Revised 12/2/83

Dome Reinfection

Interior Wall Opening - Rectangular

Shear Capacity (Contd)

Concrete \( V_c = 2bd_n f'c = 2 \times 10 \times 10 \times 26 \times \sqrt{3500} = 30700 \text{ lbs} \)

\( = 31 \text{ Kips} \)

Total Shear Capacity \( V_s + V_c = 94 + 31 = 125 \text{ K} \approx 126 \text{ OK} \)

Interior Wall Opening - Haunch

For Mid-Span: \( d = 22'' \)

For Ends: \( d = 52'' \)

\[ \frac{-M_u}{\phi f_c bd^2} = \frac{480 \times 12}{0.9 \times 3.5 \times 10 \times (52)^2} = 0.0676 \]

\( \omega = 0.71 \quad \rho = 0.071 \times \frac{3.5}{60} = 0.0041 \)

\[ -A_s = 0.0041 \times 10 \times 52 = 2.13 \text{ in}^2 \]

As Provided

4 - #5 Continuous Top = 1.24

4 - #5 x 6 - 0'' Each End = 1.24

Total 2.48 in

\[ +\frac{M_u}{\phi f_c bd^2} = \frac{93 \times 12}{0.9 \times 3.5 \times 10 \times (22)^2} = 0.0732 \]

\( \omega = 0.77 \quad \rho = 0.077 \times \frac{3.5}{60} = 0.0045 \)

\[ +A_s = 0.0045 \times 10 \times 22 = 0.99 \text{ in}^2 \]

As Provided

4 - #5 Bottom = 1.24 in²
Dome Reinf (Contd)

Interior Wall opening - Haunch

Shear Reinf Area at critical section

2-legs #3 @ 12" stirrups:  \( = 2 \times 11 = 0.22 \text{ in}^2 \)

2-legs #4 @ 12" V-bars:  \( = 2 \times 2 \times 7.07 = 2.83 \text{ in}^2 \)

1-leg #4 @ 12" Vertical Grid:  \( = 1 \times 2 = 0.20 \text{ in}^2 \)

Total Shear Reinf: \( A_v = 0.703 \text{ in}^2 \)

Shear Capacity:

Steel:  \( V_s = \frac{A_v f_y d}{s} = \frac{0.703 \times 60 \times 22}{12} = 77 \text{ K} \)

Concrete:  \( V_c = 2\sqrt{f_{ce}} \cdot b d \)

\( = 118 \times 10 \times 52 = 61360 \text{ lbs} \)

\( = 61 \text{ K} \)

Total:  \( V_s + V_c = 77 + 61 = 138 \text{ K} > 126 \text{ K} \)

Shear Reinforcement area provided adequate.
Exterior Walls:

Design Wall for lateral soil pressure of earth back fill

Coefficient of at rest lateral soil Pressure

\[ K_0 = 1 - \sin 45 = 1 - 0.5 = 0.5 \]

Height of overburden near top edge of wall = 7 ft

Height near bottom of wall = 15 ft

Lateral press. at top = \( 0.5 \times 125 \times 7 = 440 \text{ PSF} \)

Lateral press. at bottom = \( 0.5 \times 125 \times 15 = 940 \text{ PSF} \)

Average pressure on wall = \( \frac{440 + 940}{2} = 690 \text{ PSF} \)

Assume fixed end condition at top and hinge at bottom, the wall moments are:

At Top = \( \frac{0.69(8)^2}{8} = 5.52 \text{ k} \)

At Mid-span = \( 0.69(8)^2 \times \frac{9}{128} = 3.11 \text{ k} \)

Design Moment Mu

At Top = 1.7 \times 5.52 = 9.38 \text{ k-ft}

At Mid-span = 1.7 \times 3.11 = 5.29 \text{ k-ft}
Exterior Walls: (coultd)

Vertical Reinforcement:

\[ t = \frac{5}{8} \quad d = 6.0 \]

At Top:

\[ \frac{Mu}{F'_{ce}bd^2} = \frac{9.38 \times 12}{0.9 \times 3.5 \times 12 \times (6)^2} = 0.0827 \]

\[ w = 0.087 \quad \rho = 0.087 \times \frac{3.5}{60} = 0.0051 \]

Reqd. \[ A_s = 0.0051 \times 12 \times 6 = 0.37 \text{ in}^2/\text{ft} \]

Provided, \[ A_s = \#4 \times 12 \text{ Grid} = 0.20 \text{ in}^2 \]

\[ \text{\#4 @ 12\'' bent bars} = 0.20 \text{ in}^2 \]

Total \[ 0.40 \text{ in}^2/\text{ft} \]

Mid-Span:

\[ \frac{Mu}{F'_{ce}bd^2} = \frac{5.29 \times 12}{0.9 \times 3.5 \times 12 \times (6)^2} = 0.0466 \]

\[ w = 0.048 \quad \rho = 0.048 \times \frac{3.5}{60} = 0.0028 \]

Reqd. \[ A_s = 0.0028 \times 12 \times 6 = 0.20 \text{ in}^2 \quad \#4 \times 12'' = 0.20 \text{ in}^2/\text{ft} \]

For computing shear stress in wall at base use maximum lateral pressure at base, i.e., 940 PSF

Shear per ft. of wall = \[ 0.94 \times \frac{8}{2} = 3.76 \text{ k/ft} \]

Factored design shear \[ V_u = 1.7 \times 3.76 = 6.4 \text{ k/ft} \]

Shear stress \[ V_u = \frac{6400}{12 \times 5} = 107 \text{ psi} \]

\[ 2F'_{ce} = 118 \text{ psi} \]

O.K.
Footing loading:

- 24' Module

Dome Load:

Height of earth fill near dome edges = 5.0 ft

Wt. of earth fill = 5 x 125 = 625 PSF

Dead Wt. of Con. Dome = 150

Snow load = 30

Tractor load used for the design of dome is a transient load and need not be considered for footings design.

Dome load at apex excluding Tractor from Page 1 (657-314) = 343 PSF

Average dome load = \( \frac{805 + 343}{2} = 575 \) PSF

Dome load per lineal ft. of wall = \( \frac{575 \times 24 \times 24}{4 \times 24} = 3450 \) lbs

Front Wall Ftg. Load:

Dome loading = 3450 lbs/ft

Wt. of 8' high wall 8 x \( \frac{\frac{10}{12}}{2} \) x 150 = 1000 lbs/ft

Wt. of overhang slab + soil + Parapet. = 1930 lbs/ft

Wt. of Ftg. 2 x 1\( \frac{1}{2} \) x 150 = 300 lbs/ft

Total = 6680 lbs/ft

Rear Walls (no overhang load) = 3450 + 1000 + 300 = 4750 lbs/ft

Interior Walls = (2 x 3450) + 1000 + 300 = 8200 lbs/ft
Footing Loading: 28' Module

Dome loading near the edges = 1000 PSF  Page 2

Dome loading at apex excluding tractor = 343 PSF

Average Dome load = \( \frac{1000 + 343}{2} = 672 \) PSF

Dome load per linear ft. of wall = \( \frac{672 \times 28 \times 28}{4 \times 28} \)

= 4700 lbs/ft

Using the above load per linear foot of dome periphery on the walls, the total footing loads are computed on page 36.
Footing Loading = 28' Module

Front Wall:

Vertical Component of Dome Membrane Forces = 4700 lbs/ft  Page 35
Wt. of Overhang Slab + Soil + Parapet = 1930 lbs/ft  Page 21
Wt. of 8' high wall 8 x 10/12 x 1500 = 1000 lbs/ft
Wt. of Fig. 3 x 1 = 450 lbs/ft
Total 8080 lbs/ft

Rear Walls:

Vertical Component of dome membrane Forces = 4700 lbs/ft  Page 35
Wt. of 8' high wall = 1000 lbs/ft
Wt. of Fig. 3 x 1 = 450 lbs/ft
Total 6150 lbs/ft

Interior Wall Figs:

Vertical Component of dome Forces from adjacent modules = 2 x 4700
Wt. of 8' high wall = 9200 lbs/ft
Wt. of 3 x 1 Fig = 1000 lbs/ft
Total = 450 lbs/ft
Total 10650 lbs/ft
**Continuous Wall Figs. Real Walls:**

24' Module Footing Load = 4750 lbs/ft

Ftg. Width for 3500 PSF Soil bearing = \( \frac{4750}{3500} = 1.36 \) Use 2'-0 Minimum width

For 3000 PSF Soil bearing = \( \frac{4750}{3000} = 1.58 \) ft Use 2'-0

For 2500 PSF \( \ldots \) = \( \frac{4750}{2500} = 1.90 \) ft Use 2'-0

For 2000 PSF \( \ldots \) = \( \frac{4750}{2000} = 2.38 \) ft Use 2'-6

For 1500 PSF \( \ldots \) = \( \frac{4750}{1500} = 3.17 \) ft Use 3'-0

28' Module Footing Load = 6150 lbs/ft

Ftg. Width for 3500 PSF Soil bearing = \( \frac{6150}{3500} = 1.76 \) ft Use 2'-0

\( \ldots \) for 3000 PSF \( \ldots \) = \( \frac{6150}{3000} = 2.05 \) ft Use 2'-6

\( \ldots \) for 2500 PSF \( \ldots \) = \( \frac{6150}{2500} = 2.46 \) ft Use 3'-0

\( \ldots \) for 2000 PSF \( \ldots \) = \( \frac{6150}{2000} = 3.08 \) ft Use 3'-6

\( \ldots \) for 1500 PSF \( \ldots \) = \( \frac{6150}{1500} = 4.10 \) ft Use 4'-6
Continuous Wall Footings - Interior Walls

24' Module  Ftg. Load = 8200 lbs/ft

Ftg. width for 3500 PSF soil bearing = \( \frac{8200}{3500} = 2.34 \) Use 2'-6"

For 3000 PSF " " = \( \frac{8200}{3000} = 2.73 \) Use 3'-0"

For 2500 PSF " " = \( \frac{8200}{2500} = 3.28 \) Use 3'-6"

For 2000 PSF " " = \( \frac{8200}{2000} = 4.10 \) Use 4'-0"

For 1500 PSF " " = \( \frac{8200}{1500} = 5.47 \) Use 5'-6"

28' Module  Ftg. Load = 10650 lbs/ft

Ftg. width for 3500 PSF soil bearing = \( \frac{10650}{3500} = 3.04 \) Use 3'-0"

" " 3000 PSF " " = \( \frac{10650}{3000} = 3.55 \) Use 3'-6"

" " 2500 PSF " " = \( \frac{10650}{2500} = 4.26 \) Use 4'-0"

" " 2000 PSF " " = \( \frac{10650}{2000} = 5.32 \) Use 5'-0"

" " 1500 PSF " " = \( \frac{10650}{1500} = 7.10 \) Use 7'-0"
Continuous Wall Fittings: Rear Walls (Cont'd)

Footing Concrete f'c = 3000 psi Steel fy = 60 ksi

Use Footing thickness t = 12" d = 12 - 3 = 9"

Check shear at critical section for largest

Foot. width 7'-0" with net upward pressure

of 1500 PSF.

Shear at critical section = 1500 x 2.50

= 3750 lbs/ft

Factored design shear V_d = 1.7 x 3750 = 6375 lbs/ft

Concrete shear stress = \frac{6375}{12 \times 9} = 59 \text{ psi} < 2\sqrt{f_{c}}

= 110 \text{ O.K.}

Bending Moment at Critical Section

M = 1500 \times (3.0)^2 \div 2 = 6750 \text{ lbs-ft}

Factored design Moment M_d = 1.7 \times 6.75 = 11.5 \text{ K-ft/ft}

\frac{M_d}{\phi f_{c} b d^2} = \frac{11.5 \times 12}{0.9 \times 3.0 \times 12 \times (9)^2} = 0.0526

w = 0.55 \rho = 0.55 \times \frac{3.0}{60} = 0.0028

Reqd. A_s = 0.0028 \times 12 \times 9 = 0.30 \text{ in}^2/\text{ft}

Use #5 @ 12 Bottom Transverse

Bottom Continuous bars #5 @ 12 Long

All Continuous Wall Footings
Spread Footings 24' Module:

**Fig. Type A:** Tributary length of wall load supported = 12 ft

Load per ft of wall (Page 34) = 6.68 K

Fig. load = 12 x 6.68 = 80 K

<table>
<thead>
<tr>
<th>Soil Bearing (PsF)</th>
<th>Fig. Area Req'd (Sq. ft.)</th>
<th>Length L (ft)</th>
<th>Width b (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>24</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>3000</td>
<td>29</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>2500</td>
<td>35</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>2000</td>
<td>45</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td>1500</td>
<td>61</td>
<td>7</td>
<td>8</td>
</tr>
</tbody>
</table>

**Fig. Type B:**

Tributary Wall length = 12 ft

Load per ft of wall = 8.2 K/ft Page 34

Fig. load = 12 x 8.2 = 98 K

<table>
<thead>
<tr>
<th>Soil Bearing (PsF)</th>
<th>Fig. Area Req'd (ft²)</th>
<th>Length L (ft)</th>
<th>Width b (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>30</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>3000</td>
<td>35</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>2500</td>
<td>43</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>2000</td>
<td>55</td>
<td>7</td>
<td>9</td>
</tr>
<tr>
<td>1500</td>
<td>76</td>
<td>8</td>
<td>10</td>
</tr>
</tbody>
</table>
### Spread F'tgs. 24' Module (Contd.)

**Fig. Type C:** F'tg. load from tributary walls

- **Front Wall:** $24 \times 6.68 = 160\text{ kN}$
- **Interior Wall:** $12 \times 8.2 = 98\text{ kN}$
- **Total:** $258\text{ kN}$

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Req'd Area ($ft^2$)</th>
<th>L (ft)</th>
<th>B (ft)</th>
<th>C (ft)</th>
<th>Area Provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500 PSF</td>
<td>78</td>
<td>12</td>
<td>5</td>
<td>4</td>
<td>80</td>
</tr>
<tr>
<td>3000</td>
<td>92</td>
<td>13</td>
<td>6</td>
<td>4</td>
<td>102</td>
</tr>
<tr>
<td>2500</td>
<td>112</td>
<td>14</td>
<td>7</td>
<td>4</td>
<td>126</td>
</tr>
<tr>
<td>2000</td>
<td>144</td>
<td>15</td>
<td>8</td>
<td>4</td>
<td>152</td>
</tr>
<tr>
<td>1500</td>
<td>184</td>
<td>16</td>
<td>9</td>
<td>4</td>
<td>180</td>
</tr>
</tbody>
</table>

**Fig. Type D:** F'tg. load = $4 \times 12 \times 8.2 = 344\text{ kN}$

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Req'd Area</th>
<th>L (ft) x L (ft)</th>
<th>Thickness Trim</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500 PSF</td>
<td>123 $ft^2$</td>
<td>12 x 12</td>
<td>14</td>
</tr>
<tr>
<td>3000</td>
<td>146</td>
<td>13 x 13</td>
<td>14</td>
</tr>
<tr>
<td>2500</td>
<td>180</td>
<td>15 x 15</td>
<td>16</td>
</tr>
<tr>
<td>2000</td>
<td>232</td>
<td>16 x 16</td>
<td>18</td>
</tr>
<tr>
<td>1500</td>
<td>328</td>
<td>18 x 18</td>
<td>18</td>
</tr>
</tbody>
</table>
Spread Ftg. 24 Module (cont’d)

**Ftg. Type E:**

<table>
<thead>
<tr>
<th>Soil bearing</th>
<th>Ftg. Area Req’d (ft²)</th>
<th>Width b x b</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500 PSF</td>
<td>108</td>
<td>11 x 11</td>
</tr>
<tr>
<td>3000 PSF</td>
<td>128</td>
<td>12 x 12</td>
</tr>
<tr>
<td>2500 &quot;</td>
<td>155</td>
<td>13 x 13</td>
</tr>
<tr>
<td>2000 &quot;</td>
<td>198</td>
<td>15 x 15</td>
</tr>
<tr>
<td>1500 &quot;</td>
<td>275</td>
<td>17 x 17</td>
</tr>
</tbody>
</table>

Ftg. Load = (2 x 12 x 6.68) + (2 x 12 x 8.2) = 160 + 197 = 357 K

**Ftg. Type F:**

<table>
<thead>
<tr>
<th>Soil bearing</th>
<th>Ftg. Area Req’d (ft²)</th>
<th>L (ft)</th>
<th>b (ft)</th>
<th>Area (ft²) Provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500 PSF</td>
<td>48</td>
<td>8</td>
<td>4</td>
<td>64 - 16 = 48</td>
</tr>
<tr>
<td>3000 &quot;</td>
<td>57</td>
<td>8</td>
<td>5</td>
<td>64 - 9 = 55</td>
</tr>
<tr>
<td>2500 &quot;</td>
<td>70</td>
<td>9</td>
<td>6</td>
<td>81 - 9 = 72</td>
</tr>
<tr>
<td>2000 &quot;</td>
<td>89</td>
<td>10</td>
<td>7</td>
<td>100 - 9 = 91</td>
</tr>
<tr>
<td>1500 &quot;</td>
<td>114</td>
<td>11</td>
<td>8</td>
<td>121 - 9 = 112</td>
</tr>
</tbody>
</table>

Ftg. Load = 2 x 12 x 6.68 = 160 K
Spread Ftg. 24' Module (Contd)

Shear & Flexural Reinf:

Types A, B, C & F: One-Way Cantilever Action

Critical Sections for shear and flexural of Type B control the design.

All footings use thickness t = 12" d = 9"

3500 PSF Soil bearing: b = 6 ft

Shear at critical section = 3.5 x 2 = 7.0 k/ft

Bending Moment = \(3.5 \times \left(\frac{2.5}{2}\right)^2 = 10.94\) k/ft

3000 PSF Soil bearing: b = 7 ft

At critical section Shear = 3.0 x 2.5 = 7.5 k/ft

Bending Moment = 3.0 \(\times \left(\frac{3.0}{2}\right)^2 = 13.5\) k/ft

2500 PSF Soil bearing: b = 8 ft

At critical section Shear = 2.5 x 3.0 = 7.5 k/ft

Moment = 2.5 \(\times \left(\frac{3.5}{2}\right)^2 = 15.3\) k/ft

2000 PSF Soil bearing: b = 9 ft

At critical Section Shear = 2.0 x 3.5 = 7.0 k/ft

Moment = 2.0 \(\times \left(\frac{4.0}{2}\right)^2 = 16.0\) k/ft
Spread Ftg's 24 Module : (contd)

Types A, B, C & F. (contd)

1500 PSF Soil bearing \( b = 10 \text{ ft} \)

At critical section \( \text{Shear} = 1.5 \times 4 = 6.0 \text{ k/ft} \)

\[
\text{Moment} = 1.5 \frac{(4.5)^2}{2} = 15.2 \text{ k/ft}
\]

Controlling design shear and moment for all soil bearing values

\( \text{Shear} = 7.5 \text{ k/ft} \)

\( \text{Moment} = 16.0 \text{ k/ft} \)

Factored design Shear \( V_u = 1.7 \times 7.5 = 12.8 \text{ k/ft} \)

Factored design Moment \( M_u = 1.7 \times 16.0 = 27.2 \text{ k/ft} \)

Shear stress \( V_u = \frac{12800}{12 \times 9} = 119 \text{ psi} > 2 \sqrt{f_c} = 110 \text{ psi} \)

Shear reinforcement required.

Use bent bars with 30° bend at critical sections to resist shear

\#5 @ 12" bent bars alternate with straight flexural reinforcing bars

Effective shear area \( A_u = 0.31 \times \sin 30° = 0.155 \text{ in}^2/\text{ft} \)

Shear capacity of reinforcing \( V_s = \frac{A_u f_y d}{5} = \frac{0.155 \times 60 \times 9}{12} = 7.0 \text{ k/ft} \)
Spread F.tgs. 24 Module: (Contd)

Types A, B, C & F (Contd)

Shear capacity of concrete \( V_c = 2 \cdot b d \cdot \sqrt{f_{tc}} \)

\[ = 2 \times 12 \times 9 \times \sqrt{3000} \]

\[ = 11880 \text{ lbs} = 11.88 \text{ k/lft} \]

Total shear capacity \( V_s + V_c = 7.0 + 11.88 = 18.88 \text{ k/lft} \)

\[ > 12.8 \text{ k/lft} \text{ O.K} \]

Flexural Reinf.

\[ \frac{M_w}{\phi f_{rc} b d^2} = \frac{27.2 \times 12}{.9 \times 3.0 \times 12 (9)^2} = 1.244 \]

\[ \omega = 1.35 \]

\[ \rho = 1.35 \times \frac{3.0}{60} = .0068 \]

Reqd \( A_s = .0068 \times 12 \times 9 = .73 \text{ in}^2 / \text{fl} \)

Reinf. area provided \( \# 5 @ 12 \) Bottom \( = 0.31 \text{ in}^2 / \text{fl} \)

\( \# 5 @ 12 \) alt. bent bars \( = 0.31 \) "

Total \( 0.62 \text{ in}^2 / \text{fl} \)

Calculated flexural area is based on one-way slab action. Actual footing slab partially acts as two-way. Reinf area provided adequate.
Spread Figs. 24 Module: (cont'd)

Fig Type D: Shear and Flexural Reinfl. Two-way Cantilever

3,500 PSF Soil bearing 12 x 12 Fig  \( t = 14'' \)  \( d = 11'' \)

At critical section Shear \( \frac{3.5 \times 4.5}{2} = 7.9 \text{ K/ft} \)

Moment \( \frac{3.5 (4.5)^2}{2} = 17.7 \text{ K/ft} \)

Factorized Design Shear \( V_u = 1.7 \times 7.9 = 13.4 \text{ K/ft} \)

Moment \( M_u = 1.7 \times 17.7 = 30.0 \text{ K/ft} \)

Shear stress \( V_u = \frac{13400}{12 \times 11} = 102 \text{ psi} \leq \frac{2f_{c}}{f_{c}} = 110 \text{ psi} \)

\[ \frac{M_u}{qf_{c}b d^2} = \frac{30.0 \times 12}{0.9 \times 3.0 \times 12 (11)^2} = 0.92 \]
\[ \omega = 0.97 \]

\[ P = 0.97 \times \frac{3.0}{60} = 0.049 \]

\[ A_s = 0.049 \times 12 \times 11 = 0.64 \text{ in.}^2 / \text{ft} \quad \# 5 @ 6'' 130 \text{ H} \]

Provided \( A_s = 0.62 \text{ in.}^2 / \text{ft} \)

3,000 PSF Soil bearing 13 x 13 Fig  \( t = 14'' \)  \( d = 11'' \)

At critical section Shear \( \frac{3.0 \times 5.0}{2} = 7.5 \text{ K/ft} \)

Moment \( \frac{3.0 (5.0)^2}{2} = 18.75 \text{ K/ft} \)

Factorized Design Shear \( V_u = 1.7 \times 7.5 = 12.75 \text{ K/ft} \)

Moment \( M_u = 1.7 \times 18.75 = 31.85 \text{ K/ft} \)

Shear stress \( V_u = \frac{12750}{12 \times 11} = 106 \text{ psi} \leq \frac{2f_{c}}{f_{c}} = 110 \text{ psi} \)

\[ \frac{M_u}{qf_{c}b d^2} = \frac{31.85 \times 12}{0.9 \times 3.0 \times 12 (11)^2} = 0.97 \]
\[ \omega = 0.104 \]

\[ P = \frac{0.104 \times 3.0}{60} = 0.052 \]

\[ A_s = 0.052 \times 12 \times 11 = 0.68 \text{ in.}^2 / \text{ft} \quad \# 5 @ 6'' \quad \text{Bottom} \]
Spread Figs, 24' Module: (cont'd)

Fig. Type D: (cont'd)

2500 PSF Soil bearing:

At Critical Section Shear = \( \frac{2.5 \times 6}{2} \) = 7.5 k/ft
Moment = \( \frac{2.5 \times (6)^2}{2} \) = 22.5 k/ft

Factored design Shear \( V_u = 1.7 \times 7.5 \times \frac{12}{12} \times 13 \) = 12.75 k/ft
Factored design Moment \( M_u = 1.7 \times 22.5 \times \frac{12}{12} \times 13 \) = 38.2 k/ft

Shear stress \( V_u = \frac{12750}{12 \times 13} = 8.2 \text{ psi} \leq 2\sqrt{f_c} = 110 \text{ psi} \)

\[ \frac{M_u}{\phi f'c b d^2} = \frac{38.2 \times 12}{0.9 \times 3.0 \times 12 \times (13)^2} = 0.838 \]

\[ W = 0.088 \quad \rho = 0.088 \times \frac{3.0}{60} = 0.0044 \]

\[ A_s = 0.0044 \times 12 \times 13 = 0.68 \text{ in}^2 / \text{ft} \quad \#6 @ 6 \text{" Bott} \]

2000 PSF Soil bearing:

At Critical Section Shear = \( \frac{2.0 \times 6.5}{2} \) = 6.5 k/ft
Moment = \( \frac{2.0 \times (7.5)^2}{2 \times 2} \) = 28.2 k/ft

Factored design Shear \( V_u = 1.7 \times 6.5 \times \frac{12}{12} \times 13 \) = 11.05 k/ft
Factored design Moment \( M_u = 1.7 \times 28.2 \times \frac{12}{12} \times 13 \) = 48.0 k/ft

Shear stress \( V_u = \frac{11050}{12 \times 13} = 61 \text{ psi} \leq 2\sqrt{f_c} = 110 \text{ psi} \)

\[ \frac{M_u}{\phi f'c b d^2} = \frac{48.0 \times 12}{0.9 \times 3.0 \times 12 \times (15)^2} = 0.78 \quad W = 0.82 \quad \rho = 0.082 \times 3.0 \times \frac{60}{60} = 0.0041 \]

\[ A_s = 0.0041 \times 12 \times 15 = 0.73 \text{ in}^2 / \text{ft} \quad \#6 @ 6 \text{" Bott} \]
Spread Ftg. 24 Module (Contd)

Ftg. Type D (Contd)

1500 PSF Soil bearing \(18' \times 18'\) Ftg. Thickness \(t = 18''\), \(d = 15''\)

At critical section Shear \(= \frac{1.5 \times 7.5}{2} = 5.70\) K/ft

Moment \(= \frac{1.5 \times (8.5)^2}{2} = 27.1\) K/ft

Factored design Shear \(V_u = 1.7 \times 5.7 = 9.7\) K/ft

" " Moment \(M_u = 1.7 \times 27.1 = 46.0\) K/ft

Shear stress \(V_u = \frac{9700}{12 \times 15} = 54\) psi \(< 2\sqrt{f_c} = 110\) psi

\[
\frac{M_u}{\psi f_c b d^2} = \frac{46.0 \times 12}{14 \times 3.0 \times 12 (15)^2} = .076
\]

\[
\omega = .080, \quad \rho = .08 \times \frac{3.0}{60} = .0040
\]

\[
A_s = .004 \times 12 \times 15 = 0.72\ \text{in}^2/\text{ft} \# 6 \hat{@} 6\ \text{Bott}
\]
Spread Fts. 24' Module: (Contd)

**Fig. Type E:** Shear and Flexural Reinforcement

One-way cantilever action for bending and shear at critical sections.

3500 PSF Soil Bearing: Fig 11' x 11' Thickness t = 12''

\[ d = 9'' \]

At critical section Shear \[ = 3.5 \times 1.5 = 5.25 \text{ k/ft} \]

\[ \text{Moment} = 3.5 \times (1.5)^2 = 3.94 \text{ k/ft} \]

Factored design Shear \[ V_u = 1.7 \times 5.25 = 8.92 \text{ k/ft} \]

\[ \text{Moment} \ M_u = 1.7 \times 3.94 = 6.7 \text{ k/ft} \]

Shear stress \[ V_u = \frac{8920}{12 \times 9} = 82 \text{ psi} < 2 \sqrt{f_{ec}} = 110 \text{ psi} \]

\[ \frac{M_u}{4 f_{ec} b d^2} = \frac{6.7 \times 12}{9 \times 3.0 \times 12 (9)^2} = 0.031 \]

\[ \omega = 0.032 \quad \rho = \frac{0.032 \times 3.0}{60} = 0.0016 \]

\[ A_s = 0.0016 \times 12 \times 9 = 0.17 \text{ in}^2 \quad \text{Use minimum \#5@12} \]

3000 PSF Soil Bearing: 12' x 12' Fig 12''

\[ d = 9'' \]

At critical section Shear \[ = 3.0 \times 2 = 6.0 \text{ k/ft} \]

\[ \text{Moment} = 3.0 (2.0)^2 = 6.0 \text{ k/ft} \]

Factored design Shear \[ V_u = 1.7 \times 6.0 = 10.2 \text{ k/ft} \]

\[ \text{Moment} \ M_u = 1.7 \times 6.0 = 10.2 \text{ k/ft} \]

Shear stress \[ V_u = \frac{10200}{12 \times 9} = 94 \text{ psi} < 2 \sqrt{f_{ec}} = 110 \text{ psi} \]

0.0 K
Spread Figs 24' Module: (Contd)

Fig. Type E: (Contd)

\[
\frac{M_u}{\phi f'_c b d^2} = \frac{10.2 \times 12}{0.9 \times 3.0 \times 12 (9)^2} = 0.047
\]

\[
\rho = \frac{0.047 \times 3.0}{60} = 0.0025
\]

\[
A_s = 0.0025 \times 12 \times 9 = 0.27 \text{ in}^2 / \text{ft}
\]

\[
\# 5 @ 12 = 0.31 \text{ in}^2 / \text{ft}
\]

2500 PSF Soil bearing: 13' x 13' Fig. t = 12''

d = 9''

At critical section:

Shear = 2.5 x 2.5 = 6.25 k/ft

\[
\text{Moment} = 2.5 \left(\frac{2.5}{2}\right)^2 = 7.82 \text{ k/ft}
\]

Factored design Shear \( V_u = 1.7 \times 6.25 = 10.62 \text{ k/ft} \)

\[
\text{Moment} \quad M_u = 1.7 \times 7.82 = 13.3 \text{ k/ft}
\]

Shear stress \( V_u = \frac{1062.0}{12 \times 9} = 98 \text{ psi} \leq 2 \sqrt{f' c} = 118 \text{ psi} \)

\[
\frac{M_u}{\phi f'_c b d^2} = \frac{13.3 \times 12}{0.9 \times 3.0 \times 12 (9)^2} = 0.061
\]

\[
\rho = \frac{0.061 \times 3.0}{60} = 0.0032
\]

\[
A_s = 0.0032 \times 12 \times 9 = 0.35 \text{ in}^2 / \text{ft}
\]

Provided \( A_s \): \# 5 @ 12 Bottom = 0.31 in²/ft

\# 5 @ 12 alt. bent bars = 0.62 in²/ft
Spread Figs. 24′ Module (Contd)

Fig. Type E (Contd).

2000 PSF Soil bearing: 15′ x 15′ Fig t = 12″ d = 9″

At critical section Shear V_u = 2.0 x 3.0 = 7.0 k/ft

Moment M_u = 2.0 (3.0^2) / 2 = 12.25 k/ft

Factored design shear V_u = 1.7 x 7.0 = 11.90 k/ft

" " Moment M_u = 1.7 x 12.25 = 20.8 k/ft

Shear stress V_u = 11900 / 12 x 9 = 110 psi = 2 f_e = 0.95 ω = 0.101

\[
\rho = \frac{0.101 \times 3.0}{60} = 0.0051
\]

As = 0.0051 x 12 x 9 = 0.55 in^2/ft

#5 @ 12″ bottom = 0.31

#5 @ 12″ bent bars = 0.31

1500 PSF Soil bearing: 17′ x 17′ Fig t = 12″ Total 0.62 in^2/ft

At critical section Shear = 1.5 x 3.5 = 5.25 k/ft

Moment = 1.5 (4.5^2) / 2 = 15.2 k/ft

Factored design shear V_u = 1.7 x 5.25 = 8.93 k/ft

" " Moment M_u = 1.7 x 15.2 = 25.84 k/ft

Shear stress = 8930 / 12 x 9 = 82 psi < 2 f_e = 110 psi

\[
\rho = \frac{0.128 \times 3.0}{60} = 0.0064
\]

Required As = 0.0064 x 12 x 9 = 0.69 in^2/ft

Provided #5 @ 12″ bottom = 0.31

#5 @ 12″ bent bars = \frac{0.31}{0.62} in^2/ft
### Spread Footings 28' Module:

**Fig. Type A:**
- Tributary length of wall load = 14 ft
- Load per ft (Page 36) = 8.08 k/ft
- Footing load = 14 x 8.08 = 114 K

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Fig Area Req'd (ft²)</th>
<th>Length L (ft)</th>
<th>Width b (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500 PSF</td>
<td>35</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>3000 &quot;</td>
<td>39</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td>2500 &quot;</td>
<td>48</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>2000 &quot;</td>
<td>60</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>1500 &quot;</td>
<td>82</td>
<td>9</td>
<td>9</td>
</tr>
</tbody>
</table>

**Fig. Type B:**
- Tributary length of wall load = 14 ft
- Load per ft (Page 36) = 10.65 k/ft
- Footing load = 14 x 10.65 = 149 K

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Fig Area Req'd (ft²)</th>
<th>Length L (ft)</th>
<th>Width b (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500 PSF</td>
<td>44</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>3000 &quot;</td>
<td>52</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td>2500 &quot;</td>
<td>62</td>
<td>9</td>
<td>8</td>
</tr>
<tr>
<td>2000 &quot;</td>
<td>78</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>1500 &quot;</td>
<td>115</td>
<td>11</td>
<td>11</td>
</tr>
</tbody>
</table>
Spread Footings 28' Module: (contd)

**Ftg. Type C:**  
Ftg. load from tributary walls:
- Front Wall: $28 \times 8.08 = 262 \text{ K}$
- Interior Wall: $14 \times 10.65 = 149 \text{ K}$
  
<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Req'd Area ($ft^2$)</th>
<th>L (ft)</th>
<th>b (ft)</th>
<th>C (ft)</th>
<th>Area Provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>188</td>
<td>16</td>
<td>6</td>
<td>5</td>
<td>126</td>
</tr>
<tr>
<td>3000</td>
<td>137</td>
<td>17</td>
<td>6</td>
<td>5</td>
<td>132</td>
</tr>
<tr>
<td>2500</td>
<td>164</td>
<td>18</td>
<td>7</td>
<td>6</td>
<td>168</td>
</tr>
<tr>
<td>2000</td>
<td>206</td>
<td>19</td>
<td>8</td>
<td>6</td>
<td>200</td>
</tr>
<tr>
<td>1500</td>
<td>274</td>
<td>21</td>
<td>10</td>
<td>6</td>
<td>270</td>
</tr>
</tbody>
</table>

**Total 411 K**

**Ftg. Type D:**  
Load = $4 \times 14 \times 10.65 = 596 \text{ K}$

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Req'd Area ($ft^2$)</th>
<th>L (ft) x L (ft)</th>
<th>Thickness T (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>180</td>
<td>$14 \times 14$</td>
<td>14</td>
</tr>
<tr>
<td>3000</td>
<td>212</td>
<td>$15 \times 15$</td>
<td>14</td>
</tr>
<tr>
<td>2500</td>
<td>260</td>
<td>$17 \times 17$</td>
<td>16</td>
</tr>
<tr>
<td>2000</td>
<td>330</td>
<td>$19 \times 19$</td>
<td>18</td>
</tr>
<tr>
<td>1500</td>
<td>458</td>
<td>$22 \times 22$</td>
<td>18</td>
</tr>
</tbody>
</table>
## Spread Footings 28' Module: (contd)

### Fig. Type E

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Req'd Area($ft^2$)</th>
<th>$b(ft) \times b(ft)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>158</td>
<td>13 x 13</td>
</tr>
<tr>
<td>3000</td>
<td>187</td>
<td>14 x 14</td>
</tr>
<tr>
<td>2500</td>
<td>218</td>
<td>15 x 15</td>
</tr>
<tr>
<td>2000</td>
<td>292</td>
<td>17 x 17</td>
</tr>
<tr>
<td>1500</td>
<td>404</td>
<td>20 x 20</td>
</tr>
</tbody>
</table>

Load = $(2 \times 14 \times 8.08) + (2 \times 14 \times 10.65) = 226 + 298 = 524 \, K$

## Fig. Type F

<table>
<thead>
<tr>
<th>Soil Bearing</th>
<th>Req'd Area($ft^2$)</th>
<th>L(ft)</th>
<th>b(ft)</th>
<th>Area Provided ($ft^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>66</td>
<td>10</td>
<td>5</td>
<td>100 - 25 = 75</td>
</tr>
<tr>
<td>3000</td>
<td>78</td>
<td>10</td>
<td>6</td>
<td>100 - 16 = 84</td>
</tr>
<tr>
<td>2500</td>
<td>94</td>
<td>11</td>
<td>7</td>
<td>121 - 16 = 105</td>
</tr>
<tr>
<td>2000</td>
<td>120</td>
<td>13</td>
<td>7</td>
<td>169 - 36 = 133</td>
</tr>
<tr>
<td>1500</td>
<td>162</td>
<td>14</td>
<td>9</td>
<td>196 - 25 = 171</td>
</tr>
</tbody>
</table>
Spread Figs. 28' Module: (Cont'd)

Shear and Flexural Reinfl.: Types A, B, C & F.
One-way Cantilever slab action
Shear and bendingMoment at critical sections of Type B controlling design forces
Use all, Fig. thickness \( t = 12'' \) \( d = 12-3 = 9'' \)
Corresponding Fig. width \( b \) for soil bearing of
3500 psf, 3000 psf, 2500 psf + 2000 psf same as
for 24' Module
For 1500 psf soil bearing \( b = 11 \) ft

At critical section Shear \( = 1.5 \times 4.5 = 6.75 \) k/ft
Moment \( = 1.5 (5)^2 = 18.75 \) k/ft

Use factored design shear \( V_u = 1.7 \times 7.5 = 12.75 \) k/ft
Design Moment \( M_u = 1.7 \times 18.75 = 32.0 \) k/ft
Shear Capacity Courante \( V_c = 2bd \sqrt{f_{ce}} = 11880 \) lbs/ft
\( \#5 @ 12 \) bent bars steel \( V_s = \frac{\sum A_s f_y d}{5} = 7.0 \) k/ft (Page 44)

Total \( 18.8 \) k/ft \( > 12.75 \) k/ft O.K.

\[ M_u = \frac{32.0 \times 12}{4 \times 30 \times 12 (9)^2} = 0.146 \]
\[ w = 0.161 \]
\[ \rho = 0.161 \times \frac{3}{60} = 0.0081 \]

Required \( A_s = 0.0081 \times 12 \times 9 = 0.87 \) in²/ft
Couting partially two-way slab action \( \#5 @ 6 \) Eq Way O.K.
Spread Figs. 28 Module: (Contd)

Fig. Type D: By inspection of calculations for corresponding Type footing of 24' Module, it is noted that footing sizes with 3000 psf and 1500 psf soil bearing are governing the design. Since the footing thickness and reinf. bar areas for corresponding soil bearing values for both size modules i.e. 24' and 28' are the same, it is necessary to perform calculations for 3000 psf and 1500 psf soil bearing.

3000 psf Soil bearing: 15' x 15' Fig t = 14''
Wall leads = 6 ft

Use one-way action for critical sections for shear and bending at full section of footing beyond the edge of wall lead.
Spread Footings 28 Module: (Contd)

Fig. Type D (Contd)

At critical section
Shear = 3.0 x 1.5 = 4.5 k/ft
Moment = 3.0 \( \frac{(1.5)^2}{2} \) = 3.4 k/ft

Shear and moment less than the corresponding values for 24 Module. See Calculations page 46.

Fig. design adequate i.e. \( t = 14'' \)
# S @ 6'' Bolt, Ea. Way

1500 PSF Soil bearing: 22' x 22' Fig. \( t = 18'' \)
\( d = 15'' \)

At critical section
Shear = 1.5 x 3.5 = 5.3 k/ft
Moment = 1.5 \( \frac{(3.5)^2}{2} \) = 18.75 k/ft

Shear and moment less than the values at critical sections of corresponding Fig. of 24 Module. See Calculations page 48.

Fig. design adequate i.e. \( t = 18'' \)
with # 6 @ 6'' Bolt, Ea. Way
Spread Ftg's 28' Module: (Contd)

Ftg Type E:

3500 PSF Soil bearing: 13'x13' Ftg  
\[ t = 12'' \quad d = q' \]

At critical section  
Shear = \( 3.5 \times 0.5 = 1.75 \text{ klf} \)

Moment = \( 3.5 \times (0.5)^2 = 0.44 \text{ klf} \)

3000 PSF Soil bearing: 14'x14' Ftg  

At critical section  
Shear = \( 3.0 \times 1.0 = 3.0 \text{ klf} \)

Moment = \( 3.0 \times (1.0)^2 = 3.0 \text{ klf} \)

2500 PSF Soil bearing: 15'x15' Ftg  

At critical section  
Shear = \( 2.5 \times 1.5 = 3.75 \text{ klf} \)

Moment = \( 2.5 \times (1.5)^2 = 6.25 \text{ klf} \)

2000 PSF Soil bearing: 17'x17' Ftg  

At critical section  
Shear = \( 2.0 \times 2.5 = 5.0 \text{ klf} \)

Moment = \( 2.0 \times (2.5)^2 = 6.25 \text{ klf} \)

1500 PSF Soil bearing: 20'x20' Ftg  

At critical section  
Shear = \( 1.5 \times 4 = 6.0 \text{ klf} \)

Moment = \( 1.5 \times (4)^2 = 12.0 \text{ klf} \)

On inspection of calculations, page 49-51, for Ftg Type E of 24' Module, the design moments and shears for 28' Module are found to be less than for 24' Module. Hence, Ftg design for 28' Module is adequate.
Loading at Form Removal:

At the time of removal of forms, the concrete strength must be adequate to support the load safely of dead weight of dome plus any live load of personnel and equipment.

Dome Wt. At apex 5′ slab = \( \frac{150 \times 5}{12} \) = 63 PSF
Near Edges 12′ slab = 150 PSF
Average = \( \frac{63 + 150}{2} \) = 107 PSF
Live load on Dome (assumed) = 200 PSF
Total = 307 PSF

Uniform design load on dome = 830 PSF

Ratio \( \frac{Load \ at \ Form \ Removal}{Design \ Load} = \frac{307}{800} = 0.38 \)

Compressive and shear capacity of concrete is related to its strength.

Strength required at form removal = 0.38 \( f_c \)

\[ = 0.38 \times 3500 \]
\[ = 1330 \text{ psc} \]

Use Minimum Cylinder Strength at form removal 1500 psc for Dome
Loading at Form Removal: (Could)

4 ft. Overhang Cantilever Slab + Parapet

Wt. of Slab (8" thick) = 100 PSF

Wt. of 36"x10" Parapet = \(3 \times \frac{10}{12} \times 150 = 375\) lbs/ft

Refer Calculations Page 19

Cantilever bending Moment at form removal

\[= (375 \times 4) + \frac{100 \times (4)^2}{2}\]

\[= 1500 + 800 = 2300\] ft lbs

\[= 2.3\] k ft

Slab shear at form removal = 375 + (100 \times 4)

\[= 375 + 400 = 775\] lbs/ft

Design shear, unfactored (Page 19) = 375 + 4 \times 350

\[= 375 + 1400 = 1775\] lbs/ft

Ratio \[\frac{Shear\ at\ form\ removal}{Design\ shear}\ = \frac{775}{1775} = 0.43\]

Ratio \[\frac{Bending\ Moment\ at\ form\ removal}{Design\ bending\ Moment}\ = \frac{2.3}{4.2} = 0.55\]

Strength required at form removal = 0.55 \(f_c\) = 0.55 \times 3500 = 1925 psi

Use 2000 psi Minimum Concrete Strength at Form Removal