PDHonline Course C632 (4 PDH)

DOME DESIGN: Neither Intricate Nor Difficult

Instructor: Ruben A. Gomez, PE

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5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone: 703-988-0088
www.PDHonline.com

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DOME DESIGN
Neither Intricate Nor Difficult

Ruben A. Gomez, P.E.

1.0 Introduction

When an engineer engages in the design of linear or planar structures, the forces of nature work against him. Conversely, when he designs shells and membranes the forces of nature become his strongest ally. Therefore, the engineer is best served indeed when he follows the examples and lessons of nature.

As a matter of fact, almost everything in nature that is worth mentioning is, either tubular, spherical, elliptical or parabolic, so why then do we insist on living in cubic or prismatic houses. Good question, isn't it?

Years ago in a distant and different land, children would play and were marveled by the unusual strength of something otherwise very fragile: hen eggs, they would hold the raw egg in their hands and place a bet on who was the strongest one by cracking and crushing an egg by brute force, only to realize that it was an impossible task for a child to do. The illustration below (Figure 1.1) shows how to hold the egg in a vise like grip in an attempt to collapse it.

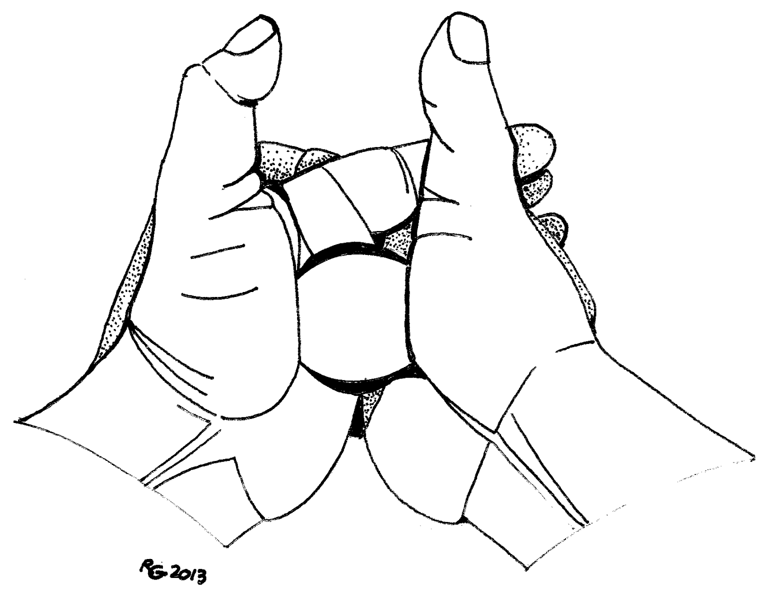

FIGURE 1.1

Why is the egg so strong? Many great minds have been baffled by that question for centuries before our generation. Why is it that something with a shell that thin and made out of such a brittle calcareous material, mostly calcium carbonate, could resist such a large force in comparison? Undoubtedly, locked in those eggs there was another lesson from nature waiting to be uncovered.

Based on those reminiscences, we made the decision to test a few eggs to failure so as to determine their ultimate compression capacity. Figure 1.2 basically depicts what we had in mind, as well as the average dimensions of the test eggs.

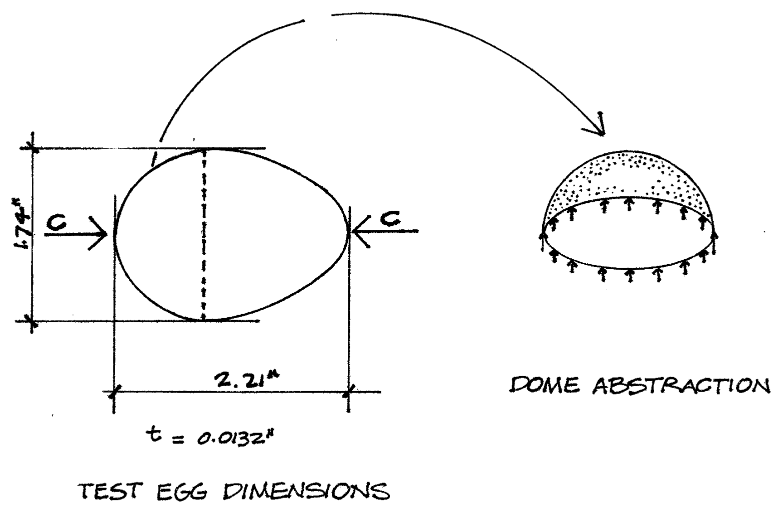

FIGURE 1.2

We built an egg compression tester composed of two parts, a stationary and permanently attached base furnished with a passive compression plate, and a sliding carrier with the other half, the active compression plate. To accurately measure the *pulling force* applied to the carrier we used a Wagner Friction Gage. Four large size eggs were used for the test as conducted by our lab assistant Miss Christine. Figure 1.3 below shows the testing set up as used to perform the test series during the afternoon hours of February 27, 2013.

Figure 1.3

From above figures we can deduct that the thickness to radius ratio was:

$$t/r = 0.0132/0.87 = 0.015 < 0.02$$

Denoting that the *membrane action* was also present as part of the very egg shell characteristics. Testing results were measured as follows:

Test #	Ultimate Compression Load (lbs)
1	18.2
2	23.8
3	17.9
4	23.2

Average ultimate compression load applied to the shell: 83.1 ÷ 4 = 20.775 (say 21 lbs.)

Lateral Area (half sphere): $12.566 \times 0.87^2 \times ½ = 4.756\ in^2 = 0.033\ ft^2$
Unitary Failure Compression Load: 21/0.033 = 636 PSF

In case you did not notice it, that value meant: the egg's failure compression load was the equivalent of a whopping 636 pounds per square feet.

Area of cross section at the base: A= 3.1416 x 1.74 x 0.0132 = 0.0722 sq. in.
Egg shell compression stress: P/A = 21/0.0722 = 291 PSI

These apparently trivial values will become of significance as we progress along the sections of this course.

2.0 HISTORICAL DEVELOPMENTS

The need to span over with the materials available, so to provide a roof to cover the spaces for human use started early in the development of human societies. At the beginning the logical material at hand was wood: timbers, logs and limbs were used as roofs to provide shelter against the inclemencies of the weather.

As early as 104 A.D. the Romans had been experimenting with masonry arches which they used on the Trajan's Bridge over the Danube River with a sequence of twenty arches for a total length of four thousand feet. They adapted the knowledge taken from the Assyrians to build masonry arches that later evolved into the masonry dome which they used to build the Pantheon.

After the collapse of the Roman Empire in 476 A.D. Europe fell into the Dark Ages and little was done to enhance the limited knowledge at hand. Slowly, contributions from great minds such as: Robert Hooke, Leonhard Euler, Charles Coulomb, Louis Navier, James Maxwell, Otto Mohr and Heinrich Muller-Breslau have brought a better grasp of the classic structural concepts which became the foundation of the accepted principles we use today.

During the period from 1945 through 1970 there were incessant and increasing efforts on research and development in the design and construction of domes, vaults, hyperbolic paraboloids, cylindrical shells and other forms of thin shells and membranes. Most of that effort took place notably in Italy, Spain, France, Mexico and the United States and a lot has been learned since from such experiences.

In 1957 the American Concrete Institute (ACI) organized its ACI Committee 334 with the mandate to prepare a set of guidelines for the design and construction of thin shells. They invited over a dozen of renowned engineers such as Alfred Parme, Anton Tedesko, Mario G. Salvadori, Milo Ketchum, Felix Candela and a few others. In 1964, after a lot of discussion and debates, they finally came up publicly with a report titled "Concrete Shell Structures, Practice and Commentaries".

But it was not without conflict, within the committee itself there were dissenting voices that rejected the objectives, wording and conclusions of the report on the basis of bias against freedom of design and excessive conservatism. In the end two members, Mario Salvadori and Felix Candela refused to endorse and sign the work of the committee.

We cannot let pass the opportunity to comment about Mr. Candela's good and bad positions when it came to express his viewpoints in the areas of design and construction of thin shells. Beyond any doubt, he was a man of great experience and had in his account the erection of scores of shells in Mexico City, and probably more so than any other engineer in the American continent. He had to his merit the construction of many short shells, cylindrical shells, funicular vaults, conoidal shells and hyperboloids.

We had the opportunity to see some of his work consisting of complex forms of paraboloids erected just by following the "rules of thumb" he had accumulated during his long practice. In fact, these words summarized well his position when it came to the value of "fancy calculations" as he called them:

"In view of the impossibility, when dealing with a structural problem, of taking into account all the secondary conditions, such as shrinking of concrete, temperature differentials, differential settlements, added to the typical imprecision of the construction process itself, it is obvious that mathematical calculations cannot be as an useful and dependable solution to the design problem".

As Mr. Candela regretted having accepted his been part of the ACI Committee 334, he adamantly continued his harangue:

"The imposing stone vaults of Gothic cathedrals and the daring domes of the glorious Renaissance were built without the help of any differential calculus, but instead, with a great deal of sense of equilibrium and sound judgment of the forces involved."

While it is true, as Mr. Candela also asserted, that some writers seem to enjoy showing their skills and their knowledge of advanced mathematics with the use of "fancy calculations", further, they in many instances even would conceive designs that are neither buildable nor comprehensive. It is not less true that some of the rest of us do show a sign of duty and responsibility by going out of our way in verifying that every part of the structure is sized according to the properly calculated and predicted stresses. Moreover, and by also making sure that the simple principle of the rules of equilibrium checks and balances are never ignored or overlooked.

3.0 WHY DOMES?

As we have already either indicated or suggested the term *thin shell* is a generic designation which comprises a myriad of surfaces of revolution from domes, vaults and conoids all the way to hyperboloids and hyperbolic paraboloids. The reason we have chosen domes is because of their relative simplicity of construction. Further, we also have picked spherical domes because they are of constant radius, which makes the lay-out easier to accomplish and allows location and relocation of particular points within the surface of the structure. Of course, like everything else in this world, the construction of thin shells has its advantages as well as disadvantages. Candidly we will go through a description of the pros and cons in such a way that any enthusiast is fully aware of the total picture ahead of him on his project.

4.0 GENERAL DESCRIPTION

The dome you are about to see is technically designated as an *icosahedron* generated by triangular panels in an overlapping pentagonal arrangement. Since such shape is part of the family of *polyhedras* which evolve towards the perfect sphere. Generically speaking we refer to them as *spherical domes*. Further, the reader will notice that the herein presented calculations are decidedly based on the stresses found in spherical surfaces as an acceptable approximation for the purpose of practical application. Of course, by making such a statement we find ourselves again falling within the confines of Mr. Felix Candela's contentions.

Formula derivations and applications will be given straight to the point, just plain math, algebra and trigonometry and, as a tribute to Mr. Candela, avoiding at all cost the intricacies of integral and differential calculus.

5.0 ADVANTAGES OF THE DOME

The advantages are many and extend far into a large area of endeavors, such as:

1. Spherical domes require less than half of the amount of materials necessary to enclose the same volume contained in a conventional structure.

2. Surface areas exposed to heat and cold are much smaller so that energy demand is cut in half.

3. Required erection time is substantially smaller.

4. They offer enormous strength against natural forces such as wind, earthquakes and snow load. Such strength is many times over the strength of post-and-beam solutions.

5. This one is dedicated to the housewives who still clean their houses on their own, it is a lot easier to clean a round surface with just one stroke of your cleaning rag than a sharp angular corner.

6. Requires less maintenance and have a longer design period than homes following

conventional construction.

6.0 DISADVANTAGES

As a matter of fairness, after having listed the many advantages of the spherical dome in its residential application, it would be only just to also go over those [relative] qualities that most people consider to be drawbacks, and here they are:

1. They resemble large *igloos* as built by the Eskimos of Alaska. Consequently, their unconventional appearance seems to be objectionable to conventional people.

2. In the construction process, there is a lesser need for unskilled labor.

3. Rooms around the perimeter are of odd shape and size and it is hard to fit in pieces of conventional rectangular furniture. For the benefit of the dome enthusiasts, it must be said here that the solution to the problem posed by the standard furniture lies in designing and fabricating those pieces of furniture along with the shape of the spaces generated by the dome, or have them custom made, if you follow our reasoning.

4. The variable headroom is an objectionable item to conventional people, especially the tall ones.

7.0 PRACTICAL APPLICATION

In early April 1994 our office entered into a contract with a client who had just purchased a set of conceptual architectural drawings on a generic spherical dome. He was a clock maker with a very accurate mind and affinity for detail, however, he had absolutely no experience in construction and no knowledge of materials nor did he know the typical jargon of the industry.

As part of his plan he had purchased a 5 acre parcel of land in the outskirts of a small town called Crawfordville in Wakulla County, a few miles southwest of Tallahassee, Florida. The scope of our work had three areas of engagement:

a. preparation of a set of engineering drawings (structural, electrical and mechanical),
b. to design a step-by-step construction program, including a CPM chart, and
c. to prepare shop drawings and provide field assistance on the as-needed basis.

The project consisted of a split level 40 ft. diameter spherical dome set over a conventionally built monolithic slab on ground and another 24 ft. diameter spherical dome set also on a concrete slab, both domes were connected together by a breezeway.

Parts (a) and (b) were completed and delivered in late July 1994. As part of the package we included a scale model of the main dome (Figure 7.1) so as to help him visualizing the finished shell. We did not hear from him again until March 1999 when he called with a sense of urgency and anxious to start working on his lifetime dream. The first change came up in a couple of days, now he wanted a full basement with a garage door and enough space for his clock shop.

FIGURE 7.1

Although we were inundated with questions and requests for changes almost on weekly basis, they progressed very efficiently during a period of 6 months in the construction of the smaller dome as a way to acquire the necessary experience to tackle the bigger job. At the completion of the smaller dome, we received a picture at this stage of the process which is herein enclosed as Figure 7.2. The owner moved in and used the smaller dome as living and operating quarters, while doing the rest of the work. Most of the routine work was done by the owner himself and his diligent wife, and the project was finally completed in December 2000.

FIGURE 7.2

Structural Features

A partial set of the prepared structural calculations are enclosed as a sampler for the reader and/or the engineer practitioner as well. Let us follow the main steps to facilitate comprehension:

Sheet #1

The common shell concrete thickness was established as 2 in., except at the base, crown and some vertices where it had to be thickened for higher stresses or to accommodate the placing of additional reinforcing bars.

The relationship of thickness to radius was 0.007 < 0.02 within the membrane action limits, such as in the case of the tested egg above.

The total dead load was calculated and the resulting value was divided by the projected area of the dome to obtain a unitary dead load of 52 PSF. The combined dead load plus live load added to 82 PSF.

Notice the meridian of stress polarity change located at the angle $\theta = 51° - 50'$.

Sheet #2

Meridian forces and stresses were determined at dome's base, toe and lintels. The stresses were small including those at the base (point C). Nevertheless, it will be noticed later that the thickness at the base of shell was increased anyway for practical reasons to accommodate the additional reinforcing bars.

The total reaction load delivered at the base of the foundation was just 1,100 lbs/ft for a resulting soil pressure of 827 PSF, suitable for almost any poor soil. In the case at hand, that load was rather to be added on top of the basement wall.

Sheet #3

Stresses due to temperature differentials were calculated and the temperature re-bars provided accordingly.

Reinforcement for lintels over window and door openings was designed accordingly. For empirical and practical purposes, a continuous tie-band was introduced at that level and the concrete section increased accordingly, as will be seen on coming details. The shear force and resulting stresses at the opening's jamb were found to be within adequate limits at the thickened base of the shell.

Sheet #4

Although wind forces acting on domes are almost irrelevant because of the way wind filaments and gusts get deflected over the curved surfaces of the shell, however, overturning moments need to be taken into account. In this case they were verified and given the proper consideration.

Sheet #5

After calculating the impact of the full 120 MPH hurricane wind forces on the shell, it was concluded that the so determined compression and tension stresses were small and in the order of 22 and 35 PSI respectively.

Sheets #1A and 2A

These two sheets were prepared to reflect and verify the impact of those changes requested by the owner, namely: a) the change from 6 in. expanded polystyrene to 7 in. extruded polystyrene, b) deletion of the inside cement stucco, and c) the addition of gypsum board as a finish choice for all the interior surfaces of the dome, as to match those finishes of the interior partitions.

RUBEN A. GOMEZ, P.E.
STRUCTURAL ENGINEER

JOB: 40' SPHERICAL SHELL DOME
SHEET NO. 1 OF
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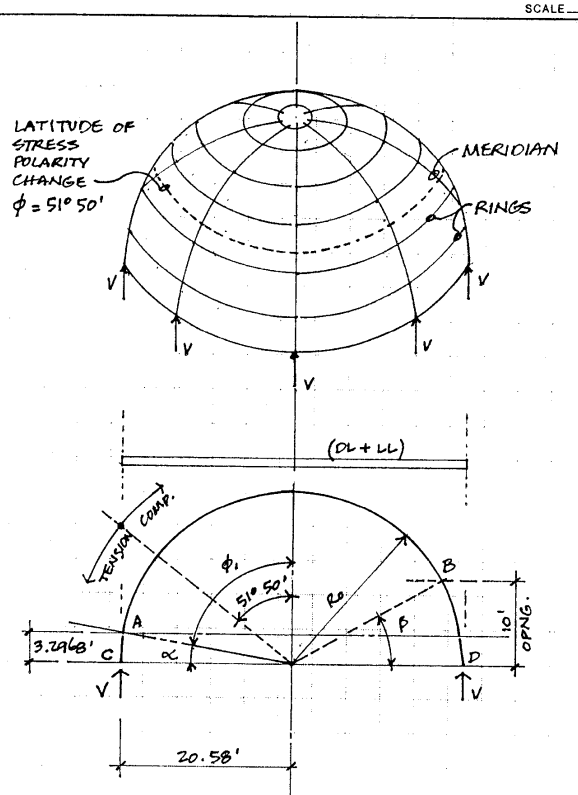
SCALE



$R_o = 20.58'$
LATERAL SURFACE : $\frac{1}{2}\pi D^2$
$D = 41.16'$
$S = 0.50 \times 3.14159 \times 41.16^2 =$
$= 0.50 \times 3.14159 \times 1694.1456 =$
$= 2{,}661.16 \ FT^2$
$t = 2"$
$p = 25$ PSF (USING 150 PCF CONCRETE)
$W = 2{,}661.16 \times 25 = 66{,}529$ LBS.
PLUS,
POLYSTYRENE :
$2661.16 \times 0.50 =$ 1,330 LBS.
WOOD FRAMING :
(2x4's)
$972 \times 1.40 =$ 1,361 "

TOTAL DEAD LOAD : 69,220 LBS.
ALLOWANCE FOR {MISC. HARW. / WTRPR. MEM.} 780 "
PROJECTED AREA :
$R_o' = 20.67'$
$D' = 41.34'$
$A = 0.7854 \times 1{,}708 = 1{,}343 \ FT^2$
UNITARY DEAD LOAD :
$\frac{70{,}000}{1{,}343} = 52$ PSF

PROJECTED : DL + LL : 82 PSF

RADIAL : DL + LL : 42 PSF

$tg\,\alpha = \frac{3.30}{20.58} = 0.16$ $\alpha = 9°$

$\phi_1 = 81°$

RELATIONSHIP OF THICKNESS TO RADIUS :
$t/R = 0.16 \div 20.58 = 0.007 < 0.02$
MEMBRANE ACTION WILL OCCUR.
$\sin\beta = 10/20.58 = 0.4859$ $\beta = 29°$
$\phi_2 = 61°$

STRESSES @ CRITICAL POINTS :

POINT A : $\phi_1 = 81°$ MERIDIAN FORCE : $T_\phi = -0.86 \times 20.58 \times 42$
$= -743$ LBS

MERIDIAN STRESS : $f_\phi = \frac{743}{24} = 31$ PSI (COMPRESSION)

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STRUCTURAL ENGINEER

JOB 40' SPHERICAL SHELL DOME
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RING FORCE: $T_\theta = 0.75 \times 20.58 \times 42 = 648$ LBS (TENSION)

RING STEEL: 2(6x6-10/10) WWF
4 WIRES (@ 0.014) = 0.056 in²
$648/24{,}000 = 0.027$ in² < 0.056

POINT B: $\phi = 61°$

MERIDIAN FORCE: $T_\phi = -0.68 \times 20.58 \times 42 = -588$ LBS (COMP.)

MERIDIAN STRESS: $f_\phi = \frac{588}{24} = 25$ PSI. < f_{ALLOW}.

RING FORCE: $T_\theta = 0.20 \times 20.58 \times 42 = 173$ LBS (T)
$A_S = 173/24{,}000 = 0.007$ in² < 0.056

POINT C (E.D): $\phi = 90°$

MERIDIAN FORCE: $T_\phi = -1.0 \times 20.58 \times 42 = 865$ LBS (C)

MERIDIAN STRESS: $f_\phi = \frac{865}{36} = 24$ PSI < f_{ALLOW}.

RING FORCE: $T_\theta = 1.0 \times 20.58 \times 42 = 865$ LBS (TENSION)
$A_S = 865/24{,}000 = 0.036 < 0.056$ in²

MERIDIAN FORCES ALWAYS COMPRESSIVE.
RING FORCES ARE COMPRESSIVE FROM THE POLE DOWN TO THE 51° 50' COLATITUDE, THEN TENSILE FROM THERE TO THE BASE.

REACTIVE FORCES ON TENSION RING:

$R_0 = 20.58'$ $t = 2"$ DL+LL = 42 PSF (RADIAL)
$\phi = 90°$ $T_{\phi_1} = -1.0 \times 20.58' \times 42 = -865$ LBS./FT.
$T_\theta = +865$ LBS (TENSION)

TOTAL TENSION ON RING: $\Sigma T = (\pi \cdot R_0) 865 = 55{,}922$ LBS.
$A_S = \frac{1}{2} \times \frac{55922}{24{,}000} = 1.16$ in² < 4 #5

ALSO, V = 1.0 x 865 = 865 LBS/LIN. FT. OF EDGE.

FOOTING: 235 "
1,100 LBS/FT.

B = 16" (1.33')

SOIL PRESSURE: $f_S = \frac{1{,}100}{1.33} = 827$ PSF

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STRESS DUE TO THE DIFFERENCE IN TEMPERATURE BETWEEN OUTER AND INNER FACE OF SHELL:

$$f_t^i = (0.50 \times 0.000008 \times 3{,}120{,}000)\left(1 + \frac{2}{3}\frac{h}{R_o}\right)\Delta t$$

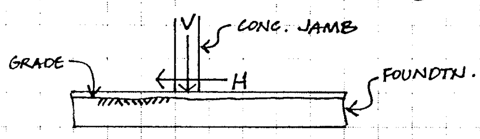
$E_c = 57{,}000\sqrt{3000} \cong 3{,}120{,}000$ PSI. $h = R_o$

THUS, $f_t^i = (0.50 \times 24.96 \times 1.67)\Delta t = 20.84\,\Delta t$

$\Delta t = 15°F$ (A/C COOLING CAP.)

$f_t^i = 313$ PSI. $A_s = 313/24000 = 0.013 < 0.056$ in²

FORCES ON CONCRETE JAMB @ OPENINGS:

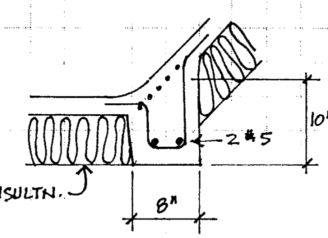


OPENING WIDTH: 15.82'

$V = 865 \times 15.82 \times \frac{1}{2} = 6{,}842$ LBS. (COMPRESSION)

H (SHEAR) = 6842 LBS.

SECTION AVAILABLE:



V_c $< 5 \times 54.77 \times 64 = 17{,}526$#
$> 2 \times 54.77 \times 64 = 7{,}011$#

$v = \frac{6842}{64} = 107$ PSI

WIND FORCES:

WIND VELOCITY: 120 MPH $h = 21'$

$W = 33$ PSF BUILDING SHAPE FACTOR: 0.70

$W' = 33 \times 0.70 = 23$ PSF

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STRUCTURAL ENGINEER

JOB 40' SPHERICAL SHELL DOME
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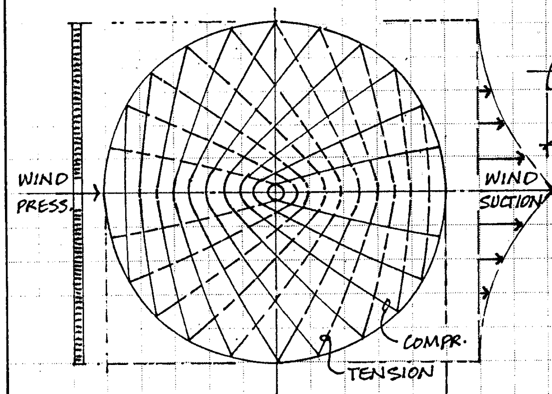
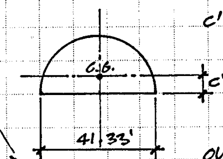


STRESS PATTERN



$c' = 0.4244 \times 20.67 = 8.77'$

$A_W = 1.571 \times 20.67^2$
$= 671.19$ FT²

$W = 671.19 \times 23 = 15,438$ LBS.

OVERTURNING MOMENT:

$A_P = 3.142 \times 20.67^2 = 1342.38$ FT²

$P = 1342.38 \times 82 = 110,075$ LBS.

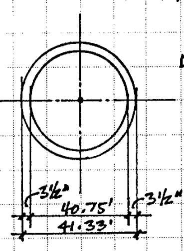
$M_o = 15.44^K \times 8.77 = 135.40$ FT-KIPS

STABILIZING MOMENT:

$M_S = 110.08 \times 20.67 = 2,275$ FT-KIPS

(CONTRIBUTION OF FOUND. SLAB DISREGARDED).

SAFETY FACTOR: $\frac{2,275}{135.40} = 16.8 > 1.50$

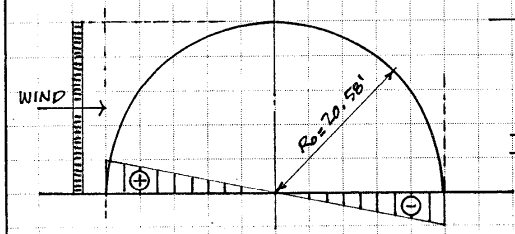


DOME FOOT PRINT @ BASE:

$A_c = 0.7854\,(41.33^2 - 40.75^2)$
$= 0.7854\,(47.61) = 37.39$ FT²

$c = \frac{1}{2} \times 41.33 = 20.67'$

$I = 0.0491\,(41.33^4 - 40.75^4)$
$= 0.0491\,(2,930.70)$
$= 143.88$ FT⁴



SECTIONAL VIEW OF STRESS DISTR.

$f_c = P/A \pm Mc/I$

$P = 110,075$ LBS.

$A_c = 37.39 \times 144 = 5,384$ in²

$M = 135.40 \times 12000 = 1,624,800$ IN-LBS

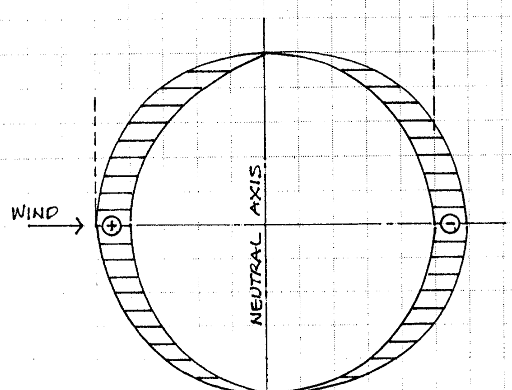
$c = 20.67 \times 12 = 248$ in.

$I = 143.88 \times 20,736 = 2,983,496$ in⁴

$P/A = 110,075/5384 = 20.44$ PSI.

$Mc/I = \frac{1,624,800 \times 248}{2,983,496} = 135.6$ PSI.

$f_c = 156.0$ PSI (COMPRESSION) @ BASE:



PLAN VIEW OF STRESS DISTRIBTN.

STRESSES DUE TO WIND + DL AT RING B:

CENTRAL ANGLE $A = 2 \times 61 = 122°$
RISE/CHORD COEFF. $= 0.2945$

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JOB 40' SPHERICAL SHELL DOME
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STRUCTURAL ENGINEER

$\frac{r'}{C} = 0.2945$ $r' = 10.58'$ $C = \frac{10.58'}{0.2945} = 35.92'$ (DIAMETER OF DOME @ B).

LATERAL SURFACE OF SPHERICAL SECTOR ABOVE RING B:

$S = 2 \times 3.1416 \times 20.58 \times 10.58 = 1,368$ FT^2

TOTAL DEAD LOAD ABOVE RING B: $1,368 \times 42 = 57,459$ LBS.

SURFACE AGAINST WIND: $A_W = 272$ FT^2 $W = 6,256$ LBS.

$c' = 4.52'$ $M_o = 6,256 \times 4.52 = 28,277$ FT-LBS. $= 339,325$ in-lbs.

$A_c = 0.7854\,(433^2 - 429^2) = 0.7854\,(3,448) = 2,708$ in^2

$c = 216$ in.

$I = 0.0491\,(433^4 - 429^4) = 0.0491\,(1,281,035,440) = 62,898,840$ in^4

$\phi_2 = 61^\circ$ $\varphi = 0^\circ$ MERIDIAN STRESS: $Z = Z_o \cdot \sin\phi \cdot \cos\varphi$

$\sin 61^\circ = 0.8746$

$\cos 0^\circ = 1.0000$

$f_c = P/A \pm Mc/I$ $P/A = \frac{57,459}{2708} = 21.2$ PSI $Mc/I = \frac{339,325 \times 216}{62,898,840} = 1.16$ PSI.

$T'_\phi = 588\left(\frac{27}{42}\right) + (22 \times 24)\,0.8746 = 376 + 462 = 838$ LBS/FT.

$f'_\phi = \frac{838}{24} = 35$ PSI $<$ f_{ALLOW}.

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JOB 40' DOME - REVISIONS TO ED OVGOS
SHEET NO. 1A OF
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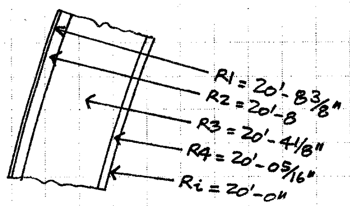
CHECKED BY DATE
SCALE 12 of 12

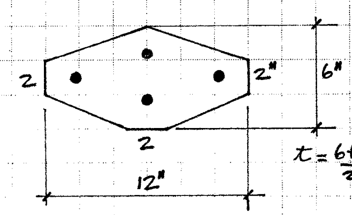


COMPONENT	R (FT)	DIA. (FT)	LAT. SURF. (SF)
WTRPROF. MEMBR.	20.70	41.40	2,692
CONC. SHELL	20.67	41.34	2,684
7" INSULATION	20.34	40.68	2,599
5/8 GYP. BOARD	20.03	40.06	2,521
FINISH SURFACE	20.0'	40.00	2,513

COMPONENT WEIGHTS

WATERPROOFING MEMBRANE :	2,692 x 0.20 =	538	LBS
CONCRETE SHELL :	2,684 x 9.06 =	24,317	"
7" POLYSTYRENE	2,599 x 0.58 =	1,507	"
5/8" GYPSUM BOARD	2,521 x 2.50 =	6,302	"
CONCRETE RIBS (WEIGHED) :	1,040 x 38.79 =	40,342	"
MISCELLANEOUS HARDWARE :	2,513 x 0.25 =	628	"
	TOTAL DEAD LOAD :	73,634	LBS

PROJECTED AREA : A = 1,346 SF



$t = \frac{6+2}{2} = 4"$

RIB CONFIGURATION

UNITARY DEAD LOAD : $\frac{73,634}{1,346} \cong 55$ PSF

PROJECTED LIVE LOAD:
SEC 1203.6 : 12 PSF

3/4" SHELL
(0.33 - 0.0625) 1.0 x 145 = 38.79 LBS/FT

PROJECTED DL + LL : 67 PSF

RADIAL DL + LL : 43 PSF

$D_o = 2R_o = 41.40'$
$L_o = 3.1416 \times 20.70 = 65.03'$
$\frac{D_o}{L_o} = \frac{41.40}{65.03} = 0.6366$

THE PROPOSED ALTERNATE IS HEREBY CHECKED AS A SPHERICAL SPACE FRAME WHERE THE PRECAST PANELS ARE NOT CONSIDERED TO CONTRIBUTE TO THE OVERALL STRENGTH, AND ACT AS FILLER PANELS, THUS PROVIDING CERTAIN FLEXIBILITY IN LOCATING WINDOW OPENINGS AND SKYLIGHTS.
THE INTERSECTING RIBS SHALL THEN CARRY ALL LOADS TO THE FOUNDTN.

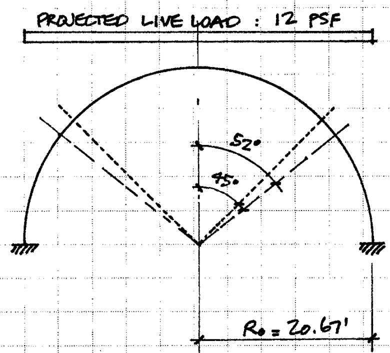
PRODUCT 204-1 NEBS Inc., Groton, Mass. 01471.

RUBEN A. GOMEZ, P.E.
STRUCTURAL ENGINEER

JOB 40' DOME
SHEET NO. 2A OF
CALCULATED BY R.G. DATE 09/19/99
CHECKED BY DATE
SCALE 11 of 12

TOTAL DL : 73,634 LBS   TOTAL RIB LENGTH : 1,040 LF

TOTAL DL PER LF : $\frac{73,634}{1,040} = 70.8$ LBS/FT. ~ 71 LBS/FT.



AXIAL FORCES ON RIB:

RADIAL DL = 71 LBS/LF

PROJ. LL = 84 " (12x7)

MAX. AXIAL FORCE : (HAAS MANUAL)

$T\phi = (1.0 \times 20.67 \times 71) + (0.50 \times 20.67 \times 84)$

$= 1,468 + 868 = 2,336$ LBS.

UNBRACED LENGTH : 8'-0"

$\ell/d = \frac{96}{6} = 16$ $P/A = \frac{2336}{48} = 49$ PSI.

$As_{min} = 0.005 \times 12 \times 4 = 0.24$ in² < 2 #4

TOTAL THRUST @ BASE :

$T_B = 0.55 \times 20.67 \times 155 = 1,762$ LBS/FT.

$\Sigma T_B = 0.47 \times 20.67 \times 1,762 = 17,118$ LBS.   12 RIBS.

TOTAL THRUST PER RIB: $17,118 \div 12 = 1,427$ LBS   SHEAR STRESS: $\frac{1427}{48} = 30$ PSI (OK)

REACTION @ BASE : $R = 0.57 \times 1,762 \times 3.1416 \times 41.40 = 130,627$ LBS.

CRUSHING FORCE (PER RIB) IF FILLER PANELS ARE TOTALLY IGNORED: $\frac{130,627}{12} = 10,886$ LBS.

CRUSHING STRESS : $\frac{10886}{48} = 227$ PSI. (OK)

WIND PRESSURE BASED ON A VELOCITY OF 120 MPH   W' = 23 PSF

TOTAL LATERAL FORCE : H = 15,484 LBS.

THRUST FORCE PER RIB : $15,484 \div 12 = 1,290$ LBS (WIND ONLY)

654 " (DL ONLY)

1,944 LBS   SHEAR $v = \frac{1,944}{48} = 41$ PSI (OK)

WIND UPLIFT :

V = 120 MPH   h = 21'   W = 33 PSF   SHAPE FACTOR : 0.70

RADIAL UPLIFT. : 23 PSF   A = 2,513 SF $F_W = 57,799$ LBS.

MIN. VERT. STEEL : $A_S = \frac{57,799}{20,000} = 2.89$ in²   WEIGHTLESSNESS ASSUMED.

STEEL FURNISHED : $2 \times 12 \times 0.20 = 4.80$ in² > 2.89   (#4 BARS) USE 2 #4

$2 \times 12 \times 0.11 = 2.64$ in² < 2.89   OR 4 #3

[MINIMIZE & STAGGER SPLICES]

PRODUCT 204-1 NEBS Inc., Groton, Mass. 01471.

Energy Conservation Features

To improve even further the inherent energy conservation characteristics of the dome, it was added a 6 in. inner layer of *expanded* polystyrene with an insulation value of R = 18 with the extra benefits of being walkable and also used as a "permanent formwork" for the concreting of the dome shell.

Even though the smaller dome was built on the above design premise, during construction of the larger dome the owner decided to further increase the insulation panels from 6 to 7 inches and switch to *extruded* polystyrene with the resulting increased of the R-value to 35. This change was to a large degree unnecessary but he did not want to hear any arguments to the contrary, so in addition to the added dead weight to the dome, he had to face the typical consequences of the "overkill" or what has come to be known to many as *the diminishing returns.* More details will follow as the course further develops.

Construction Details

Enclosed Figure 7.3 is depicting a partial view of the first floor plan of the dome showing the focal point at the center of the polygonal plan. We picked the half view of the plan where the openings where located for a better illustration. By being a spherical dome, the dimension of 20'-5" is a constant measurement from the center to all vertices. TB-2 is a tie-beam below and over the now added basement masonry bearing wall, while FB-2 and FB-3 are concrete beams spanning over basement garage door openings.

Door and window openings were allowed for as long as they were kept typically the same in form and dimensions. That was one of the main reasons why the tie-band was introduced above all openings and along the entire perimeter of the dome.

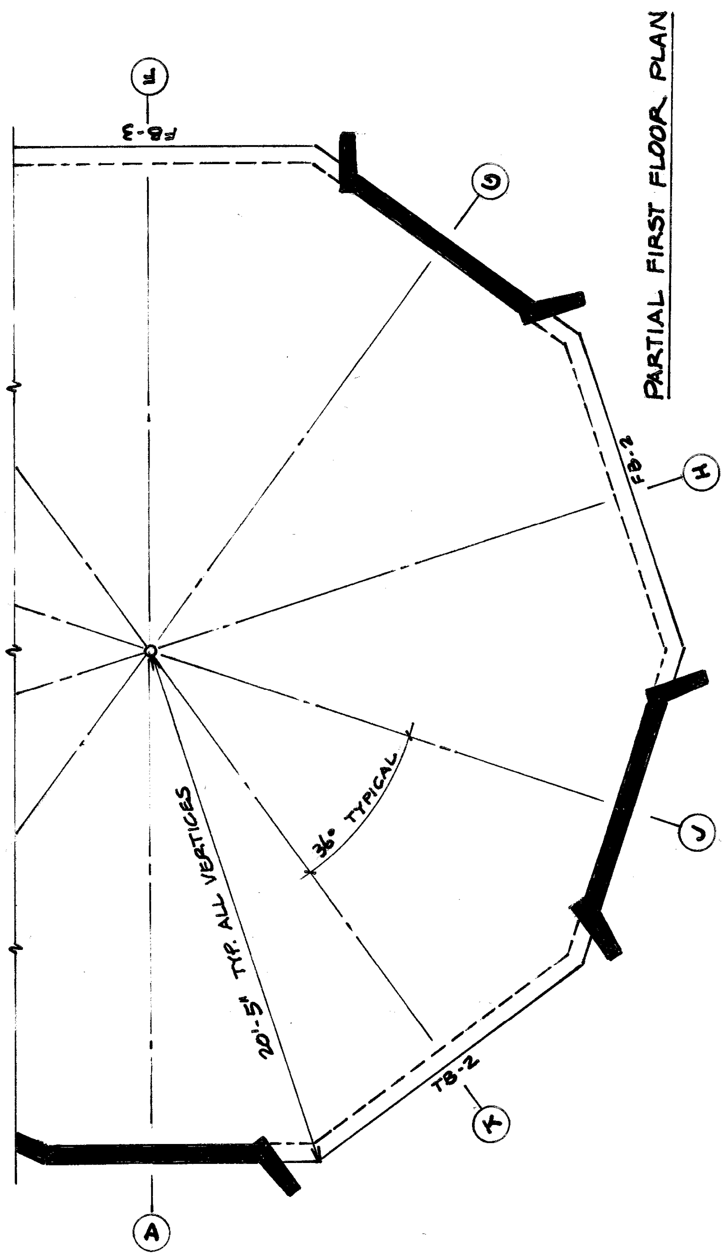

PARTIAL FIRST FLOOR PLAN
F
FB-3
G
H
FB-2
J
K
TB-2
A
20'-5" TYP. ALL VERTICES
36° TYPICAL

FIGURE 7.3

Figure 7.4 shows a partial view of the roof. Panel joints have been indicated as single lines for simplicity although they were actually double lines.

A comment is proper at this juncture: reinforced concrete develops cracks for two main reasons, if we dismiss the possibilities of poor quality, creep and substandard handling, those reasons are: material shrinkage and temperature differentials. The former can be helped by observing proper curing procedures, but other than adding temperature reinforcement little can be done to help the latter. Consequently, some cracks are to be expected, actually some colleagues are of the conviction and belief that "cracks belong in concrete".

Since the problem of shrinkage can be helped with the proper curing procedure. Two conditions are necessary for the proper curing to take place: a) the presence of moisture for the hydration process to be uninterrupted, and b) the presence of a warm and steady temperature. Let us examine what can reasonably be done to make sure that hydration takes place as intended. These are the common options:

Steam Curing
Steam curing is the ideal method because it fulfills the two conditions of maintaining moisture and temperature on a steady basis. In order to achieve correct steam curing the freshly poured concrete has to be enclosed in a curing chamber. The following items are necessary: steam generator, three blowers, steel or fiberglass framing, enough canvas to completely cover the structure, power and water. Although ideal, in most cases the cost of this procedure may be too expensive to impose on a little job. Some other methods may have to be implemented if its cost is over budget.

Wet Burlap Covering
This method contemplates the structure to be covered with overlapping burlap sheets in its entirety and the burlap saturated with clean water at frequent intervals so to maintain a steady humidity and be heavy enough to remain in place even during windy days. This is the second best method after steam curing. What are needed here are the burlap sheathing, a hose and a steady source of water.

Open-sky Sprinklers
As a third alternative, a previously piped over-ground sprinkler system can be used in such a way that the sprinkler heads deliver a steady and continuous mist over the entire area of the dome for several days.

Hand Watering
The fourth alternative could be called "the poor man's method" consisting of frequent and intermittent water applications over the concrete surface starting two hours after concreting completion. Since loss of humidity is what causes concrete to shrink, the material must be kept constantly humid. Therefore, water applications must be managed in such a way that the concrete surface is never allowed to dry and avoid the wet-dry-wet condition, otherwise, this method would be worse that not doing anything at all.

By effectively using one of the above methods, cracking due to shrinkage could be averted. However, those undesirable cracks still may occur due to temperature changes taking place during the life of the structure.

Considering all the above and even if cracks were greatly reduced, it is an inescapable fact that the dome concrete surface needs be treated for water-tightness and very fortunately, there are several options to achieve that purpose in an effective and cost competitive way. The first solution that comes to mind is the *elastomeric membrane,* which comes in buckets in a liquid form and is applied with either brush or rollers. The second solution consists of the use of a torch applied *modified bitumen membrane* which has been used in the roofing market for over thirty years with very satisfactory results. Once the roofing problem is solved in an adequate manner, the consumer will have a dome that will satisfactorily perform its intended purpose for a lifetime.

On the roof plan depicted we purposefully have not shown any penetrations such as, pipe vents, exhaust fans, conduits, skylights or air exchangers, so as not to distract from the inherent facts and values of the herein described system. It is however important to notice that every single penetration has to be properly protected with boots, cants and flashings as necessary to prevent loss of integrity of the roofing system.

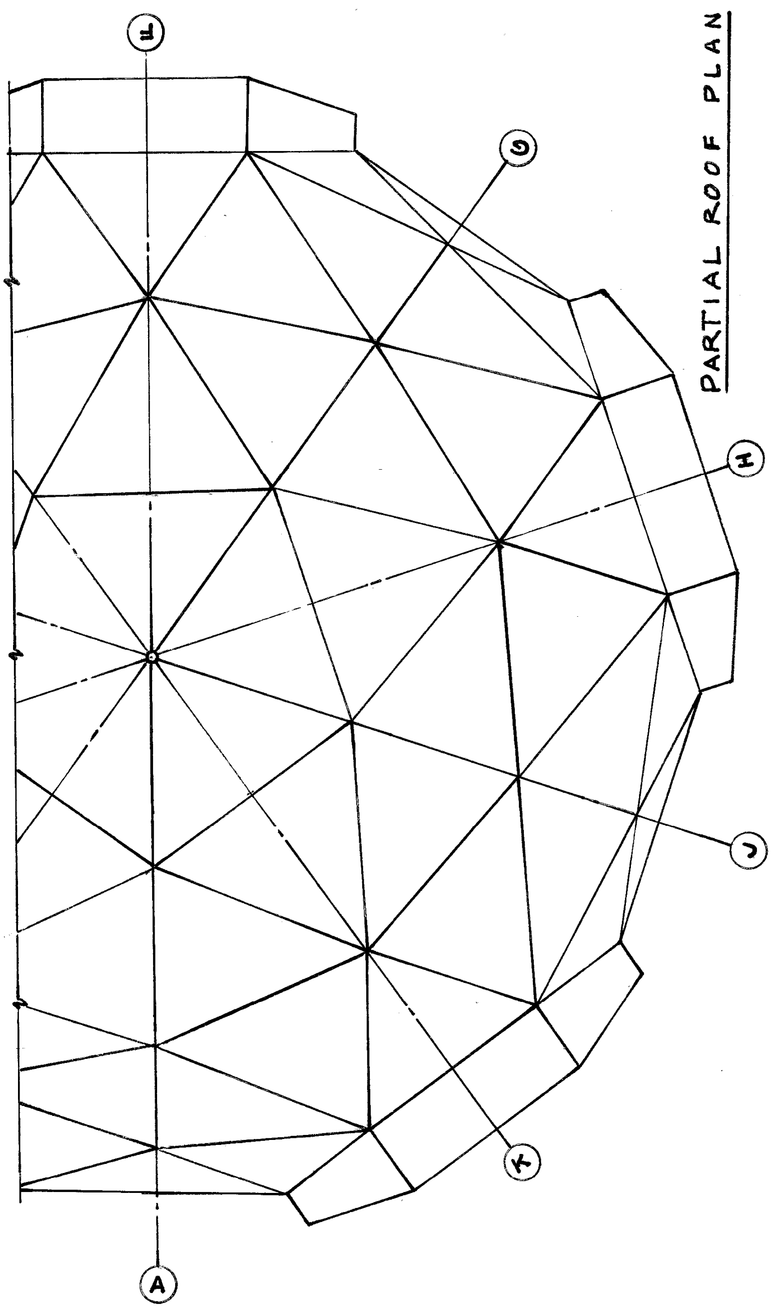


FIGURE 7.4

Figure 7.5 is showing a side view of the dome with its typical reinforcement consisting of a double layer of galvanized welded wire fabric as used through the entire shell. In addition to the regular fabric, 2 #3 bars (6 ft. long) were used at each side of every opening. In order to make a safe transition (again Mr. Candela's viewpoint) between the continuous shell and the lower part where the window and door openings occurred, a tie-band was provided reinforced with 4 #3 continuous bars in a thickened panel joint as being shown on the next figure. #3 x 2 ft. @ 16 in. dowels were also provided to anchor the shell to the foundation, or in the client's case, to the basement wall tie-beams.

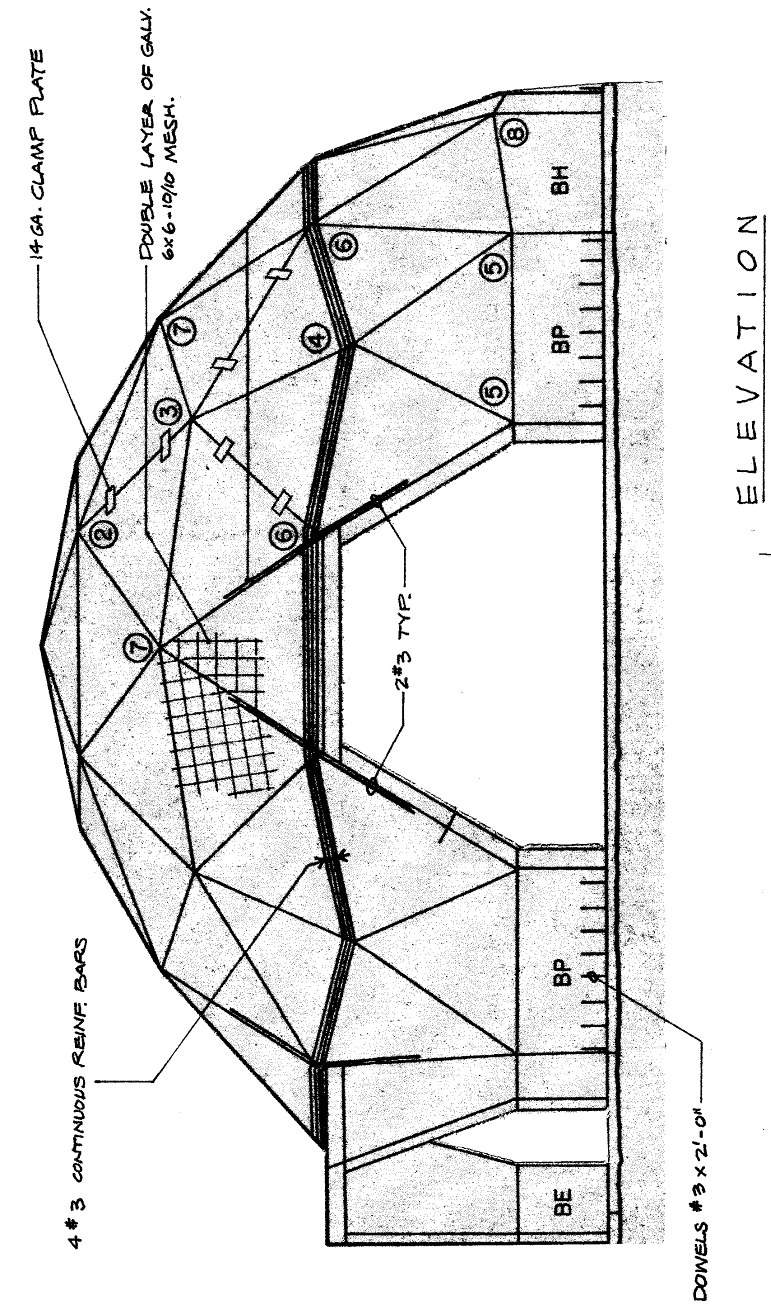

14 GA. CLAMP PLATE
DOUBLE LAYER OF GALV.
6x6-10/10 MESH.
2#3 TYP.
4#3 CONTINUOUS REINF. BARS
DOWELS #3 x 2'-0"
BH
BP
BP
BE
ELEVATION

FIGURE 7.5

Figure 7.6 describes two different sections through the shell panel points, the one on the left was typically applied to all joints, except those falling along the tie-band which were further thickened to 4 inches so as to make room for the added reinforcement and its splices. When it came to splices, the recommendation was not to allow bar splices to take place at the same place, but rather a minimum of four feet apart.

Those above described were the details originally provided to the client. Later on he decided to do away with the interior stucco and add drywall finish instead with the resulting three drawbacks being described on the next figure.

Going back to the energy conservation topic, the following was the R-Value break down as it had been prepared on the original design:

Outside air film:	0.17	
Waterproof membrane:	0.20	
2″ concrete shell:	0.48	
6″ expanded polystyrene:	18.00	
¾″ cement stucco:	0.18	
Inside air film:	0.68	Total R-Value: 19.71

To compensate for all the tapering and shaving of the polystyrene at the different places where called for, we rated the assembly as R-19.

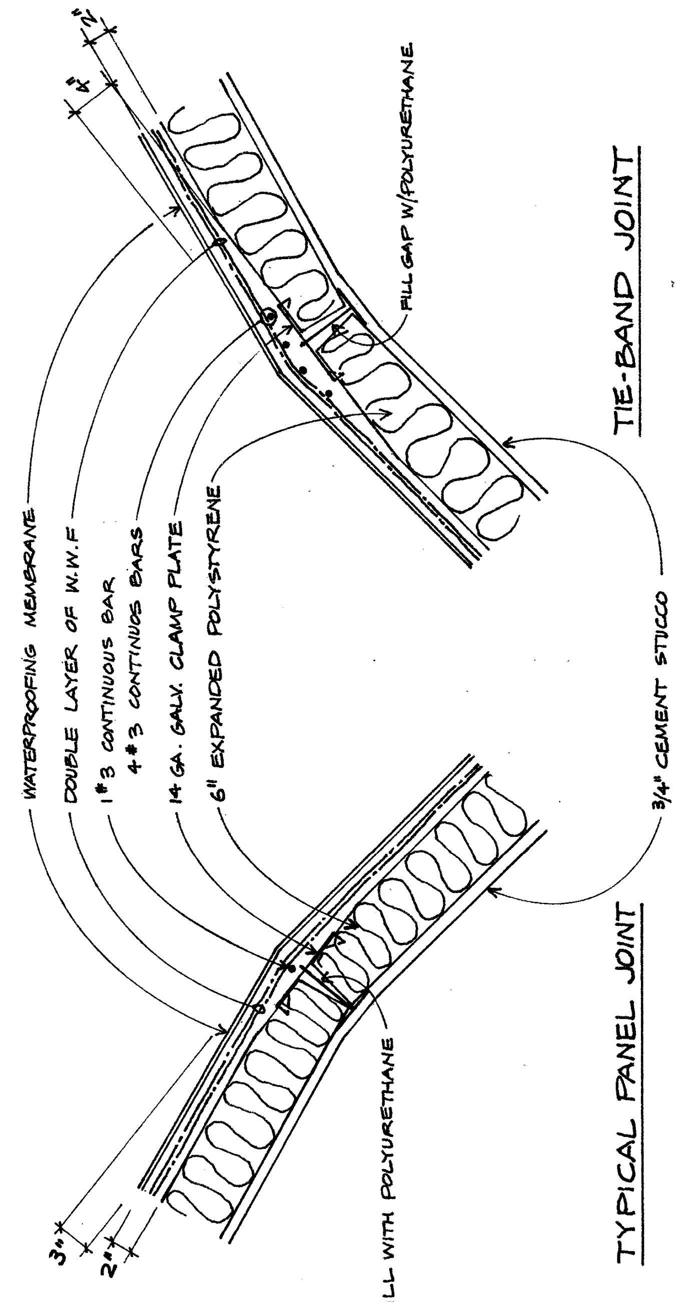

TIE-BAND JOINT
TYPICAL PANEL JOINT
WATERPROOFING MEMBRANE
DOUBLE LAYER OF W.W.F
1 #3 CONTINUOUS BAR
4 #3 CONTINUOUS BARS
14 GA. GALV. CLAMP PLATE
6" EXPANDED POLYSTYRENE
3/4" CEMENT STUCCO
FILL GAP W/POLYURETHANE
FILL WITH POLYURETHANE
4"
2"
3"
2"

FIGURE 7.6

Figure 7.7 was borrowed from another project which showed a close resemblance of what the owner wanted. Since he did not wish to have any wood because of its combustibility, we used galvanized metal studs fastened to the panel points by means of anchor bolts as shown. By using such an assembly the thickness grew from 8¾ to 11 inches, a net increase of 2¼ inches. For a surface area approximately of 2,500 square feet, that represents a loss of some 450 cubic feet.

Let us see now what happened to the R-Value:

Outside air film:	0.17	
Waterproof membrane:	0.20	
2″ concrete shell:	0.48	
7″ extruded polystyrene:	35.00	
1½″ trapped air:	0.50	
Wallboard paper backing:	0.05	
½″ gypsum board:	0.45	
Inside air film:	0.68	Total R-Value: 37.53

Again, to compensate for the loss of insulation due to cuttings and shavings, we rated the assembly as R-36.

Considering the geographical location where the dome was going to be erected, we deemed the structure as "over-insulated". We mentioned before the drawbacks of an excessive R-Value, in this case in addition to the increased materials cost, we also had a loss of 450 cubic feet of otherwise usable space. A third problem was discovered later on after completion: accumulation of warm and humid air at and/or near the apex of the dome created a problem of mold development. It was necessary to install ceiling fans with adjustable blades to force such volume of exhausted air down to mix it with the circulating flow.

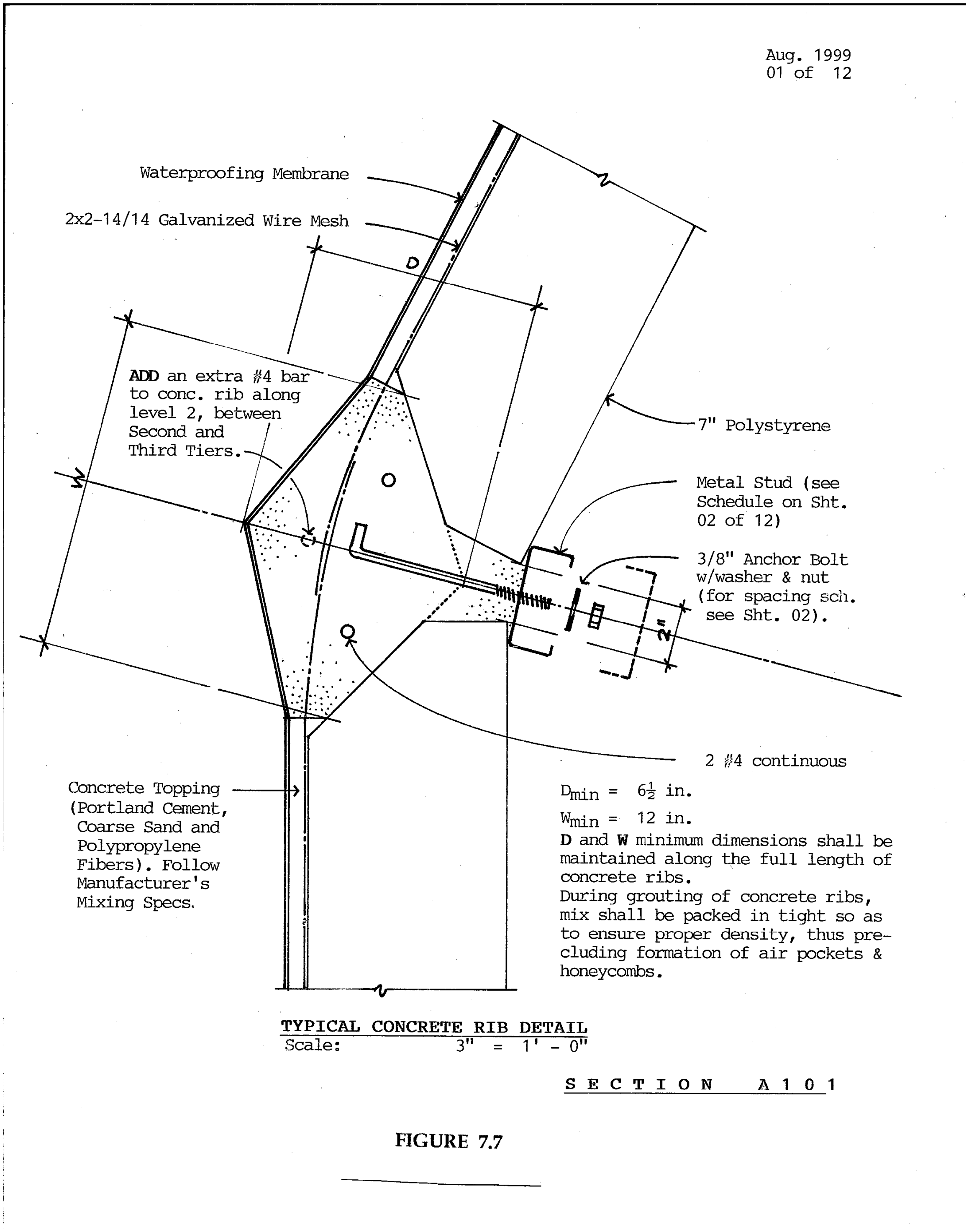


FIGURE 7.7

Finally, Figure 7.8 shows the crown detail which was used to lock up the top of the dome and to guarantee the integrity of the structure.

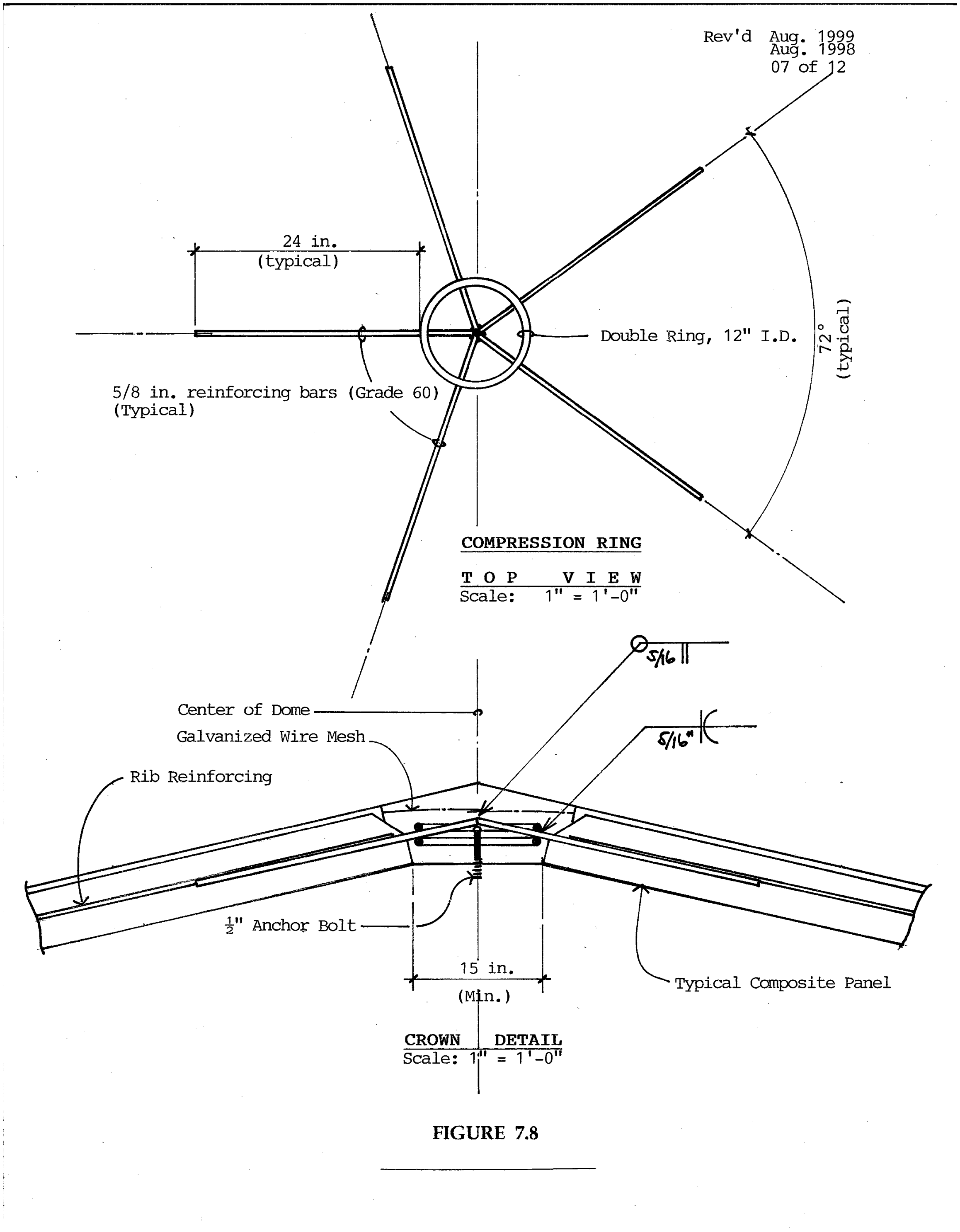

Rev'd Aug. 1999
Aug. 1998
07 of 12
24 in.
(typical)
Double Ring, 12" I.D.
72°
(typical)
5/8 in. reinforcing bars (Grade 60)
(Typical)
COMPRESSION RING
T O P V I E W
Scale: 1" = 1'-0"
5/16
5/16"
Center of Dome
Galvanized Wire Mesh
Rib Reinforcing
½" Anchor Bolt
15 in.
(Min.)
Typical Composite Panel
CROWN DETAIL
Scale: 1" = 1'-0"

FIGURE 7.8

8.0 CONCLUSION

Man felt very proud and accomplished having invented the right angle as well as the cube, the parallelogram and the polyhedron. Comfortably he moved out of his round hut into a more “advanced” squared dwelling with its sharp edges and corners. By doing so, he moved away from his given wisdom. However, nature does not produce such things, it rather employs round surfaces made out of very thin walls, with the least amount of material, and yet able to resist much more effectively the destructive action of its own events as they daily take place on the surface of this planet.

While doing so, man forgot the lessons of the egg shape and its many advantages while keeping in mind its shortcomings. However, it is about time we remember to consider that the dome is one of the most viable and effective alternatives to certain enormous problems rapidly closing in on us such as energy shortage, as well as the destructive effects of bomb blasts, hurricanes, tornadoes, earthquakes, snow and rain.

If a man’s home has been traditionally considered as his “castle”, a very poor example of a castle would be a house made out of wood, plastics and paper as we have come to end up with in our search for the so called “competitive” home. A colleague of ours in defending this idea of the “modern” home which could not even live to match its mortgage term, still says that “long lasting homes are bad for the economy”. In spite of that conception and excuse, and on the other side of the coin, we have the option of the dome as the house which could be made to last for generations to come.