0.860 \times 90.46}\right) = 3.76 \text{ in}^2/\text{ft} \]

Then #14 bar @ 5 in. actual area 5.40 in² in adequate.

Check minimum depth required by bending moment.

Maximum bending moment = 862 k-ft

\[ d = \sqrt{\frac{2M}{f_c \times b \times j \times k}} \]

\[ = \sqrt{\frac{(2 \times 862)}{(1.8 \times 1 \times 0.860 \times 0.419)}} \]

\[ = 51.6 \text{ in.} \lesssim 92.154 \text{ in.} \text{ O.K.} \]
Figure 13

Reinforcement of Foundation at Long Direction
BIBLIOGRAPHY


