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Client Terra Dome Corporation Page A Of

Project 24 + 28 Modules Date 6-10-83 Made By B.S. SANDHU

Generic Design Checked By

Preliminary Final ✓

INDEX

Revised 12/2/83

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Ref. Terra-Dome Drg. E-2 Dated 6-29-83

Loading: Snow Load = 30 PSF

Level earth back fill 2 ft thickness at apex

Vehicle Load 12000 lbs at dome center

applied over 7'-0" diameter circle.

Wt. of earth back fill = 125 lbs/cu ft

Use shell thickness minimum at apex = 5"

Max. Shell thickness at edges = 12"

Total load on Dome = Snow + Earth fill + Vehicle + Dead wt. of dome.

Equivalent uniform load due to wt. of Vehicle

$$= \frac{12000}{\frac{\pi}{4}(7)^2} = 314 \text{ PSF}$$

with 2 ft thick level earth fill at apex, the thickness of fill at the edges = 6.5 ft

Total equivalent uniform load at apex:

Snow Load	= 30	PSF
Wt of earth fill 2 × 125	= 250	PSF
Vehicle Load	= 314	PSF
Wt. of Shell $150 \times \frac{5}{12}$	= 63	PSF
Total	657	PSF

Loading (contd)

Loading near the edges with 6.5 ft thickness of earth fill $= 6.5 \times 125 =$ 820 PSF

Snow Load = 30 PSF

wt. of shell = 150 PSF

Total 1000 PSF

Since this maximum load occurs only on peripheral area, for design purpose, average of apex load and maximum load will be used.

Thus equivalent uniform design load $= \frac{657 + 1000}{2}$

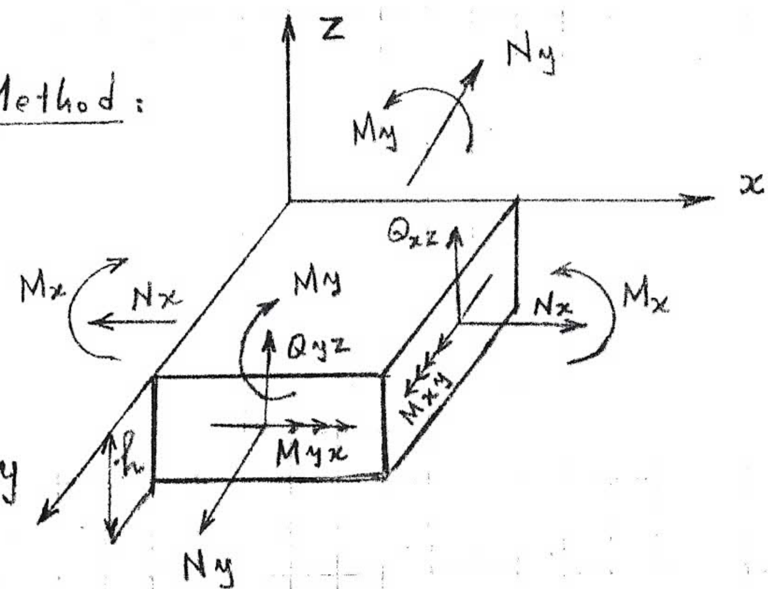
$= 830$ PSF

Shell Analysis:

The design of dome reinforcement is based on the results of stress analysis using Finite Element Computer program.

The stresses and reinforcing steel areas computed by Finite Element Method are verified by hand calculations using simplifying assumptions. The results of hand calculations as shown on subsequent pages check fairly well with computer calculations, the two methods thus affording verification of one by the other.

Basis of Finite Element Method:



N_x, N_y = Normal Forces

M_x, M_y = bending Moments

M_{xy}, M_{yx}: twisting Moments

Q_{xz}, Q_{yz} = Out-of-plane Shear Forces

h = shell thickness

Shell Analysis : (contd)

Finite Element analysis computes forces and stresses as shown

N_x, N_y, M_x, M_y produce normal stresses

M_{xy}, M_{yx} produce in-plane shear stresses

Q_{xz}, Q_{yz} produce out-of-plane shear stresses

Resultant Normal stresses

$$\sigma_x = \frac{N_x}{h} \pm \frac{6 M_x}{h^2}$$

$$\sigma_y = \frac{N_y}{h} \pm \frac{6 M_y}{h^2}$$

Shear stresses

$$\tau_{xy} = \frac{6 M_{xy}}{h^2}$$

$$\tau_{yx} = \frac{6 M_{yx}}{h^2}$$

$$\tau_{xz} = \frac{3}{2} \frac{Q_{xz}}{h}$$

$$\tau_{yz} = \frac{3}{2} \frac{Q_{yz}}{h}$$

Shell Analysis (contd)

1st Subscript represents the plane perpendicular to the subscript axis

2nd Subscript represent the direction of shearing force.

Reinforcement Design Basis:

1. Concrete resists all compression stresses and out-of-plane shear stresses.
2. Steel resists all tensile stresses and in-plane shear stresses.

Thus equivalent normal stresses for the purpose of reinforcement design to account for the in-plane shear stresses are given by

$$\sigma_x = \frac{N_x}{h} \pm \frac{6M_x}{h^2} \pm \frac{6M_{yx}}{h^2}$$

$$\sigma_y = \frac{N_y}{h} \pm \frac{6M_y}{h^2} \pm \frac{6M_{xy}}{h^2}$$

Shell Analysis : (Contd)

Membrane Stresses

At the apex $\frac{N_\theta}{r_\theta} + \frac{N_\varphi}{r_\varphi} = -p$

$p = 657$ psf Page 1.

Also by symmetry at apex $N_\theta = N_\varphi$

and $r_\theta = r_\varphi = 20$ ft

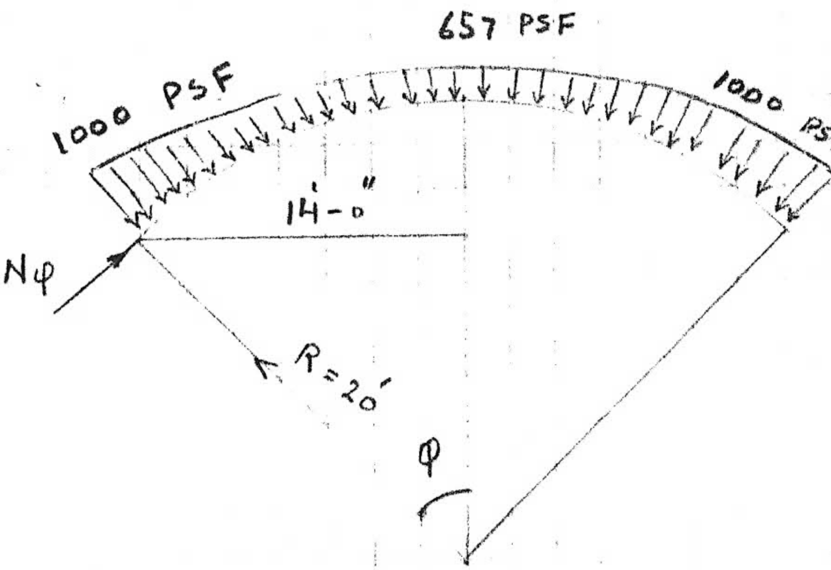
Thus $N_\theta = N_\varphi = -\frac{657 \times 20}{2} = -6570$ lbs/ft

Compressive stress $= \frac{6570}{12 \times 5} = 110$ psi Small o.k

Near Edges:

$\frac{N_\varphi}{r_\varphi} + \frac{N_\theta}{r_\theta} = -1000$

$r_\varphi = r_\theta = 20'$



gives

$N_\varphi + N_\theta = -1000 \times 20$

$= -20{,}000$ lbs/ft

Also $N_\varphi = -\frac{W}{2\pi r \sin\varphi}$

Shell Analysis : (contd)

W = Total Vertical load on 28' Dia Area

$= \pi (14)^2 \times 830 = 51114 0 \text{ lbs}$

Also $\sin\varphi = \dfrac{14}{20} = 0.7 \qquad \varphi = 45^\circ$

$N_\varphi = -\dfrac{511140}{2\pi \times 14 \times 0.7} = -8300$ lbs/ft (Compression)

$N_\theta = -20{,}000 + 8300 = -11700$ lbs/ft (Compressio

Shell thickness $h = 12''$

Meridional Comp. stress $S_\varphi = \dfrac{8300}{12 \times 12} = 58$ psi

Hoop Comp. stress $S_\theta = \dfrac{11700}{12 \times 12} = 82$ psi

Thus all normal stresses in dome are compressive except at the edges there is bending moment due to edge restraint and discontinuity

Shell Analysis (Contd).

Check Top 5" thick shell for buckling:

Critical buckling stress $\sigma_{cr} = \dfrac{Eh}{a\sqrt{3(1-\nu^2)}}$

Timoshenko "Theory of Elastic stability"

Section 11.13 Buckling of Uniformly compressed spherical shells. Eq. 11-32

Where $E = E_{con} = 3600{,}000$ psi

$h = 5"$

$a = 20\,ft = 240"$

$\nu = 0.18$

Thus $\sigma_{cr} = \dfrac{3600{,}000 \times 5}{240\sqrt{3(1-.0324)}} = 44020$ psi large

Buckling no problem

Shell Analysis (contd)

Check Punching Peripheral shear in Dome at various annular sections

Radius of Annular Area (ft.) R	Shell Thickness (inches) t	Horizontal Area (ft^2) A	Ave Unit Load (PSF) p	Total Load (lbs) $W=pA$	Shear Stress (psi) $\frac{W}{24\pi R t}$
2	5	12.57	700	8800	12
4	5	50.27	700	35200	24
6	6	113.12	700	79200	29
8	8	201.1	800	160870	34
10	10	314.2	900	282780	38
12	11	452.5	900	407250	41
14	12	616.0	900	554400	44

Allowable shear $= 1.1\sqrt{f'_c} = 1.1\sqrt{3500} = 65$ psi > 44
O.K
(Working stress)

Though the shell load is supported by membrane action, the above peripheral shear check assumes plate action for the purpose of a conservative and simplified analysis.

Shell Analysis (Contd)

Dome Edge Moment:

Assuming the edges of dome restrained against rotation by the rigidity of the walls, there will be restraining edge moments at the junction of dome and walls

Ref. Timoshenko, "Theory of Plates and shells"
Section 130, Pages 547 - 552

Edge Moment $M_\alpha = -\frac{p a h}{4} \cdot \sqrt{\frac{1-\nu}{3(1+\nu)}}$

where p = uniform pressure or load on shell

a = radius of shell

h = shell thickness

ν = Poissons ratio of concrete

Though load on dome and shell thickness are varying, for the purpose of computing edge moment, the values of p and h at the dome edge may be used in the above formula.

Shell Analysis : (contd)

Edge Moment $M_\alpha = -\frac{pah}{4}\sqrt{\frac{1-\nu}{3(1+\nu)}}$

$p = 1000$ PSF $= 6.95$ psi

$a = 20$ ft $= 240$ in

$h = 12.0$ in

$\nu = 0.2$

$$M_\alpha = -\frac{6.95 \times 240 \times 12}{4}\sqrt{\frac{1-.2}{3(1+.2)}}$$

$= 2360$ lbs.in / in $= 2360$ lbs.ft / ft

Dome Reinforcement:

Concrete $f'_c = 3500$ psi
Steel $f_y = 60{,}000$ psi

Except at the edges and corners, the roof dome stresses are compressive. Use shell reinforcement in accordance with ACI 318-77 Section 19.5

Maximum re-bar spacing 5 times shell thickness or 18 inches.

Max. area of reinf. per ft $= 7.2\, h \dfrac{f'_c}{f_y}$ or $29000 \dfrac{h}{f_y}$

Thus Max $A_s = 7.2 \times 5 \times \dfrac{3500}{60{,}000} = 2.1$ in²/ft

or $A_s = 29000 \times \dfrac{5}{60{,}000} = 2.42$ in²/ft

Minimum $A_s = .0018 \times 5 \times 12 = 0.11$ in²/ft

Use #3 @ 12" Ea. Way A_s provided $= 0.11$ in²/ft

Edge Reinf.

Edge Moment = 2360 lbs.ft/ft
= 2.36 k'/ft

Factored Design Moment $M_u = 1.4 \times 2.36 = 3.3$ k'/ft

At edges $t = 12''$ $d = 9.0$ in

$\dfrac{M_u}{\varphi f'_c b d^2} = \dfrac{3.3 \times 12}{.9 \times 3.5 \times 12 (9)^2} = .013$ $\omega = .014$

$\rho = \omega \dfrac{f'_c}{f_y} = .014 \times \dfrac{3.5}{60} = .0008$

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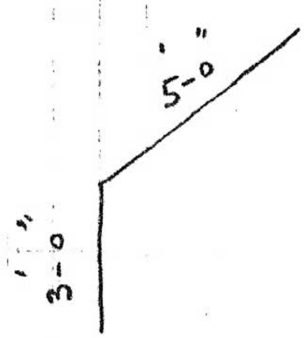
Dome Reinf. (Contd)

Edge Reinf $A_s = .0008 \times 12 \times 9 = .09 \text{ in}^2/\text{ft}$

Use # 4 @ 12" Top $A_s = 0.20 \text{ in}^2/\text{ft}$

Anchor the bent bars in walls as shown all edges of the dome



Corner bars :

Special Corner reinforcement to resist bending moments at corners to be equal to the edge reinforcement

Use # 4 @ 9" Bottom perpendicular to the diagonal

4 @ 9" Top parallel to diagonal

Dome Reinf. (contd).

Check Moment capacity of vertical wall at the Dome-wall Junction for the shell edge moment of 2360 lbs ft/ ft.

Wall thickness $t = 9\frac{5}{8}''$ $d = 6.0''$

Design $M_u = 2.36 \times 1.4 = 3.3$ k/ ft

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{3.3 \times 12}{.9 \times 3.5 \times 12 (6)^2} = .029$$

$\omega = .03$

Steel ratio $\rho = \omega \frac{f'_c}{f_y} = .030 \times \frac{3.5}{60}$

$= .0018$

$A_s = .0018 \times 12 \times 6 = 0.13$ in²/ ft

A_s provided #4 @ 12 $= 0.20$ in²/ft O.K

Dome Reinf. (Contd)

Edge Supports

Design the edge supports for the loading due to vertical and horizontal components of membrane force $N\varphi$ at the edges

At edges $N_\phi = 8300$ lbs/ft Page 7

$\varphi = 45^\circ$

Vertical Component $= N_\varphi \sin\phi = 8300 \times .707$

$= 5870$ lbs/ft

Horizontal Component $= N_\varphi \cos\phi = 8300 \times .707$

$= 5870$ lbs/ft

Wall Support on Back:

Vertical component of $N\varphi$ is resisted by the compressive stress in wall.

Horizontal component is balanced by the lateral pressure of backfill.

Dome Reinf. (contd)

Edge Supports - Back wall

Average height of fill behind wall = 10 ft

Passive resistance of soil (ignoring cohesion)

$$= \frac{1+\sin\varphi}{1-\sin\varphi}\,\gamma H$$

$$= \frac{1+.5}{1-.5} \times 125 \times 10 = 3750 \text{ PSF}$$

Height of wall required to be effective to resist 5870 lbs/ft horizontal load $= \frac{5870}{3750} = 1.56$ ft

Thus horizontal component of $N\varphi$ is effectively resisted by the passive resistance of backfill behind the wall.

Dome Reinf. (Contd) Revised 12/2/83

Edge Supports. Front Side

On front side of dome, maximum thickness of soil backfill is 4 ft and therefore the value of membrane force $N\phi$ will be smaller than on back side.

Total Dome loading near the edge,

Snow Load = 30 PSF
wt. of earth 4×125 = 500 PSF
wt of shell 1×150 = 150

Total 680 PSF

Thus by proportioning from the calculation of $N\phi$ at back side,

$$\text{Front side } N\phi = 8300 \times \frac{680}{830} = 6800 \text{ lbs/ft}$$

$$\text{Vertical Component} = N\phi \sin 45^\circ = 6800 \times .707 = 4800 \text{ lbs/ft}$$

$$\text{Horizontal Component} = N\phi \cos 45^\circ = 6800 \times .707 = 4800 \text{ lbs/ft}$$

In front, 8" × 4'-0" overhang slab acts as shear deck and resists the horizontal component $N\phi \cos\phi$ by the shear deck action and transfers the total

Dome Reinf: (contd) Revised 12/2/83

horizontal force to the side walls.

Span of shear deck = 28 ft

Bending moment in shear deck acting as deep girder $M = \frac{9 \times 4.8 (28)^2}{128} = 264$ K'

(ONE END CONTINUOS, THE OTHER END SIMPLY SUPPORTED)

Horizontal Shear at critical Section $= 4.8 (14-4)$
$= 48.0$ K

Factored design Moment $M_u = 1.4 \times 264 = 370$ K'

Factored design shear $V_u = 1.4 \times 48.0 = 67$ K

Using effective area of shear deck $= 68 \times 8 = 544$ in²

shear stress $v = \frac{67000}{544} = 123$ psi $\approx 2\sqrt{f'_c} = 118$ psi

Since overhang slab and dome are monolithic, assume 2 ft. width of dome acting as part of shear deck. Thus width of shear deck $t = 6$ ft $= 72''$
Effective depth $d = 72 - 4 = 68$ inches

Flexural Reinf:

$$\frac{M_u}{\phi f'_c b d^2} = \frac{370 \times 12}{.9 \times 3.5 \times 8 (68)^2} = .0381$$

$$w = .039$$

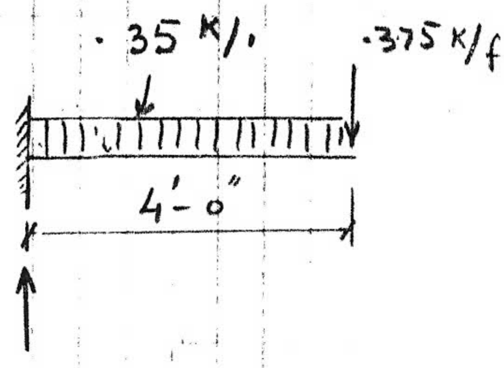
$$P = w \frac{f'_c}{f_y} = .039 \times \frac{3.5}{60} = .0023$$

$A_s = .0023 \times 8 \times 68 = 1.25$ in² Use 6 - #4 continuous.

Dome Reinf. (contd)

Revised 12/2/83

Check Overhang slab as cantilever supporting 2'-0" of earth-fill



Cantilever span = 4'-0"

Wt of 3'×10" Parapet $= 3 \times \frac{10}{12} \times .15$

$= .375$ K/ft

Dead wt of slab $= \frac{8}{12} \times .15 = .10$ K/□

Wt. of earth fill $= 2 \times .125 = .25$ K/□

Total $.35$ K/□

Considering 1'-0" width of slab

Bending Moment $M = (.375 \times 4) + \frac{.35 \times 4 \times 4}{2}$

$= 1.5 + 2.7 = 4.2$ K'/ft

Factored design Moment $M_u = 1.4 \times 4.2 = 6.0$ K'/ft

Factored design shear $V_u = 1.4(.375 + 4 \times .35)$

$= 2.70$ K/ft

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Dome Reinf: (contd) Revised 12/2/83

Front Overhang Slab

At critical section $t = 10''$ $d = 10'' - 2'' = 8''$

Shear stress $v = \frac{2700}{12 \times 8} = 28$ psi $< 2\sqrt{f_c'}$

$= 118$ psi

O.K

$$\frac{M_u}{\varphi f_c' b d^2} = \frac{6.0 \times 12}{.9 \times 3.5 \times 12 (8)^2} = .030$$

$$\omega = .031$$

$$\rho = \omega \frac{f_c'}{f_y} = .031 \times \frac{3.5}{60} = .0018$$

$$A_s = .0018 \times 12 \times 8 = 0.17 \text{ in}^2/\text{ft}$$

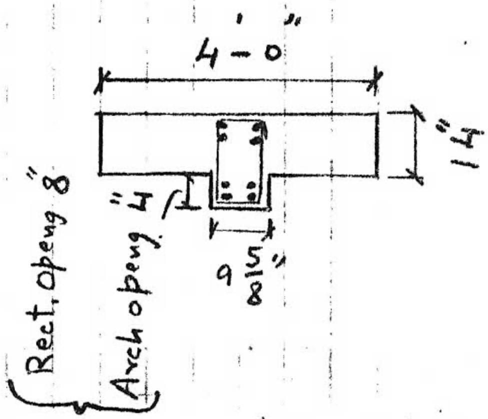
Use # 4 @ 12'' A_s Provided $= 0.20$ in²/ft

Dome Reinf. (contd) Revised 12/2/83

Front Edge Support Beam at opening - Rectangular

Clear opening $= 16'-0''$

Design as a T-ec beam with equivalent section as shown.



Vertical loading:

Vertical component of Dome membrane Force = 4.80 K/ft

Wt. of overhang slab + soil (.35 × 4) + .375 = 1.93 K/ft

Total 6.73 K/ft

For rectangular opening $-M = \frac{6.73\,(16)^2}{12} = 144\,K'$

$$+M = \frac{6.73\,(16)^2}{24} = 72\,K'$$

$$\text{Shear } V = \frac{6.73 \times (16-2)}{2} = 47\,K$$

Factored design Moment $-M_u = 1.4 \times 144 = 202\,K'$

$$+M_u = 1.4 \times 72 = 101\,K'$$

Design Shear $V_u = 1.4 \times 47 = 66\,K$

Dome Reinf: (contd) Revised 12/2/83

Front Edge Support Beam at opening - Rectangular

$b = 10''$ $t = 22''$ $d = 19''$

Shear stress $v_u = \frac{66{,}000}{10 \times 19} = 347$ psi $> 2\sqrt{f'_c}$

$= 118$ psi

Shear reinforcement is required

Use #3 stirrups @ 6"

Minimum shear area $A_v = 50 \frac{bs}{f_y}$

$= \frac{50 \times 10 \times 6}{60{,}000} = 0.05$ in^2

Provide A_v 2 - #3 vertical legs $= 2 \times .11 = .22$ in^2

Total shear capacity

Steel $V_s = \frac{A_v f_y d}{s} = \frac{.22 \times 60 \times 19}{6} = 42.0$ K

Concrete $V_c = 2\sqrt{f'_c}\, b\, d = 118 \times 10 \times 19 = 23600$ lbs

$= 23.6$ K

Total Shear Capacity $V_s + V_c = 42.0 + 23.6 = 65.6$ K ≈ 66 O.K

Dome Reinf. : (contd) Revised 12/2/83

Front Edge Support beam at opening- Rectangular

$t = 22''$ $d = 19''$

Negative Moment Reinf (Top)

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{202 \times 12}{.9 \times 3.5 \times 10 (19)^2} = .213$$

$$\omega = 0.25$$

$$\rho = \omega \frac{f'_c}{f_y} = .25 \times \frac{3.5}{60} = .0146$$

$$-A_s = .0146 \times 10 \times 19 = 2.70 \text{ in}^2$$

$-A_s$ provided 4-#5 Continuous $= 1.24 \text{ in}^2$

4-#5 at Ends $= 1.24$

Total 2.48 in^2

Positive Moment Reinf.

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{101 \times 12}{.9 \times 3.5 \times 10 (19)^2} = .107$$

$\omega = .115$ $\rho = .115 \times \frac{3.5}{60} = .0067$

$$+A_s = .0067 \times 10 \times 19 = 1.27 \text{ in}^2$$

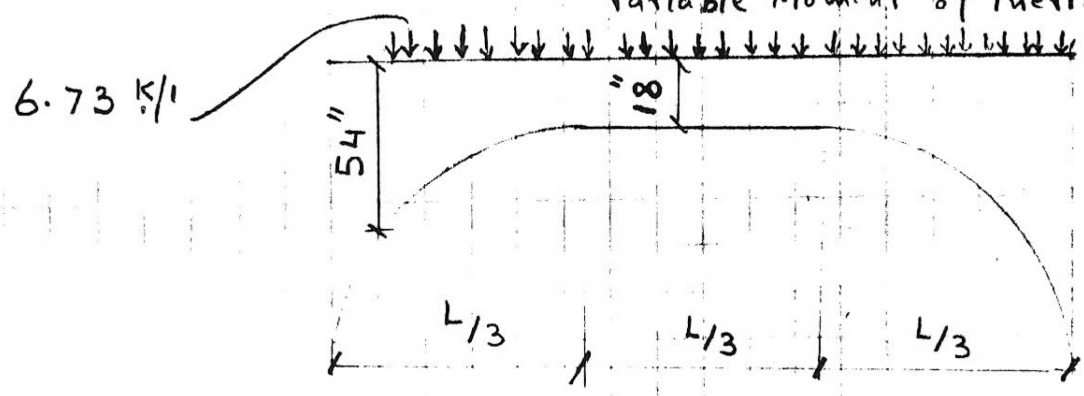
Use 4-#5 Continuous bottom $A_s = 1.24 \text{ in}^2$

Dome Reinf. (contd). Revised 12/2/83

Front Edge Support Beam at opening - Haunch

Beam Moments and shears for Haunch opening

Ref. Portland Cement Association ST 103 "Concrete Members with Variable Moment of Inertia"



Assume Parabolic haunch.

For negative moment and shear, consider the critical section at location where $d = 3\,d_{min} = 3 \times 18 = 54''$

From PCA charts Fig. 3. $a = \frac{1}{3} = .33$

$$\frac{d_{min}}{d_{max}} = \frac{1}{3} = .33$$

Fixed end Moment Coefficient = .105

Negative Moment $-M = .105\,(6.73)(16)^2 = 180\,K'$

Positive Moment $+M = \frac{6.73(16)^2}{8} - 180 = 215 - 180 = 35\,K'$

Dome Reinf. : (contd) Revised 12/2/83

Front Edge support beam at opening - Haunch

Shear at critical section $V = \frac{6.73\,(16-2)}{2} = 47\ K$

Factored design $-M_u = 1.4 \times 180 = 252\ K'$

Factored design. $+M_u = 1.4 \times 35 = 49\ K'$

Factored design Shear $V_u = 1.4 \times 47 = 66\ K$

Negative Reinf.
$t = 54''$
$d = 52''$

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{252 \times 12}{.9 \times 3.5 \times 10\,(52)^2} \doteq .0355$$

$$\omega = .036$$

$$\rho = .036 \times \frac{3.5}{60} = .0021$$

$$-A_s = .0021 \times 10 \times 52 = 1.09\ in^2$$

Provided A_s Enclosed in stirrups 4 - #4 = 0.80 in^2
In overhang slab 2 - #4 = 0.40
Total 1.20 in^2

Positive Reinf.
$t = 18''$
$d = 16''$

$$\frac{M_u}{\varphi f'_c b d^2} = \frac{49 \times 12}{.9 \times 3.5 \times 10\,(16)^2}$$

$$= .073$$

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

$$+A_s = .0045 \times 10 \times 16 = 0.72\ in^2$$

Use 4 - #4 Continuous Bott. $A_s = 0.80\ in^2$

Dome Reinf. (Contd) Revised 12/2/83

Front Edge Support Beam at opening - Haunch

Shear stress at critical section

$$v_u = \frac{66000}{10 \times 52} = 126 \text{ psi} \approx 2\sqrt{f'_c}$$
$$= 118 \text{ psi}$$

Minimum shear reinf. area required as per ACI 318

Use #3 stirrups @ 12"

Minimum shear reinf area $A_v = \dfrac{50\,bs}{f_y}$

$$= \frac{50 \times 10 \times 12}{60{,}000} = 0.10 \text{ in}^2$$

A_v provided 2 - #3 vertical legs = $2 \times .11 = .22 \text{ in}^2$
$> .10$ O.K

Shear Capacity :

Concrete $V_c = 2\,bd\sqrt{f'_c} = 2 \times 10 \times 52\sqrt{3500} = 61000$ lbs
$= 61.0$ K

Steel $V_s = \dfrac{A_v f_y d}{s} = \dfrac{.22 \times 60 \times 16}{12} = 17.6$ K

Total Capacity $V_c + V_s = 61.0 + 17.6 = 78.6$ K > 66.0 O.K

Dome Reinf: (Contd) Revised 12/2/83

Beam at Interior Wall opening 16'-0" Span.

The total vertical load on this beam is the sum of vertical components of membrane force $N\varphi$ of domes from adjacent modules plus the weight of soil on 10" width valley and dead wt of beam itself.

The horizontal components of shell membrane forces balance each other and therefore there is no resultant horizontal load on the interior beam.

Total Vertical load on beam:

Page 15

Vertical component of dome membrane forces	= 2 × 5870 =	11740 #/'
Wt. of soil on 1'-0" width + 6'-6" high = 125×6.5	=	815
Dead wt. of beam 18"×10" 1.5×1×150	=	225
Total		12780 #/'

Using the equivalent T-beam section similar to the beam at front opening page 21, the Moments and shears at critical sections can be proportioned from the values computed for the beam at front opening.

Dome Reinf.: (contd) Revised 12/2/83

Interior Wall opening 16'-0"

Total beam load per ft = 12780 lbs/ft

Total load on Front Beam = 6730 lbs/ft

$$\text{Ratio} = \frac{12780}{6730} = 1.90$$

By Proportions from pages 21 and 25, the interior beam design moments and shears are as follows:

Rectangular opening:

$$-M_u = 202 \times 1.90 = 384\ K'$$

$$+M_u = 101 \times 1.90 = 192\ K'$$

$$V_u = 66 \times 1.90 = 126\ K$$

Haunch opening:

$$-M_u = 252 \times 1.90 = 480\ K'$$

$$+M_u = 49 \times 1.90 = 93\ K'$$

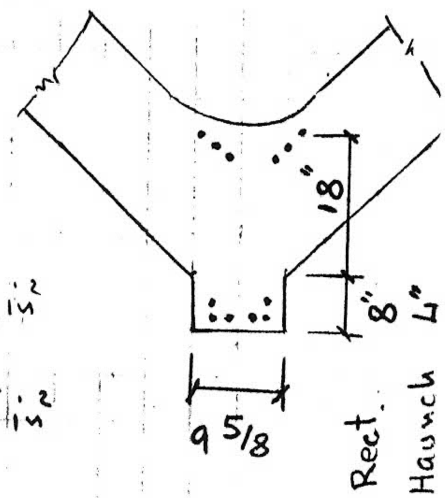
$$V_u = 66 \times 1.90 = 126\ K$$

Revised 12/2/83

Dome Reinf. (contd). Interior Wall Opening

Rectangular: $\pm d = 26''$

$$\frac{-M_u}{\varphi f'_c b d^2} = \frac{384 \times 12}{.9 \times 3.5 \times 10 (26)^2} = .2164$$



$\omega = .255 \qquad \rho = .255 \times \frac{3.5}{60} = .0149$

$-A_s = .0149 \times 10 \times 26 = 3.87 \text{ in}^2$

Provided 6 - #5 TOP continuous $= 1.86 \text{ in}^2$
6 - #5 × 6'-0" TOP Each End $= 1.86$
Total 3.72 in^2

$$\frac{+M}{\varphi f'_c b d^2} = \frac{192 \times 12}{.9 \times 3.5 \times 10 (26)^2} = .1082$$

$\omega = .116 \qquad \rho = .116 \times \frac{3.5}{60} = .0068$

$+A_s = .0068 \times 10 \times 26 = 1.77 \text{ in}^2$

As Provided 6 - #5 Bottom Continuous $= 1.86 \text{ in}^2$

Shear Reinf:-

#3 vertical stirrups @ 6" $\qquad A_{v_1} = .11 \times 2 = .22 \text{ in}^2$
$S_1 = 6''$

#4 bent V shaped bars @ 12"

Effective steel area in vertical direction
$A_{v_2} = 2 \times .2 \times .707 = .283 \text{ in}^2$
$S_2 = 12''$

Shear Capacity:

Steel $V_s = \frac{A_{v_1} f_y d}{S_1} + \frac{A_{v_2} f_y d}{S_2}$

$= \frac{.22 \times 60 \times 26}{6} + \frac{.283 \times 60 \times 26}{12} = 57 + 37 = 94 \text{ K}$