The Steel Framed Dome

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MANY ENGINEERS when first confronted with the design of a steel framed dome are dismayed by the complexity of these often highly indeterminate, three dimensional space frames. Furthermore, on locating references in the literature, many are further confused to find that there is no unanimity of opinion among the various investigators as to proper methods of analysis and design. Despite the fact that domes are one of the oldest structural forms in existence and that recent years have seen a revival of this form and a growth of its popularity, few scientific investigations have been made. Opinions vary widely from those who advocate rigorous methods of analysis based on deformations or simultaneous displacements to those who advocate analysis by simple statics.

Many steel framed domes have been designed and built in recent years. However, despite the increasing popularity of this type of structure, little new design information has been published. For this reason, engineers who wish to investigate the steel framed dome for the first time can find little guidance based on the practical experience of others.

The purpose of this paper is to provide a practical discussion of the analysis and design of the Schwedler dome, one of the more popular types of steel framed domes. Included are a design example and a list of references. The opinions expressed herein are those of the authors, based on both field and office experience over the past fifteen years and upon a study and evaluation of those publications listed in the bibliography at the end of this paper.

GENERAL

The circle will enclose more space with less perimeter than will any other geometric form. The circular arch rib has long been recognized as one of the most efficient and economical methods for achieving long spans in structures. It follows that when such an arch rib is revolved about its vertical axis, the framing of the spherical surface generated thereby should be economical of materials. The resulting circular floor plan, generated in the horizontal plane, is also economical of space. These two factors tend to make the domical-roofed, circular building efficient. There are other factors. A tension ring can be used around the periphery to resist the horizontal arch thrust, leaving vertical loads alone to be transferred to the foundations. Intermediate circumferential rings can be added which reduce the unbraced length of the ribs and provide a means of uniformly distributing the live and dead loads. The upper rings are in compression and their shorter length results in lower slenderness ratios, hence higher allowable stresses. The lower ring usually carries the largest forces in the structure, taken in tension with no reduction in allowable stresses because of buckling considerations.

The resulting building encloses a large column free area. It is especially popular for public assembly buildings for several different reasons:

(1) It is economical.
(2) The circle places the greatest number of seats close to the stage or the arena floor.
(3) There are no columns to interfere with vision.
(4) Radial aisles permit maximum efficiency of ingress and egress.
(5) Minimum circulation through the building to the various seating sections is necessary.

In addition, the arch and the circle are among the most pleasing of architectural forms. Far from being an aesthetic liability, the dome is usually the salient architectural feature of the building. It has an inherent dignity of character and stateliness when viewed both from within and without. Solid rib domes present an especially neat appearance from underneath and the structural members are often left exposed to enhance the appearance and to serve as acoustical baffles. To insure architectural and structural compatability, as well as economy, the structural design must be an integral part of the project from the formative stages.

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HISTORICAL

The early history of dome structures has been well documented and will not be dealt with extensively in this discussion. Use of the dome as an architectural feature dates back at least two thousand years. Earlier applications were largely for religious structures and included such famous buildings as the Church of Santa Sophia, built in Constantinople in 537, St. Mark's Cathedral in Venice, St. Paul's in London, and hundreds of other equally famous churches. The dome was also used extensively by the Moslems who modified it into the bulbar or onion shape which has become synonomous with the mosques of Islam.

For hundreds of years domes remained the symbol of religious architecture and their construction was the exclusive province of the stone or brick mason. Both concepts were changed in 1865 with the construction of the cast-iron dome for the United States Capitol in Washington. Many state capitols built thereafter featured a similar central dome and the use of a dome came to be associated with governmental buildings.

Practically all the earlier domes were ornamental. Their functional potential was largely ignored until comparatively recent years. In the steel industry, tank and plate fabricators were the first to take advantage of the economic potential inherent in the geometry of such forms. Early in the twentieth century, thin plate domical and conical roofs began to appear over water and oil storage tanks. A column free span of 200 ft is not considered unusual today for such structures, and this can be achieved with a plate thickness of only 5/8 in.

It has been during the years following World War II that the full structural potential of the dome has been realized. Structural efficiency has now become a full partner with majestic architectural beauty. The dome continues to be used in a variety of applications and with many variations of its basic geometry and construction. There has been a revival of its use to span church sanctuaries and governmental buildings, and its use has also become frequent in commercial and industrial structures. The most popular use of all continues to be in public assembly buildings such as gymnasiums, field houses, auditoriums and coliseums. Many such buildings built since 1950 have been circular in shape and most of these have been roofed with a steel framed dome.

A variety of shapes have been employed—spherical, elliptical, conical and parabolic, with variations often added to break the roof lines, such as folded plates and barrel arches. A variety of construction systems have been employed—smooth shells, arch ribs, Schwedler domes, lattices, lamellas, and geodesics. All have one common denominator—economy of materials. If extensive shoring and form work can be avoided, there is yet another common denominator—economy of cost.

DESIGN—GENERAL CONSIDERATIONS

The discussion in this paper will be restricted to spherical domes of the Schwedler type, primarily because this type is the most popular. Little has been done to adapt the thin-shell steel dome to building construction, perhaps because of acoustical or insulation problems or because the very thinness of steel suggests buckling and discourages investigation. Lamella, lattice and geodesic domes were relatively unknown until the 1950's. None of these have so far achieved much popularity with consulting engineers for two reasons: first, they are highly complex, statically indeterminate space frames and their design is still dependent to some degree upon model analysis; secondly, their design and construction can only be achieved under proprietary rights. As these interesting and economical framing systems grow in popularity, it is hoped that more information will be made available to the design profession. The bibliography references contain project descriptions of several such domes constructed in recent years along with outstanding examples of Schwedler dome construction.

THE ARCH RIB AND THE SCHWEDLER DOME

As is the arch, the dome is primarily a direct thrust structure. In other words, its individual members are designed as columns or ties subject to compression or tension.* A dome may be imagined to consist of a number of horizontal rings of decreasing diameter, each one placed on top of the other. The upper rings tend to move downward and thrust outward against the adjacent lower rings. The latter have a tendency to expand while the former tend to contract. The steel framed dome substitutes structural steel members for the principal stress trajectories of the concentric rings and keeps these separated by structural steel ribs.

Fig. 1a shows a schematic diagram for an arch rib dome and Fig. 1b shows a section cut through this dome on a diametrical plane. The individual ribs are usually designed as three-hinged arches subjected to triangular loading and are supported at the top by a compression or lantern ring, and at the base by either the foundation, or, as in the case shown, a tension ring which may be supported by vertical columns. This structure is statically determinate and its solution under loads which have polar symmetry can be handled quite easily through the resolution of forces. Under an asymmetric loading

* *The authors emphasize that due attention must also be given in design to all moments, both primary and secondary, which are introduced into the structure. At the same time it is suggested that every attempt be made to reduce such moments as far as possible in the primary members (ribs and rings) by the arrangement of purlins and other secondary members and by considering an area increase of the rings in some cases to reduce the secondary effect of rib deflections due to ring elongation or shortening.*

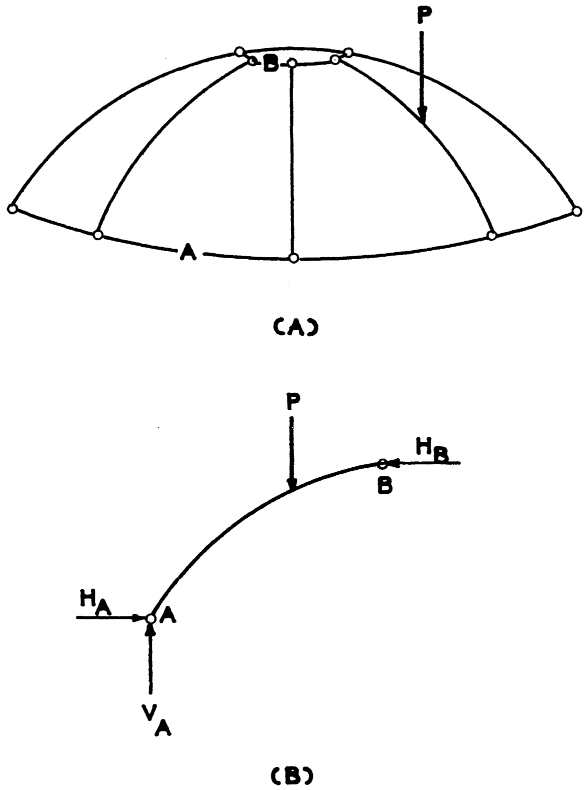


Figure 1

condition such as shown in Fig. 1a, the foot ring poses a special problem; it can even become unstable under certain conditions. Solution of the foot ring problem, however, can still be accomplished through the equations of simple statics.*

The arrangement shown in Fig. 1a, whereby the ribs are stopped at the compression ring instead of being continuous over the pole, is used in practically all dome designs. Three distinct advantages result. First, the pole joint where a great number of ribs converge, and which is consequently difficult to fabricate, is eliminated. Secondly, a large opening which can be used for light and ventilation is created. Thirdly, the structure becomes statically determinate for all load conditions, whether symmetrical or unsymmetrical.

The arch dome is still popular, primarily because of its simplicity of design and because the strong-limbed, uncluttered effect it produces has aesthetic appeal. However, full economic potential of the spherical shape is not realized in this design. It is completely devoid of

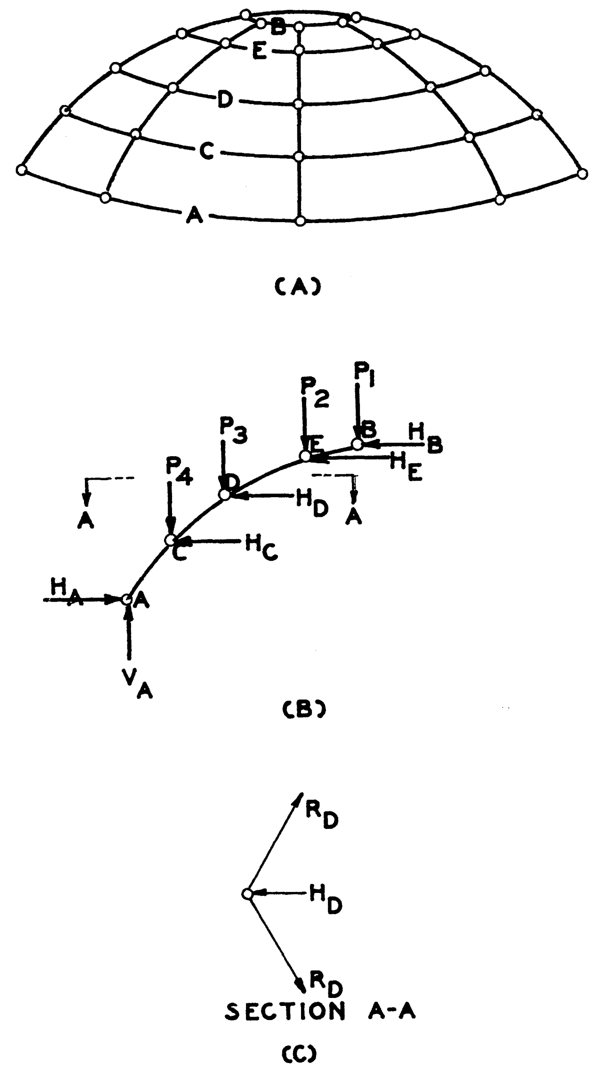


Figure 2

internal circumferential force members which, when used, permit an excellent means of uniformly distributing the frame dead loads. Such members also serve to reduce the unbraced length of the arch ribs.

Fig. 2a is the same as Fig. 1a, except that three intermediate rings have been added to the structure. Fig. 2b shows a rib under symmetrical load. The same considerations discussed for the arch dome of Figs. 1a and 1b are true for this dome with the added consideration that horizontal interaction between the ribs and rings at points **C**, **D** and **E** must be taken into account. The conformation of the rib forms an equilibrium polygon for the vertical loads P and the horizontal loads H, and consequently the stresses can be found by the equations of statics. Under symmetrical load, stresses in segments of the rib such as **DE** are identical with the stresses in

* *A complete discussion of dome design under asymmetric loading and the foot ring problem is considered beyond the scope of this paper. The reader is referred to references 1, 2 and 5 which contain discussions and design examples with regard to this problem. As pointed out later in this paper, the authors have some reservations as to the validity of static analyses under asymmetric loads.*

the corresponding segment of the other ribs in the structure. The stresses can be found in the ring members by further resolution of forces and symmetry as indicated by Fig. 2c. Prof. F. Dischinger[15] and W. Schwedler[3] have derived general formulas for domes based on the simple resolution of forces which give a direct solution to the problem. The method used by the authors is practically identical with these except the designer has a means of visualizing the structure at all times. This method will be described in detail later.

MODIFICATIONS

In addition to the modification usually made at the pole, which was discussed previously, certain other modifications are often made in actual practice. First of all, auxiliary members such as purlins are added, and frequently diagonal bracing is used between the ribs and rings in the plane of the roof surface. For many years it was held that such diagonal bracing was essential for the stability of the structure under asymmetrical loading. Theoretically, this is true, but research[9] has shown that such bracing serves no useful purpose in the completed structure. Most designers prefer to retain it, at least in alternate bays, as an aid in erection.

Another simplification usually made in actual practice is in connection with the rib members. Here again, this can be considered a concession to the economics of erection. Insofar as design is concerned, direct stresses are usually computed in accordance with the assumption that the juncture between the rib segments and ring elements is articulated as shown in Figs. 2a and 2b. Although many domes have been constructed in this manner, usual practice is to make the rib continuous in order to eliminate all construction falsework except a central erection tower. A modification of this type naturally requires that any primary moments at supports (rings) and secondary moments due to changes in ring diameter be considered along with the principal stresses.

Finally, the geometry of the dome itself is usually altered so that only the panel points lie on the true spherical surface. All members between these points are made straight, resulting in considerable fabrication savings. If a sufficient number of ribs and rings are used, the fact that the structure is now polygonal instead of spherical usually escapes the casual observer.

With the foregoing modifications, the Schwedler dome takes on the appearance of the umbrella shaped structure shown in Fig. 3a. Possible load conditions and the actual design of such a structure will now be considered.

LOADING

In practice, most domes are designed for polarly symmetrical load conditions only. Even if the dome is to be subjected to unsymmetrical loads, the recommended procedure is to evaluate this load in combination with

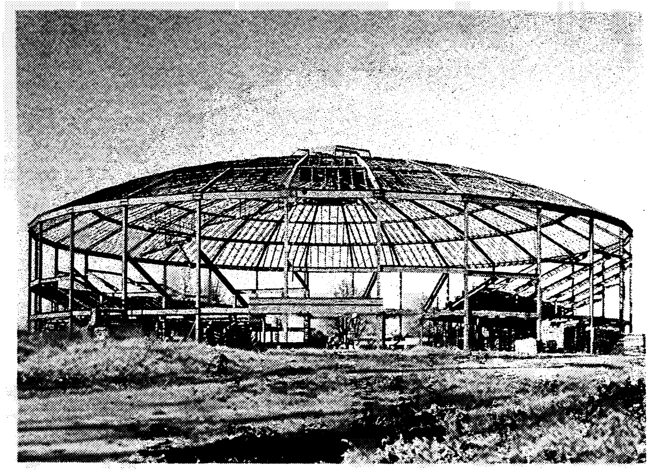


Figure 3

such additional loads as would be needed for polar symmetry.

The most important unsymmetrical loading to which a dome is subjected is wind pressure. Timoshenko[6] has suggested a distribution of pressure on the windward side and of equal suction on the leeward side based upon a sine law. The intensity of wind pressure is assumed to act normal to the surface of the dome and is expressed in a mathematically convenient manner. This method is said to be used widely in European practice.

Other authorities feel that the variation in wind pressure on a surface having curvature in two directions is a highly conjectural matter. Further, it should be recognized that little is known at the present time with regard to the modifying effects of rib continuity and connection restraint, particularly under unsymmetrical loading. Such being the case, these authorities feel that exact design refinements are not indicated and recommend that the structure be designed to resist an equivalent vertical load. This is usually a conservative procedure and the authors' observation is that it is the method most frequently used in design offices. In the southeastern United States, from 10 to 15 psf is usually added to the vertical live loading to compensate for wind effects.

The structure shown in Fig. 3 is shown in isometric form in Figs. 4a through 4c in order that distribution of live load can be considered. The shaded portion indicates live load distribution, and for Fig. 4a, the live load covers the entire roof. This is the maximum stress condition for all ribs, for tension ring **A**, and for compression ring **B**. The following discussion of design stresses refers only to the intermediate rings **C**, **D** and **E**. These may be either in tension or compression, depending on the rise-to-span ratio of the roof, the live load and the location or distribution of the live load. Any given intermediate ring is in maximum tension when all of the dome above the ring is fully loaded, and in maximum compression when all of the dome below the ring and the

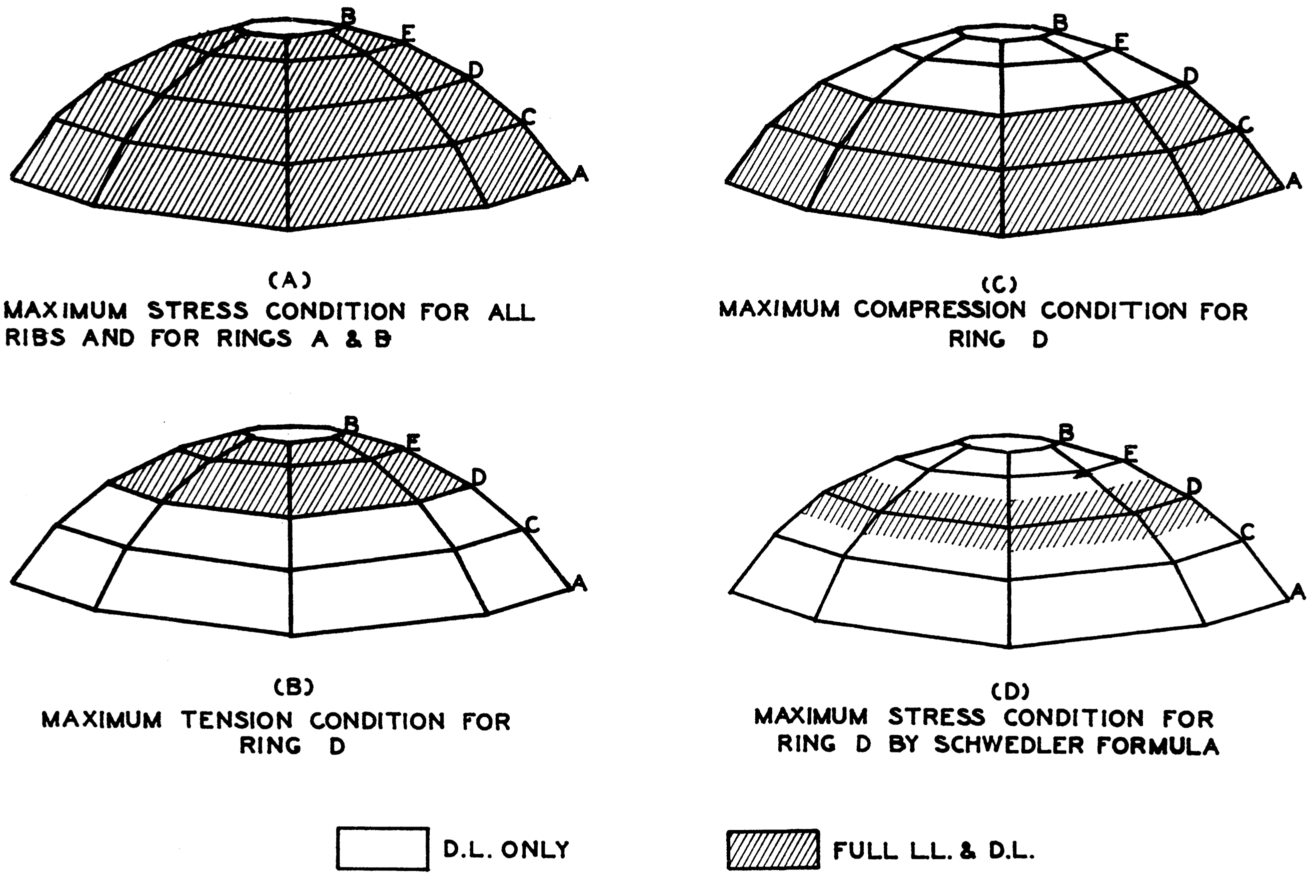


Figure 4

ring itself is fully loaded. Fig. 4b shows the maximum tension condition for ring **D**; Fig. 4c shows the maximum compression condition for the same ring. The design stress which should be used seems to be a point of disagreement between designers. Many simply assume the condition shown in Fig. 4a and use the net difference in tension caused by the live and dead load above the ring and compression caused by full live and dead load below the ring.

Others employ the system based on assumptions used in the derivation of the Schwedler formula. For maximum compression, the ring under consideration is loaded with full live and dead load; dead load only exists on the balance of the structure. This load condition is illustrated in Fig. 4d.

The authors prefer a more conservative approach. Maximum tension is computed based on full live load and dead load above the ring and dead load only below the ring. Maximum compression is computed based on dead load only above the ring and full live and dead load below the ring. These are the same conditions depicted in Figs. 4b and 4c. The more critical of these stress values is used in design.

It is recognized that this rather stringent load condition would seldom be met under actual conditions although it could be approached by a band of snow or ice building up on the structure. There are several reasons for advocating such a design procedure:

(1) Should premature failure occur in the bottom tension ring because of faulty workmanship, the next intermediate ring would tend to take over its function. Added strength in this ring would probably allow it to do so, at least temporarily. Otherwise, there would be an unbuttoning effect as each successive ring picked up the load and elongated excessively or failed in tension.

(2) The secondary moments in continuous ribs caused by the deformations of the ribs and rings will often be large. Such moments can be minimized by the larger areas and lower stresses (under uniform live load) characteristic of rings designed by this method.

(3) The amount of increased ring material resulting from use of this method is usually modest.

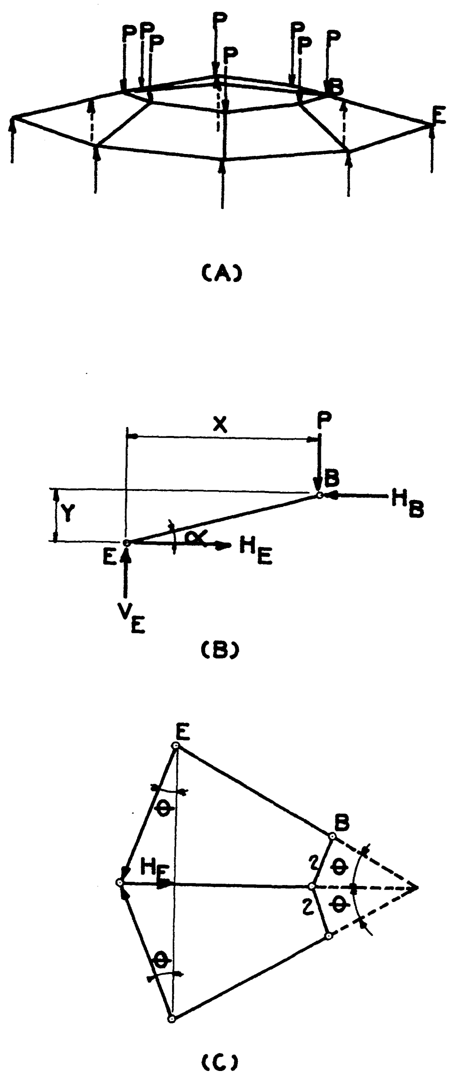


Figure 5

DESIGN

The structure shown in Figs. 3 and 4 can be considered to be made up of four simple structures, frustrums of cones, stacked one above the other and with successively decreasing slopes. In design, it is necessary to consider only one such conical element at a time. Fig. 5a shows the upper cone of the dome consisting of the upper compression ring **B**, rib **EB**, and intermediate ring **E**. Collar loads, P, act vertically at the intersection of rib **EB** and ring **B**. For maximum compressive stress in both rib and ring, P should be the sum of all live and dead loads acting on ring **B**. Fig. 5b is a free body diagram of rib **EB**. The horizontal forces, H_B and H_E must be equal and opposite and are the interaction between the rings and the ribs, and represent compression in ring **B** and tension in ring **E**. The rib is held in vertical equilibrium by the elements of the cone below.

Taking moments about point **E**, H_B equals $P \cot \alpha$. Considering equilibrium at joint **B**, the compressive force in rib **EB** must equal $H/\cos \alpha$ or $P/\sin \alpha$. Referring to Fig. 5c, by symmetry, each ring segment must take half of the computed H force. It may be seen from the figure then that the ring force must equal $H/2 \sin \theta$ where 2θ is the angle between the horizontal projection of ribs **EB**. Written in another manner, ring force equals $P \cot \alpha/2 \sin \theta$.

Since $H_B = H_E$ in the above example, the ring forces would be equal and opposite in rings **B** and **E**. However, this would be true only for a single isolated conical element. To obtain a proper solution, the designer must also consider the cone immediately below, that which consists of top ring **E**, bottom ring **D**, with ribs **DE**. The forces thus obtained for ring **E** must then be added algebraically to the force obtained by the solution of the upper cone. The sample calculations shown in Fig. 6 illustrate this procedure under "ring stresses".

DISCUSSION OF EXACT METHODS VERSUS STATICS

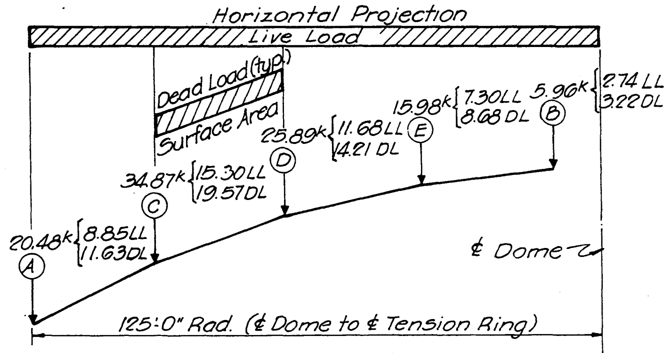
The authors have indicated a preference for the use of statics rather than other so-called "exact methods" in the foregoing discussion. This preference is based upon experience and the results of tests.[7,9] Under a symmetrical load condition there seems to be an excellent agreement between observed stresses and stresses computed by the method of "least work" used by the investigators. However, the same is true of stresses computed by statics, and there seems to be no justification for the more involved, indeterminate approach.

Under unsymmetrical loads, the reverse is true. The method of statics described above not only gives extremely wide variations between computed and observed stresses; it presents a completely erroneous stress pattern, and in the authors' opinion this method is invalid for stress analysis under unsymmetrical loading.

Stresses obtained by the method of "least work" show a better correlation with the observed stresses for unsymmetrical loading. However, such wide variations still exist that this method can also be classed as an ineffective tool for the analysis of the Schwedler dome.

It is the opinion of investigators that the extreme variation between observed and actual stresses for unsymmetrical loading is caused by the resistance of the joints. While the validity of the usual assumption of frictionless joints or pin-ended members has been consistently proven for two-dimensional structures, apparently this is not true for three-dimensional structures, such as domes. For such an assumption to hold, each joint should be free to turn in any direction similar to a ball and socket or universal joint.

For symmetrical loading, the uniform deflections and



RIB LOADING

RIB DESIGN (Dome Loaded Full LL + DL)

Segment A-C:

$\Sigma F_V = 5.96 + 15.98 + 25.89 + 34.87 = 82.70^k$

Axial force $P = \frac{\Sigma F_V}{\sin\alpha_4} = \frac{82.70}{0.449} = 184^k$

Moment $= 31.9^{\prime k}$ (Same as purlins A-C)

Use ASTM A36 Steel

Try 14WF 78: $r_y = 3.0$; $r_x = 6.09$; $A = 22.94$ in.^{2}; $S = 121.1$ in^3

$\frac{\ell}{r} = \frac{28.95(12)}{3.00} = 116$; $F_a = 10.85$ ksi; $F_b = 24$ ksi

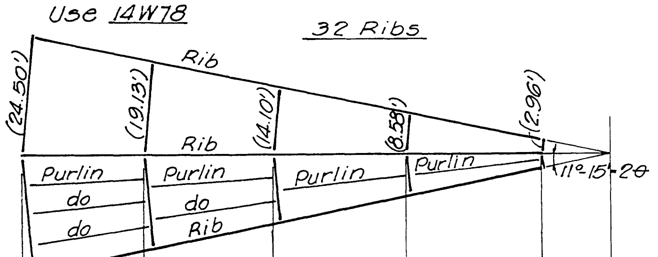
$f_a = \frac{184}{22.94} = 8.02^k/\square''$; $f_b = \frac{31.9(12)}{121.10} = 3.16^k/\square''$

$K = 1.0$; $C_m = 0.93$

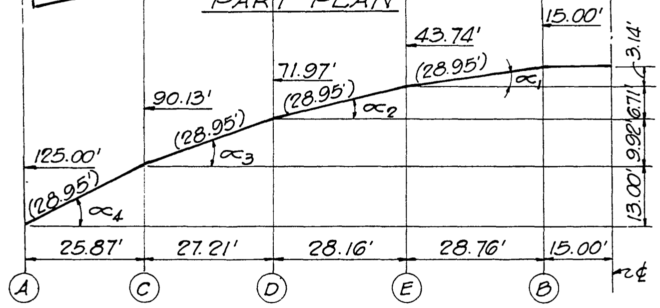
$\frac{KL}{r_b} = \frac{28.95(12)}{6.09} = 57.2$ $\quad F'_e = 44.33$ ksi

Formula (7a): $\frac{8.02}{10.85} + \frac{0.93(3.16)}{\left(1-\frac{8.02}{44.33}\right)24} = 0.739 + 0.150 = \underline{0.889}$ OK

Use 14W78



PART PLAN



ELEVATION OF TYPICAL RIB

Rise to Span Ratio = 0.133

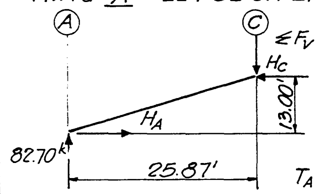
DESIGN LOADS:

Assumed L.L.	= 30 psf *	
Dead Loads		
Roof Deck	= 10 psf	
Bulb Tees	= 1 "	
Roofing	= 7 "	
Steel	= 15 "	
Elec. & Mech.	= 2 "	
	35 psf	

* Includes 10psf equivalent vertical Load for Wind.

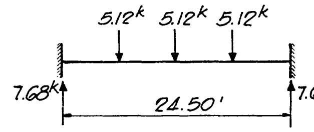
RING DESIGN

RING A: LL + DL on Entire Dome



$H_A = P \cot\alpha_4 = 82.70\left(\frac{25.87}{13.0}\right) = 165^k$

$T_A = \frac{H_A}{2\sin\theta} = \frac{165}{2(0.099)} = 834^k$ Tension



M (At Support) $= 37.1^{\prime k}$

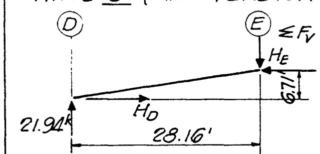
Try 14WF142 $A = 41.85$ in.^{2} $S = 226.7$ in^3

$\frac{T_A}{A} = \frac{834}{41.85} = 19.80$

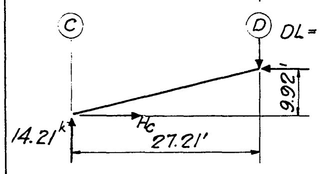
$\frac{M}{S} = \frac{37.1(12)}{226.7} = \frac{1.97}{21.77^k/\text{in}^2}$ OK

Use 14WF142

RING D: (MAX. TENSION CONDITION)



$H_D = 21.94\left(\frac{28.16}{6.71}\right) = 92.0^k \rightarrow$

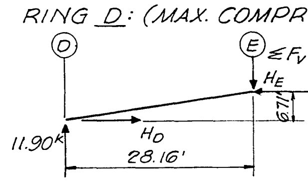


$H_D = 14.21\left(\frac{27.21}{9.92}\right) = 39.1^k \leftarrow$

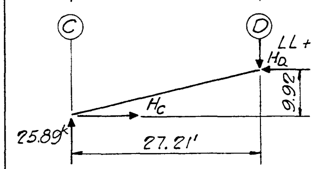
$\Sigma H_D = \overrightarrow{92.0} - \overleftarrow{39.1} = \overrightarrow{52.9^k}$

$T_D = \frac{52.9}{2(0.099)} = 268^k$ Tension

RING D: (MAX. COMPRESSION CONDITION)



$H_D = 11.90\left(\frac{28.16}{6.71}\right) = 50.0^k \rightarrow$



$H_D = \frac{25.89(27.21)}{9.92} = 71.0^k \leftarrow$

$\Sigma H_D\ \overrightarrow{50.0} - \overleftarrow{71.0} = -\overleftarrow{21.0}$

$C_D = \frac{-21.0}{2(0.099)} = -106^k$ Compression

Ring D must be designed for 268^k Tension and/or 106^k Compression + Bending, whichever produces the most severe condition.

Note:

1. Ribs to be welded at each connection as required for fully continuous design.
2. Rings are also designed with fully continuous joint development.
3. Other members, such as purlins, bracing, cat walks, etc., may be connected with bolts.
4. The use of non-destructive weld testing should be carefully studied for proper and logical application to the structure; the authors feel such methods need be considered for Tension Rings only; visual inspection otherwise.

Fig. 6. Sample calculations

displacements seem to explain the more reliable results obtained through rational analysis.

There is little doubt that additional investigation is needed in the case of domes under unsymmetrical loads which would enable designers to consider the effect of resistance offered by various types of joints and to obtain a true picture of stresses. Fortunately the proper selection of a substitute symmetrical vertical load, as previously discussed, provides a method whereby economical and safe designs may be produced. The authors agree with Professor Paul Anderson[9]—" . . . in the meantime we should go right ahead building more steel-framed domes."

Most designers, fabricators and erectors who have had experience with the Schwedler dome are convinced that it has a reserve of strength far greater than calculations indicate. Some attribute this to the resistance of the hundreds of connections which is usually ignored in design. Others feel it is due to the dome slightly changing its geometry to meet new load situations. As one point deflects more than its neighboring points, stresses are developed to transmit load away from the loaded point. Many designers feel that the membrane or shell action of the roof deck adds considerable strength. Others feel that the plastic strength is significant (as a higher stress is reached in the lower members they refuse to accept further load and hence transfer it to the less-stressed upper members). Perhaps this reserve of strength is due to a combination of all the foregoing.

DISCUSSION OF DOME ECONOMICS

It has been stated previously that the engineer should participate in the design of a building incorporating a dome from the beginning of the project. It is the planning stage which has more effect on dome economics than any other. Every dome structure has a point of optimum economy which depends primarily upon the rise-to-span ratio selected and the number of ribs and rings to be employed. For maximum economy the total roof surface area and the unit cost of the roof material must be taken into account along with the tonnage of structural steel and its unit cost. In other words, the lower the rise-to-span ratio, the smaller the roof surface, but the steel tonnage increases because of the higher thrusts associated with the flatter roof.

The authors believe that maximum structural economy is approached when the spherical radius is made approximately equal to the diameter of the dome. This results in a rise-to-span ratio of 0.13. The geometrics are also simplified since the subtended angle is exactly 60 degrees. A study of many domes built in this country since 1953 reveals that this relationship of $R = D$ is closely adhered to. The rise-to-span ratio of 25 domes surveyed varied from 0.12 to 0.18 with most falling around 0.14. Live and dead loading for these domes was

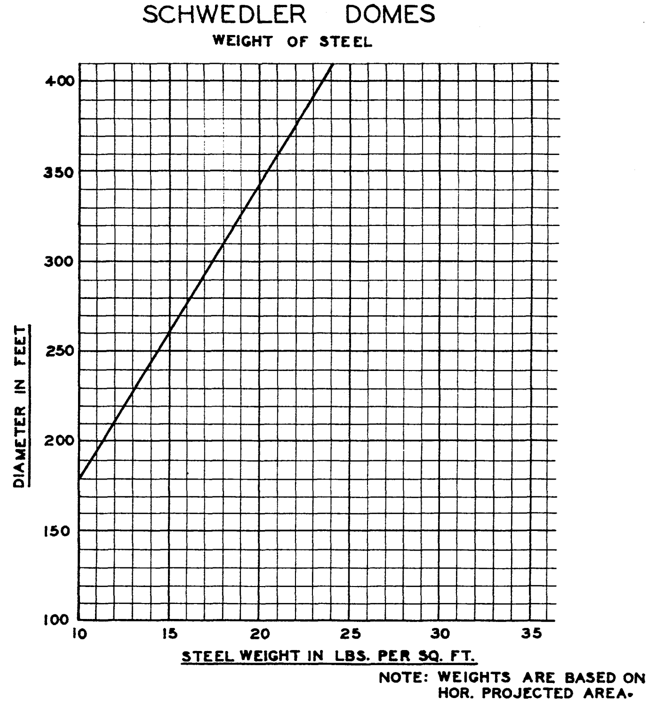


Figure 7

fairly constant which permitted the preparation of a chart of steel weight versus span which has proven to be sufficiently accurate for use in estimating the structural steel weight for proposed Schwedler domes. This chart is shown in Fig. 7. Since it is based on domes designed prior to the advent of A36 steel and the new AISC Specification, steel weights shown thereon are slightly on the high side.

CONCLUSION

It is hoped that the future will see additional research in this important field, not only for the Schwedler domes discussed, but also for the other types mentioned earlier in this paper. Work is especially needed for nonsymmetric loading and investigation into the modifying effects of continuity of ribs and resistance of connections. Weld shrinkage seems significant also in some cases. The authors have attempted to present practical design information for the Schwedler dome based upon their own experience and study. It is hoped that this paper will encourage others who have had experience in the design of these structures to offer their comments.

The bibliography which follows lists those references used by the authors in their work with steel framed domes and in the preparation of this paper. It is not intended to be a complete listing of available literature on the subject of domes, but should prove helpful to designers who seek reference sources in this field.

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