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Dome Reinf (contd)

Revised 12/2/83

Interior Wall opening - Rectangular

Shear Capacity : (contd)

Concrete $V_c = 2bd\sqrt{f'_c} = 2 \times 10 \times 26 \times \sqrt{3500} = 30700$ lbs
$= 31$ Kips

Total Shear Capacity $V_s + V_c = 94 + 31 = 125K \approx 126$ O.K

Interior Wall opening · Haunch:

At Mid span $+d = 22''$
At Ends $-d = 52''$

$$\frac{-M_u}{\varphi f'_c b d^2} = \frac{480 \times 12}{.9 \times 3.5 \times 10 (52)^2} = .0676$$

$\omega = .071$ $\rho = .071 \times \frac{3.5}{60} = .0041$

$-A_s = .0041 \times 10 \times 52 = 2.13$ in^2

A_s Provided 4 - #5 Continuous Top = 1.24
4 - #5 × 6'-0" Each End = 1.24

Total 2.48 in^2

$$\frac{+M_u}{\varphi f'_c b d^2} = \frac{93 \times 12}{.9 \times 3.5 \times 10 (22)^2} = .0732$$

$\omega = .077$ $\rho = .077 \times \frac{3.5}{60} = .0045$

$+A_s = .0045 \times 10 \times 22 = 0.99$ in^2

A_s Provided 4 - #5 Bottom = 1.24 in^2

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Dome Reinf (contd) Revised 12/2/83

Interior Wall opening - Haunch

Shear Reinf Area at critical section

2-legs #3 @12" stirrups $= 2 \times .11 = .22$ in^2

2-legs #4 @ 12 V-bars $= 2 \times .2 \times .707 = .283$ in^2

1-leg #4 @ 12 Vertical Grid $= 1 \times .2 = .20$

Total Shear Reinf. $A_v = .703$ in^2

Shear Capacity:

Steel $V_s = \dfrac{A_v f_y d}{S} = \dfrac{.703 \times 60 \times 22}{12} = 77$ K

Concrete $V_c = 2\sqrt{f'_c}\, b d$

$= 118 \times 10 \times 52 = 61360$ lbs

$= 61$ K

Total $V_s + V_c = 77 + 61 = 138$ K > 126 O.K

shear reinf. area provided adequate.

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Exterior Walls:

Revised 12/2/83

Design wall for lateral soil pressure of earth back fill

Co-efficient of at rest lateral soil Pressure

$$K_o = 1 - \sin\varphi = 1 - 0.5 = 0.5$$

Height of overburden near top edge of wall

$$= 7 \text{ ft}$$

Height near bottom of wall $= 15$ ft.

lateral press. at top $= .5\,\gamma H = .5 \times 125 \times 7 = 440$ PSF

Lateral press. at bottom $= .5 \times 125 \times 15 = 940$ PSF

Average pressure on wall $= \dfrac{440 + 940}{2} = 690$ PSF
$= 0.69$ KSF

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

At Top $= \dfrac{.69(8)^2}{8} = 5.52$ K'

At Mid-span $= .69\,(8)^2 \times \dfrac{9}{128} = 3.11$ K'

Design Moment M_u

At Top $= 1.7 \times 5.52 = 9.38$ K' / ft

At Mid-span $= 1.7 \times 3.11 = 5.29$ K' / ft

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Exterior Walls : (contd)

Revised 12/2/83

Vertical Reinf. $t = 9\frac{5}{8}''$ $d = 6.0''$

At Top. $\dfrac{M_u}{\varphi f_c' b d^2} = \dfrac{9.38 \times 12}{.9 \times 3.5 \times 12\,(6)^2} = .0827$

$\omega = .087$, $\rho = .087 \; \dfrac{3.5}{60} = .0051$

Reqd. $A_s = .0051 \times 12 \times 6 = 0.37 \text{ in}^2/\text{ft}$

Provided. A_s #4 @ 12" Grid = 0.20 in^2
#4 @ 12" bent bars = 0.20
Total $0.40 \text{ in}^2/\text{ft}$

Mid-span $\dfrac{M_u}{\varphi f_c' b d^2} = \dfrac{5.29 \times 12}{.9 \times 3.5 \times 12\,(6)^2} = .0466$

$\omega = .048$ $\rho = .048 \times \dfrac{3.5}{60} = .0028$

Reqd $A_s = .0028 \times 12 \times 6 = 0.20 \text{ in}^2$ #4 @ 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 $= .94 \times \dfrac{8}{2} = 3.76$ K/ft

Factored design shear $V_u = 1.7 \times 3.76 = 6.4$ K/ft

Shear stress $v_u = \dfrac{6400}{12 \times 5} = 107$ psi

$< 2\sqrt{f_c'} = 118$ psi

O.K.

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Footing Loading :- 24' Module

Dome Loading:

Height of earth fill near dome edges = 5.0 ft

Wt. of earth fill	$= 5 \times 125$	$= 625$ PSF
Dead wt. of Conc. Dome		$= 150$
Snow load		$= 30$
	Total	805 PSF

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	$8 \times \frac{10}{12} \times 150$	$= 1000$	"
Wt. of overhang slab + Soil + Parapet.		$= 1930$	" Page 21
Wt. of Ftg. 2'x1'	2×150	$= 300$	"
	Total	6680	lbs/ft

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

Interior Walls = (2×3450) + 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 lineal ft. of wall = $\frac{672 \times 28 \times 28}{4 \times 28}$

= 4700 lbs/ft

Using the above load per lineal 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 Force	= 4700 lbs/ft	Page 35
Wt. of overhang slab + Soil + Parapet	= 1930 "	Page 21
Wt. of 8' high Wall $8 \times \frac{10}{12} \times 1500$	= 1000 "	
Wt. of Ftg. 3' × 1' 3×150	= 450 "	
Total	8080 lb/ft	

Rear Walls :

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

Interior Wall Ftgs.

Vertical Component of dome Forces from two adjacent modules = 2 × 4700	= 9200 lbs/ft
Wt. of 8' high wall	= 1000 "
Wt. of 3' × 1' Ftg	= 450 "
Total	10650 lbs/ft

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Continuous Wall Ftgs. Rear Walls:

24' Module Ftg. 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 " " $= \frac{4750}{2500} = 1.90'$ Use 2'-0"

For 2000 PSF " " $= \frac{4750}{2000} = 2.38'$ Use 2'-6"

For 1500 PSF " " $= \frac{4750}{1500} = 3.17'$ Use 3'-0"

28' Module Footing Load = 6150 lbs/ft

Ftg. Width for 3500 PSF soil bearing $= \frac{6150}{3500} = 1.76'$ Use 2'-0"

" " for 3000 PSF " " $= \frac{6150}{3000} = 2.05$ Use 2'-6"

" " for 2500 PSF " " $= \frac{6150}{2500} = 2.46$ Use 3'-0"

" " for 2000 PSF " " $= \frac{6150}{2000} = 3.08$ Use 3'-6"

" " for 1500 PSF " " $= \frac{6150}{1500} = 4.10$ Use 4'-6"

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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

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'-6"

" " 2000 PSF " " $= \frac{10650}{2000} = 5.32$ Use 5'-6"

" " 1500 PSF " " $= \frac{10650}{1500} = 7.10$ Use 7'-0"

Continuous Wall Ftgs. - Rear walls: (contd)

Footing Concrete $f'_c = 3000$ psi steel $f_y = 60$ ksi

Use Ftg. thickness $t = 12''$ $d = 12 - 3 = 9''$

Check shear at critical section for largest Ftg. width 7'-0" with net upward pressure of 1500 PSF.

Shear at critical section $= 1500 \times 2.50$
$= 3750$ lbs/ft

Factored design shear $V_u = 1.7 \times 3750 = 6375$ lbs/ft

Concrete shear stress $= \dfrac{6375}{12 \times 9} = 59$ psi $< 2\sqrt{f'_c} = 110$ O.K

Bending Moment at Critical Section

$$M = 1500 \times \frac{(3.0)^2}{2} = 6750 \text{ lbs.ft}$$
$$= 6.75 \text{ K.ft./ft}$$

Factored design Moment $M_u = 1.7 \times 6.75 = 11.5$ K ft/ft

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{11.5 \times 12}{.9 \times 3.0 \times 12\,(9)^2} = .0526$$

$\omega = .055$ $\rho = .055 \times \dfrac{3.0}{60} = .0028$

Reqd. $A_s = .0028 \times 12 \times 9 = 0.30$ in²/ft

Use #5 @ 12 Bottom Transverse

Bottom Continuous bars #5 @ 12 Long

All Continuous Wall Footings

Spread Footings 24' Module:

Ftg. Type A: Tributar length of wall load supported = 12 ft
Load per ft. of wall (Page 34) = 6.68 K

Ftg. load = 12 × 6.68 = 80 K

Soil Bearing	Ftg. Area Reqd. Sq. ft.	Length L (ft)	Width b (ft)
3500 PSF	24	5	5
3000 "	29	6	5
2500 "	35	6	6
2000 "	45	7	6
1500 "	61	7	8

Ftg. Type B:

Tributary wall length = 12 ft
Load per ft of wall = 8.2 K/ft Page 34
Ftg Load = 12 × 8.2 = 98 K

Soil bearing	Ftg Area Reqd (ft^2)	Length L (ft)	Width b (ft
3500 PSF	30	6	6
3000 PSF	35	6	7
2500 PSF	43	7	8
2000 PSF	55	7	9
1500 PSF	76	8	10

Spread Ftgs. 24' Module (contd)

Ftg. Type C: Ftg. load from tributary walls

Front Wall	24 × 6.68	= 160 K
Interior Wall	12 × 8.2	= 98
Total		258 K

Soil bearing	Reqd. Area (ft^2)	L (ft)	b (ft)	C (ft)	Area Provided
3500 PSF	78	12	5	4	80
3000 "	92	13	6	4	102
2500 "	112	14	7	4	126
2000 "	144	15	8	4	152
1500 "	184	16	9	4	180

Ftg. Type D Ftg. load = 4 × 12 × 8.2 = 394 K

Soil Bearing	Reqd. Area	L (ft) × L (ft)	Thickness T (in)
3500 PSF	123 ft^2	12 × 12	14
3000 "	146	13 × 13	14
2500 "	180	15 × 15	16
2000 "	232	16 × 16	18
1500 "	328	18 × 18	18

Spread Ftgs. 24' Module (contd)

Ftg. Type E : Ftg. Load $= (2 \times 12 \times 6.68) + (2 \times 12 \times 8.2)$
$= 160 + 197 = 357\ K$

Soil bearing	Ftg. Area Reqd (ft^2)	width b × b
3500 PSF	108	11 × 11
3000 PSF	128	12 × 12
2500 "	155	13 × 13
2000 "	198	15 × 15
1500 "	275	17 × 17

Ftg. Type F : Ftg. Load $= 2 \times 12 \times 6.68 = 160\ K$

Soil bearing	Ftg. Area Reqd. (ft^2)	L (ft)	b (ft)	Area (ft^2) Provided
3500 PSF	48	8	4	64-16 = 48
3000 "	57	8	5	64-9 = 55
2500 "	70	9	6	81-9 = 72
2000 "	89	10	7	100-9 = 91
1500 "	114	11	8	121-9 = 112

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Spread Ftgs. 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 $= 3.5 \times 2 = 7.0$ K/ft

Bending Moment " " $= 3.5 \dfrac{(2.5)^2}{2} = 10.94$ K'/ft

3000 PSF Soil bearing : $b = 7$ ft

At critical section Shear $= 3.0 \times 2.5 = 7.5$ K/ft

Bending Moment $= 3.0 \dfrac{(3.0)^2}{2} = 13.5$ K'/ft

2500 PSF Soil bearing : $b = 8$ ft

At critical section Shear $= 2.5 \times 3.0 = 7.5$ K/ft

Moment $= 2.5 \dfrac{(3.5)^2}{2} = 15.3$ K'/ft

2000 PSF Soil bearing $b = 9$ ft

At critical Section Shear $= 2.0 \times 3.5 = 7.0$ K/ft

Moment $= 2.0 \dfrac{(4.0)^2}{2} = 16.0$ K'/ft

Spread Ftgs 24' Module : (contd)

Types A, B, C + F (Contd)

1500 PSF Soil bearing $b = 10$ ft

At critical section shear $= 1.5 \times 4 = 6.0$ k/ft

$$\text{Moment} = \frac{1.5\,(4.5)^2}{2} = 15.2 \text{ k}'/\text{ft}$$

Controlling design shear and Moment for all soil bearing values :

Shear $= 7.5$ k/ft
Moment $= 16.0$ k'/ft

Factored design shear $V_u = 1.7 \times 7.5 = 12.8$ k/ft

Factored design Moment $M_u = 1.7 \times 16.0 = 27.2$ k'/ft

Shear stress $v_u = \frac{12800}{12 \times 9} = 119$ psi $> 2\sqrt{f'_c} = 110$ psi

Shear reinforcement required.

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

#5 @ 12" bent bars alternate with straight flexural reinf. bars

Effective shear area $A_v = .31 \times \sin 30° = .155$ in²/ft

shear capacity of reinf. $V_s = \frac{A_v f_y d}{s} = \frac{0.155 \times 60 \times 9}{12} = 7.0$ k/ft

Spread Ftgs. 24' Module : (contd)

Types A, B, C + F (contd)

Shear capacity of concrete $V_c = 2\,bd\sqrt{f'_c}$

$= 2 \times 12 \times 9 \times \sqrt{3000}$

$= 11880$ lbs $= 11.88$ K/ft

Total shear capacity $= V_s + V_c = 7.0 + 11.88 = 18.88$ K/ft

> 12.8 K/ft O.K

Flexural Reinf.

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{27.2 \times 12}{.9 \times 3.0 \times 12\,(9)^2} = .1244$$

$\omega = .135$

$\rho = .135 \times \frac{3.0}{60} = .0068$

Reqd $A_s = .0068 \times 12 \times 9 = .73$ in²/ft

Reinf. area Provided #5 @ 12 Bottom $= 0.31$ in²/ft

#5 @ 12 alt. bent bars $= 0.31$ "

Total 0.62 in²/ft

Calculated flexural area is based on one-way slab action. Actual footing slab partially acts as two-way. Reinf area provided adequate.

Spread Ftgs. 24' Module :(contd)

Ftg. Type D : Shear and Flexural reinf. Two-way Cantilever

3500 PSF Soil bearing 12' x 12' Ftg Thickness $t = 14''$
$d = 11''$

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

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

Factored Design shear $V_u = 1.7 \times 7.9 = 13.4$ K/ft
" " Moment $M_u = 1.7 \times 17.7 = 30.0$ K'/ft

Shear stress $v_u = \frac{13400}{12 \times 11} = 102$ psi $< 2\sqrt{f'_c} = 110$ psi
O.K.

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{30.0 \times 12}{.9 \times 3.0 \times 12 (11)^2} = .092$$

$\omega = .097$

$$P = .097 \times \frac{3.0}{60} = .0049$$

$A_s = .0049 \times 12 \times 11 = 0.64$ in²/ft #5 @ 6" Both
Provided $A_s = 0.62$ in²/ft

3000 PSF Soil bearing 13' x 13' Ftg $t = 14''$ $d = 11''$

At critical section shear $= \frac{3.0 \times 5.0}{2} = 7.5$ K/ft

Moment $= \frac{3.0}{2} \frac{(5)^2}{2} = 18.75$ K'/ft

Factored design shear $V_u = 1.7 \times 7.5 = 12.75$ K/ft
Moment $M_u = 1.7 \times 18.75 = 31.85$ K'/ft

Shear stress $v_u = \frac{12750}{12 \times 11} = 106$ psi $< 2\sqrt{f'_c} = 110$ psi O.K

$\frac{M_u}{\varphi f'_c b d^2} = \frac{31.85 \times 12}{.9 \times 3.0 \times 12 (11)^2} = .097$ $\omega = .0104$ $P = \frac{.104 \times 3.0}{60} = .0052$

$A_s = .0052 \times 12 \times 11 = 0.68$ in²/ft #5 @ 6" Bottom

Spread Ftgs. 24' Module : (contd)

Ftg. Type D : (contd)

2500 PSF Soil bearing : 15' × 15' Ftg. Thickness $t = 16''$

$d = 13''$

At Critical Section Shear $= \frac{2.5 \times 6}{2} = 7.5$ K/ft

Moment $= \frac{2.5}{2}\frac{(6)^2}{2} = 22.5$ K'/ft

Factored design shear $V_u = 1.7 \times 7.5 = 12.75$ K/ft

Factored design Moment $M_u = 1.7 \times 22.5 = 38.2$ K'/ft

Shear stress $v_u = \frac{12750}{12 \times 13} = 82$ psi $< 2\sqrt{f'_c} = 110$ psi O.K

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{38.2 \times 12}{.9 \times 3.0 \times 12 (13)^2} = .0838$$

$\omega = .088$ $\rho = .088 \times \frac{3.0}{60} = .0044$

$A_s = .0044 \times 12 \times 13 = 0.68$ in²/ft #6 @ 6" Bott

2000 PSF Soil bearing : 16' × 16' Ftg. Thickness $t = 18''$

$d = 15''$

At critical Section Shear $= \frac{2.0 \times 6.5}{2} = 6.5$ K/ft

Moment $= \frac{2.0 \; (7.5)^2}{2 \times 2} = 28.2$ K'/ft

Factored design shear $V_u = 1.7 \times 6.5 = 11.05$ K/ft

" " Moment $M_u = 1.7 \times 28.2 = 48.0$ K'/ft

Shear stress $v_u = \frac{11050}{12 \times 15} = 61$ psi $< 2\sqrt{f'_c} = 110$ psi O.K.

$\frac{M_u}{\varphi f'_c b d^2} = \frac{48.0 \times 12}{.9 \times 3.0 \times 12 (15)^2} = .078$ $\omega = .082$ $\rho = .082 \times \frac{3.0}{60} = .0041$

$A_s = .0041 \times 12 \times 15 = 0.73$ in²/ft #6 @ 6 Bott.

Spread Ftgs. 24' Module (contd)

Ftg. Type D (contd)

1500 PSF Soil bearing 18' x 18' Ftg. Thickness $t = 18''$
$d = 15''$

At critical section shear $= 1.5 \times \frac{7.5}{2} = 5.70$ K/ft

Moment $= \frac{1.5}{2} \times \frac{(8.5)^2}{2} = 27.1$ 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
O.K

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{46.0 \times 12}{.9 \times 3.0 \times 12 (15)^2} = .076$$

$\omega = .080$ $\rho = .08 \times \frac{3.0}{60} = .0040$

$A_s = .004 \times 12 \times 15 = 0.72$ in²/ft #6 @ 6 Bott.

Spread Ftgs. 24' Module : (contd)

Ftg. Type E: Shear and Flexural Reinforcement

Oneway cantilever action for bending and shear at critical sections.

3500 PSF Soil bearing: Ftg 11' x 11' Thickness $t = 12"$
$d = 9"$

At critical section Shear $= 3.5 \times 1.5 = 5.25$ K/ft

Moment $= 3.5 \times \frac{(1.5)^2}{2} = 3.94$ K'/ft

Factored design shear $V_u = 1.7 \times 5.25 = 8.92$ K/ft
" " Moment $M_u = 1.7 \times 3.94 = 6.7$ K'/ft

Shear stress $v_u = \frac{8920}{12 \times 9} = 82$ psi $< 2\sqrt{f'_c} = 110$ psi
O.K

$$\frac{M_u}{\phi f'_c b d^2} = \frac{6.7 \times 12}{.9 \times 3.0 \times 12\,(9)^2} = .031$$

$\omega = .032$ $\rho = .032 \times \frac{3.0}{60} = .0016$

$A_s = .0016 \times 12 \times 9 = 0.17$ in^2 Use minimum #5@12

3000 PSF Soil bearing: 12' x 12' Ftg $t = 12"$
$d = 9"$

At critical section Shear $= 3.0 \times 2 = 6.0$ K/ft
Moment $= 3.0 \frac{(2.0)^2}{2} = 6.0$ K'/ft

Factored design shear $V_u = 1.7 \times 6.0 = 10.2$ K/ft
" " Moment $M_u = 1.7 \times 6.0 = 10.2$ K'/ft

Shear stress $v_u = \frac{10200}{12 \times 9} = 94$ psi $< 2\sqrt{f'_c} = 110$ psi
O.K

Spread Ftgs. 24' Module : (contd)

Ftg. Type E : (contd)

$$\frac{M_U}{\varphi f'_c b d^2} = \frac{10.2 \times 12}{.9 \times 3.0 \times 12 (9)^2} = .047 \qquad \omega = .049$$

$$\rho = .049 \times \frac{3.0}{60} = .0025 \qquad As = .0025 \times 12 \times 9 = 0.27 \text{ in}^2/\text{ft}$$

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

2500 PSF Soil bearing: $13' \times 13'$ Ftg $t = 12''$, $d = 9''$

At critical section shear $= 2.5 \times 2.5 = 6.25$ K/ft

$$\text{Moment} = 2.5 \frac{(2.5)^2}{2} = 7.82 \text{ K}'/\text{ft}$$

Factored design shear $V_u = 1.7 \times 6.25 = 10.62$ K/ft

" " Moment $M_U = 1.7 \times 7.82 = 13.3$ K'/ft

$$\text{Shear stress } v_u = \frac{10620}{12 \times 9} = 98 \text{ psi} < 2\sqrt{f'_c} = 118$$

O.K

$$\frac{M_U}{\varphi f'_c b d^2} = \frac{13.3 \times 12}{.9 \times 3.0 \times 12 (9)^2} = .061 \qquad \omega = .064$$

$$\rho = .064 \times \frac{3.0}{60} = .0032$$

$$As = .0032 \times 12 \times 9 = 0.35 \text{ in}^2/\text{ft}$$

Provided As. #5 @ 12" Bottom $= 0.31$ in²/ft

#5 @ 12 alt. bent bars $= 0.31$

0.62 in²/ft

Spread Ftgs. 24' Module (contd)

Ftg. Type E : (contd)

2000 PSF Soil bearing : 15' x 15' Ftg $t = 12''$
$d = 9''$

At critical section Shear $= 2.0 \times 3.0 = 7.0$ k/ft
Moment $= 2.0 \dfrac{(3.5)^2}{2} = 12.25$ k'/ft

Factored design shear $V_u = 1.7 \times 7.0 = 11.90$ k/ft
" " Moment $M_u = 1.7 \times 12.25 = 20.8$ k'/ft

Shear stress $v_u = \dfrac{11900}{12 \times 9} = 110$ psi $= 2\sqrt{f'_c}$ O.K

$\dfrac{M_u}{\varphi f'_c b d^2} = \dfrac{20.8 \times 12}{.9 \times 3.0 \times 12 (9)^2} = .095$ $\omega = .101$

$\rho = .101 \times \dfrac{3.0}{60} = .0051$

$A_s = .0051 \times 12 \times 9 = 0.55$ in²/ft #5 @ 12" bott = 0.31
#5 @ 12 bent bars = 0.31
Total 0.62 in²/ft

1500 PSF Soil bearing : 17' x 17' Ftg $t = 12''$

At critical section shear $= 1.5 \times 3.5 = 5.25$ k/ft
Moment $= 1.5 \dfrac{(4.5)^2}{2} = 15.2$ k'/ft

Factored design shear $V_u = 1.7 \times 5.25 = 8.93$ k/ft
" " Moment $M_u = 1.7 \times 15.2 = 25.84$ k'/ft
Shear stress $= \dfrac{8930}{12 \times 9} = 82$ psi $< 2\sqrt{f'_c} = 110$ psi
O.K

$\dfrac{M_u}{\varphi f'_c b d^2} = \dfrac{25.84 \times 12}{.9 \times 3.0 \times 12 (9)^2} = 0.1182$ $\omega = .128$

$\rho = .128 \times \dfrac{3.0}{60} = .0064$

Req.d $A_s = .0064 \times 12 \times 9 = 0.69$ in²/ft
Provided #5 @ 12" bottom = 0.31
#5 @ 12' bent bars = 0.31
Total 0.62 in²/ft

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Spread Footings 28' Module :

Ftg. Type A :

Tributary length of wall load = 14 ft
Load per ft (Page 36) = 8.08 K/ft

Ftg. Load = 14 × 8.08 = 114 K

Soil Bearing	Ftg. Area Reqd. (ft^2)	Length L (ft)	Width b (ft)
3500 PSF	35	7	5
3000 "	39	7	6
2500 "	48	8	6
2000 "	60	8	8
1500 "	82	9	9

Ftg. Type B :

Tributary length of wall load = 14 ft
Load per ft (Page 36) = 10.65 K/ft
Footing Load = 14 × 10.65 = 149 K

Soil Bearing	Ftg. Area Reqd. (ft^2)	Length L (ft)	Width b (ft)
3500 PSF	44	8	6
3000 "	52	8	7
2500 "	62	9	8
2000 "	78	10	9
1500 "	115	11	11

Spread Footings 28' Module : (Contd)

Ftg. Type C : Ftg. load from tributary walls

Front Wall $28 \times 8.08 = 262$ K
Interior Wall $14 \times 10.65 = 149$

Total 411 K

Soil Bearing	Reqd. Area (ft^2)	L (ft)	b (ft)	C (ft)	Area Provided
3500	128	16	6	5	126
3000	137	17	6	5	132
2500	164	18	7	6	168
2000	206	19	8	6	200
1500	274	21	10	6	270

Ftg. Type D : Load $= 4 \times 14 \times 10.65 = 596$ K

Soil Bearing	Reqd Area (ft^2)	L(ft) x L(ft)	Thickness T(in)
3500	180	14 x 14	14
3000	212	15 x 15	14
2500	260	17 x 17	16
2000	330	19 x 19	18
1500	458	22 x 22	18

Spread Footings 28' Module : (contd)

Ftg. Type E

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

Soil Bearing	Reqd. Area (ft)^2	b(ft) × b(ft)
3500	158	13 × 13
3000	187	14 × 14
2500	218	15 × 15
2000	292	17 × 17
1500	404	20 × 20

Ftg. Type F :

Load = $2 \times 14 \times 8.08 = 226$ K

Soil Bearing	Reqd. Area (ft^2)	L(ft)	b(ft)	Area Provided (ft^2)
3500	66	10	5	100 - 25 = 75
3000	78	10	6	100 - 16 = 84
2500	94	11	7	121 - 16 = 105
2000	120	13	7	169 - 36 = 133
1500	162	14	9	196 - 25 = 171

Spread Ftgs. 28' Module : (contd)

Shear and Flexural Reinf.: Types A, B, C + F.

One-Way Cantilever slab action

Shear and bending Moment at critical sections of Type B controlling design forces

Use all. Ftg. thickness $t = 12''$ $d = 12 - 3 = 9''$

Corresponding Ftg. 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 $= 1.5 \frac{(5)^2}{2} = 18.75$ 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 Concrete $V_c = 2 b d \sqrt{f'_c} = 11880$ lbs/ft
$= 11.8$ K/ft

#5 @ 12 bent bars steel $V_s = \frac{A_v f_y d}{s} = 7.0$ K/ft (Page 44)

Total 18.8 K/ft > 12.75
O.K

$\frac{M_u}{\phi f'_c b d^2} = \frac{32.0 \times 12}{.9 \times 3.0 \times 12 (9)^2} = 0.146$

$\omega = 0.161$ $\rho = 0.161 \times \frac{3}{60} = .0081$

Reqd $A_s = .0081 \times 12 \times 9 = 0.87$ in^2/ft #5@6 Ea Way O.K.

Counting partially two-way slab action

Spread Ftgs. 28' Module: (Contd)

Ftg. 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' Ftg $t = 14''$
$d = 11''$

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.

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Spread Footings 28' Module : (contd)

Ftg. Type D (contd)

At critical section Shear = $3.0 \times 1.5 = 4.5$ k/ft

Moment $= 3.0 \dfrac{(1.5)^2}{2} = 3.4$ k/ft

Shear and Moment less than the corresponding values for 24' Module See calculations Page 46.

Ftg. design adequate i.e. t = 14"
#5 @ 6" Bott. Ea. way

1500 PSF Soil bearing : 22' x 22' Ftg. t = 18"
d = 15"

At critical section shear = $1.5 \times 3.5 = 5.3$ k/ft

Moment = $1.5 \dfrac{(5)^2}{2} = 18.75$ k'/ft

Shear and moment less than the values at critical sections of corresponding Ftg. of 24' Module See calculations page 48.

Ftg. design adequate i.e. t = 18"

with #6 @ 6 Bott
Ea. Way

Spread Ftgs. 28' Module : (contd)

Ftg. Type E :

3500 PSF Soil bearing $13' \times 13'$ Ftg $t = 12''$
$d = 9''$

At critical section Shear $= 3.5 \times 0.5 = 1.75$ K/ft

Moment $= 3.5 \frac{(0.5)^2}{2} = 0.44$ K'/ft

3000 PSF Soil bearing: $14' \times 14'$ Ftg

At critical section Shear $= 3.0 \times 1.0 = 3.0$ K/ft

Moment $= 3.0 \frac{(1.0)^2}{2} = 3.0$ K'/ft

2500 PSF Soil bearing: $15' \times 15'$ Ftg

At critical section Shear $= 2.5 \times 1.5 = 3.75$ K/ft

Moment $= 2.5 \frac{(1.5)^2}{2} = 2.82$ K'/ft

2000 PSF Soil bearing: $17' \times 17'$ Ftg.

At critical Section Shear $= 2.0 \times 2.5 = 5.0$ K/ft

Moment $= 2.0 \frac{(2.5)^2}{2} = 6.25$ K'/ft

1500 PSF Soil bearing: $20' \times 20'$ Ftg.

At critical section Shear $= 1.5 \times 4 = 6.0$ K/ft

Moment $= 1.5 \frac{(4)^2}{2} = 12.0$ K'/ft

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

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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 $= 150 \times \frac{5}{12} = 63$ 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 Page 2.

Ratio $\frac{\text{Load at Form Removal}}{\text{Design Load}} = \frac{307}{800} = 0.38$

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

Strength required at form removal $= 0.38 f'_c$

$= 0.38 \times 3500$

$= 1330$ psi

Use Minimum Cylinder strength at form removal 1500 psi for Dome

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Loading at Form Removal: (contd)

4 ft. overhang Cantilever Slab & Parapet

wt. of slab (8" thick) = 100 PSF

wt. of 36" x 10" Parapet = $3 \times \frac{10}{12} \times 150$ = 375 lbs/ft

Refer Calculations Page 19.

Cantilever bending Moment at form removal

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

= 1500 + 800 = 2300 ft. lbs

= 2.3 K. ft

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

= 375 + 400 = 775 lbs/ft

Design shear unfactored (Page 19) = 375 + 4 × 350

= 375 + 1400 = 1775 lbs/ft

Ratio $\frac{\text{Shear at form removal}}{\text{Design shear}} = \frac{775}{1775} = 0.43$

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

Strength required at form removal = $0.55 f'_c = 0.55 \times 3500 = 1925$ psi

Use 2000 psi Minimum Concrete Strength at Form Removal