Space Frame Structures

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INTEREST ON THE PART of architects and engineers in the so-called space frame has prompted the preparation of this paper. Its purpose is to present some of the factors to be considered for designing structures of this type.

Among these factors are descriptions of various types of space frames, the effect of support locatiosn, the selection and arrangement of chord spacing, web systems, joint details, the collection of loads at support, and the methods available for analysis and design.

TYPES OF SPACE FRAMES

Basically, the most rigid and efficient structure is a lattice network wherein the optimum spatial distribution of material is achieved.

Space frames which fall into this categorization broadly include: transmission towers, radar antennae, radio telescope reflectors, geodesic domes, cable suspended roof structures, and certain other dome type structures.

All of these seek to dispose material spatially for maximum flexural capacity and rigidity.

Also, each of these structural types depends upon a disciplined repetition of detail and geometric modularity to satisfy design and economic considerations.

The transmission tower, while having two axes of symmetry, generally produces only four identical members due to symmetry for any location and achieves repetition and modularity because many of these structures are required for any given project.

Radar antennae and radio telescope reflectors generally have a polar and annular symmetry which satisfy the criteria of repetition and modularity.

Geodesic domes achieve perhaps the highest order of detail repetition and modularity of any of the aforementioned structural types. Their economy of both material and erection man-hours is clearly to be envied, and indeed establishes a goal which is difficult to achieve with other structures.

Cable suspended roof structures are perhaps a close second to the geodesic dome in economy of material if one neglects the boundary conditions for a circular or rectilinear plan. The massive compression and tension rings or horizontal edge members tend to rapidly deteriorate the economics found using high strength steel in pure tension.

The horizontal space frame which is to be discussed here is a lattice type structure which satisfies the need of most building roof structures and does not subject the architect to acceptance of a dominant design feature which may be incompatible with his architectural expression (see Figs. 1, 2, and 3).

SUPPORT LOCATIONS

The location of supports is an important factor in determining the degree of optimization for this type of structure. The plan projection of moment contours resulting from vertical loads (Fig. 4) can be said to be equivalent to that of a flat plate. When cantilevers are provided by a proper support location, negative moments are produced which reduce the positive moment, etc. It should be appreciated that for a given span of a truss system with cantilevers the theoretical material requirements are less than for a simple span. The location of supports which produce cantilevers of approximately 0.3 of the clear span will result in a structure that has less deflection, less material and the best distribution of chord material. Cantilevers have little effect on the size of web members.

The number and location of supports used is principally a function of the plan requirements of the structure. The desired support locations are those which produce symmetry about two or three axes (see Figs. 5 and 6). It is recommended that supports be located so as to produce negative moments which are equal to the positive moment at mid-span as determined by utilizing an analogous flat plate with non-restraining support points.

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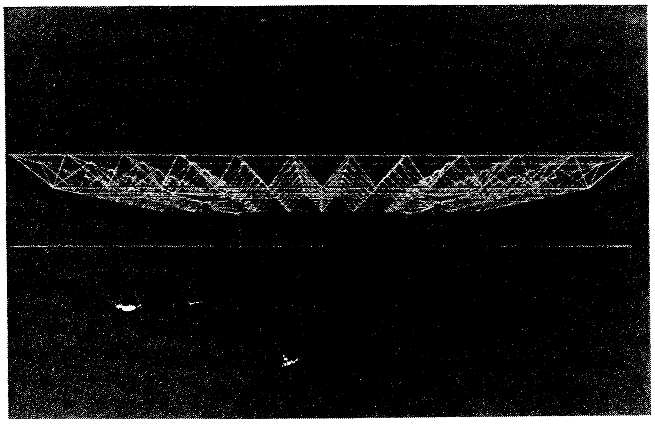

Figure 1

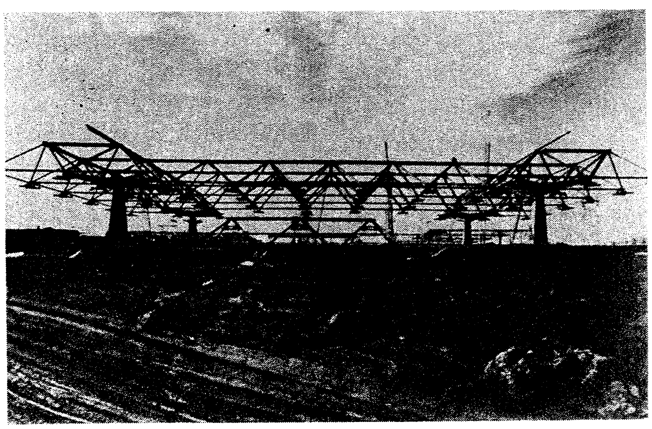

Fig. 2. Pekin Community High School Gymnasium, Pekin, Ill.

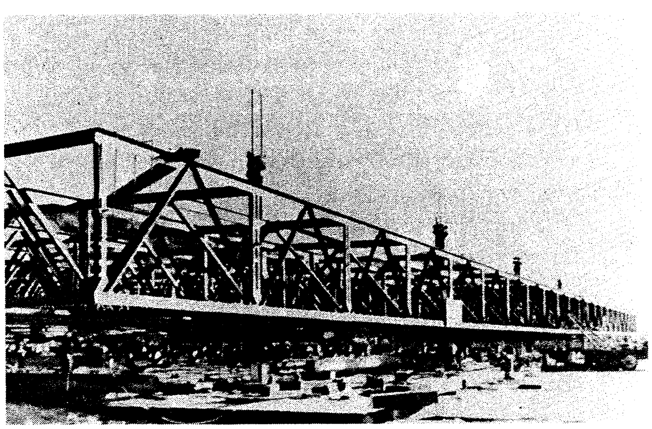

Fig. 3. Air Force Academy Dining Hall, Denver, Colo.

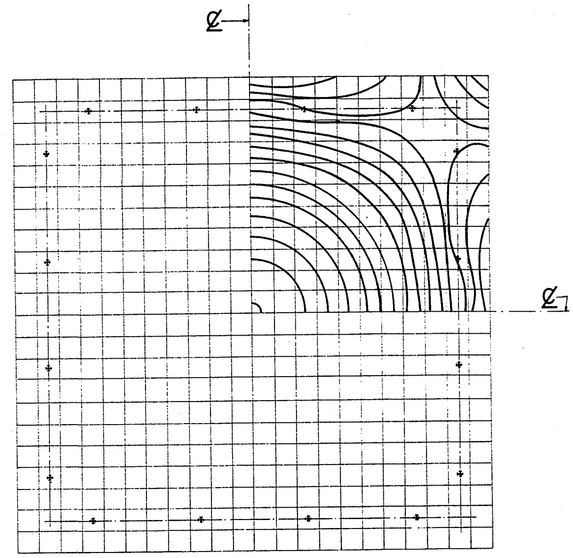

Fig. 4. Moment contours

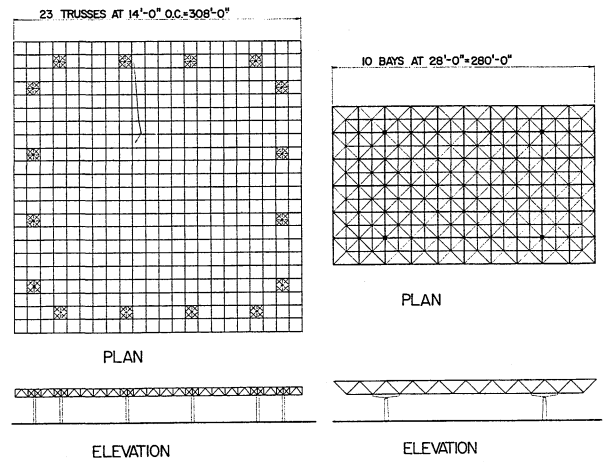

Fig. 5. Left, two-way vertical truss system. Right, three-dimensional Warren truss system

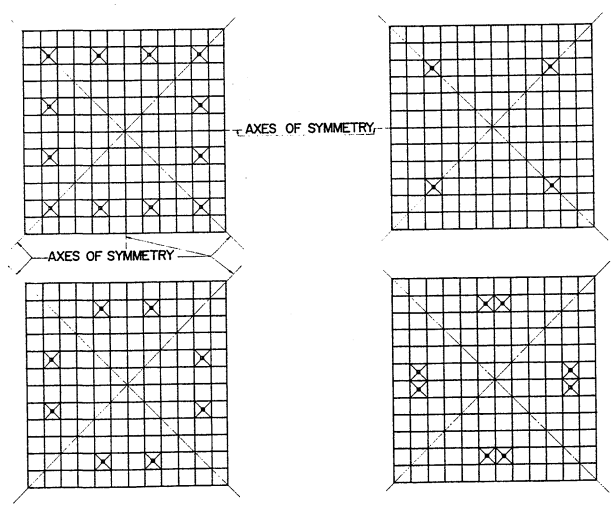

Fig. 6. Column arrangements give axial symmetry

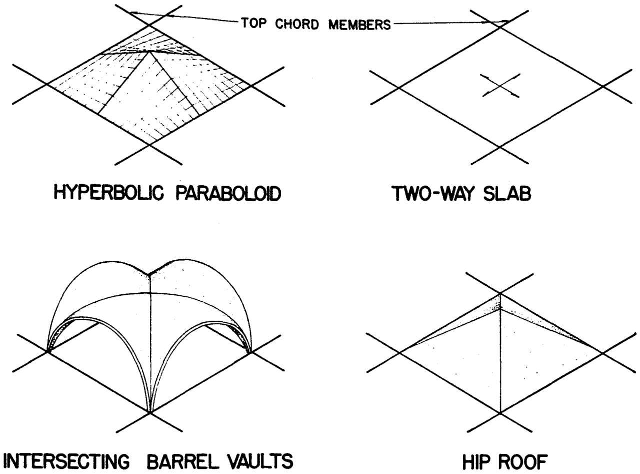

Fig. 7. Two-directional roof infill systems

SELECTION OF CHORD SPACING

Factors to be considered in establishing the chord spacing are:

1. Building planning module
2. Joint costs and the number of square feet tributary to each joint
3. Horizontal roof infill system
4. Ratio of top chord bending stress to direct stress
5. Ratio of horizontal spacing of chords to depth of truss

The building planning module, or a convenient multiple of it, is perhaps the most influential factor in the determination of the horizontal module for chords. In general these range from 3 ft-0 in. to 8 ft-0 in. Therefore resulting structural modules (or bay sizes) range from about 21 ft-0 in. to 48 ft-0 in.

Assuming a 7 ft-0 in. module and a space frame span of 280 ft, one of the more logical modules would be 28 ft-0 in. because 280 ft is divisible by 28 ft which is also divisible by 7 ft. In addition, a rough approximation of the optimum depth is $\frac{1}{20}$ of the span or 14 ft which is also divisible by 7.

The number of square feet tributary to each joint is a factor which most seriously influences the relative cost of various space frames. Generally, the larger the ratio of square feet to each joint the more economical the structure because of a greater proration of roof area to joint costs.

A good ratio is in the area of 200 to 450 sq ft per joint. These relate to space frame spans of 200 to 300 ft with chord spacings of 20 to 30 ft.

The horizontal roof infill system ideally should span in two directions to chords at its boundaries. This can be accomplished by two-way slabs, hyperbolic paraboloids, smaller order reticulated space lattices, shallow double curved arches, and pyramidal structures (see Fig. 7).

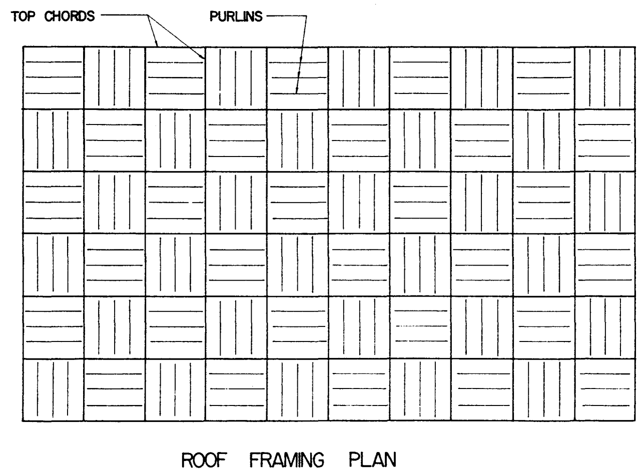

Fig. 8. "Checkerboard" panel system

While these solutions are available to provide an ideal equal distribution of load to the top chords, the same distribution can be achieved by using one-directional spanning systems. All that need be done is the "checkerboarding" of each panel span direction, as illustrated in Fig. 8. Thus, each chord is equally loaded. The benefit of this lies in the establishment of a constant bending moment for each chord which greatly simplifies chord design.

While this approach does not produce the same effect on the perimeter space frame chords, these are generally minimum members and are not affected by the resulting alternate panel loading.

The new interaction equation in the 1963 AISC Specification has basically increased the allowable capacity of compression members subject to bending moments. Thus, it becomes important to select top chord members which have a better compression than bending capacity.

The optimum ratio of bending stress to direct stress has been found to be 1 : 1 or less.

The design of compression chords requires the designer to establish a length for an analysis of the compression chords. It is the author's opinion that the joint rigidities created by the welded intersection of web and chord members is sufficient to permit the use of 70 per cent of the distance between panel points for the chord member length.

Assuming that the chords are equally spaced in both directions the author believes that optimum web lengths are achieved by adopting a three-dimensional Warren truss arrangement. This requires that bottom chords be located midway between top chords in plan. When each chord panel point is then connected to the opposite chord a pyramidal volume is developed which has 45° sides. Thus, the resulting space frame has a depth equal to one-half the horizontal chord spacing.

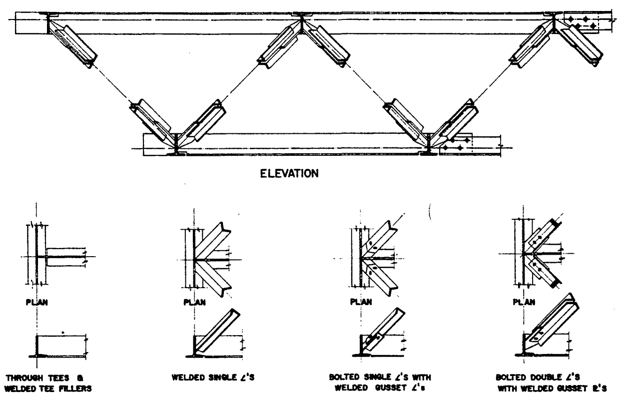

Figure 9

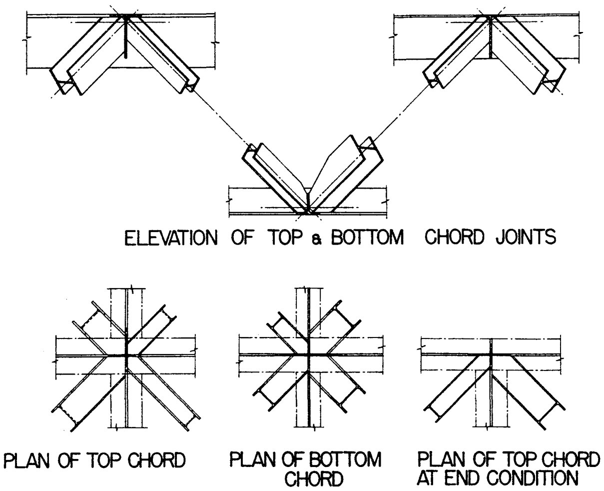

Figure 10

WEB SYSTEMS

While the "optimum" web lattice just described is preferred by the author, one should not imply that other arrangements cannot be employed. Some other web arrangements are:

1. Vertical Warren web systems in two directions
2. Vertical Pratt web systems in two directions
3. Vertical Warren web systems which run diagonally in one direction from panel point to panel point
4. Other triangulated systems

JOINT DETAILS

The most important economic consideration in the design of space frames is that of determining how the joints are to be fabricated.

There are numerous possibilities which depend on the following considerations:

1. Types of members
2. Sizes of members
3. Geometric relationships of members
4. Connection techniques such as welding, bolting or the use of special connectors
5. Desired appearance

Space frames are being built utilizing tubular members, structural tees, angles, and wide flange members (Figs. 9 and 10). Each of these has an implied connection discipline.

The sizes of members will often dictate the type of connection. For example it is extremely difficult to directly weld a 12 in. WF web member to a 6 in. structural tee chord, and in cases such as this either the member size must be revised or a special connection developed.

The geometric relationships between members requires special study of angles of intersection and accessibility for welding or other connection techniques.

Welded connections are generally preferred because of the elimination of connection material or connection devices. However, in some instances such as the use of tubular members it is often required to use a joint piece to obtain enough weld length to develop the member stress.

Bolted connections generally require joint material or joint assemblies which extend out on the members and have been found to be more expensive than welding.

The use of special connectors with tubular members has long preoccupied many designers. There are internally threaded spheres, spheres with threaded projecting shanks, special end pieces for tubes which interconnect and are secured by pinning, and many others (see Fig. 11). The author has found that the premiums paid for most of these patented devices do not justify their use.

Of all things which affect the joint details the most important is that of appearance and in the author's opinion the best detail is that which has the best appearance. In general, welded joints which do not use joint material or connection devices seem to best satisfy the requirement of appearance.

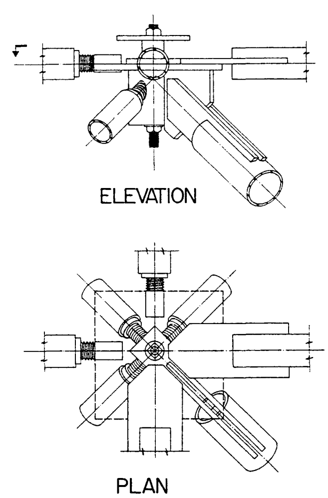

Figure 11

COLLECTION OF LOADS AT SUPPORTS

There are really only two choices for the transfer of load from a space frame to its vertical supporting elements. The vertical support can be made continuous with the space frame, or the juncture between the vertical support and the space frame can be a pinned connection (see Fig. 12).

When a vertical support is made continuous with the space frame it generally allows several panel points to be supported simultaneously by "column capital" action which in turn reduces web stress concentrations under vertical load. However, this continuity can provide even greater web stresses than those which would exist if only one panel point were supported. This condition can occur with certain combinations of lateral and vertical loading and vertical support stiffness.

The author feels the introduction of continuity between the vertical support and the space frame needlessly complicates the analysis of an already highly indeterminate structure. The justifications for continuity in the minds of some designers are:

1. Reduced deflection of the space frame
2. Resistance to lateral loads
3. More support perimeter at each vertical support
4. Conservation of floor area

These are subject to the following criticism:

1. Space frames are generally deep structures which are many times stiffer than the vertical supports and deflection is seldom the limiting design criteria.
2. The dead load of space frame structures and their support spacings are such that lateral loads can be effectively resisted by vertical supports with fixed bases without uplift being produced.
3. More support perimeter can be achieved regardless of continuity.
4. Floor area for a fixed base member need not be larger than that for continuous vertical support.

The use of a pinned joint at the intersection of the space frame and its vertical supports must allow rotation in all directions while transmitting lateral shear to the top of the vertical support. This can be accomplished by the use of a portion of a sphere seated in a lubricated pocket (Fig. 12). Also, for certain orders of magnitude of load and rotation, neoprene and other bearing pad materials may be used.

Where it is desired to increase the support perimeter a "shear head" type element can be introduced at the top of the vertical support element (Fig. 13). These are connected to the space frame panel points and project out from the center of the vertical support as cantilever beams, compression struts or tension hangers.

METHODS OF ANALYSIS AND DESIGN

The methods available for the analysis of space frames are:

1. The method of consistent deflections
2. The use of an equivalent anisotropic flexural grid treated by the method of finite differences
3. The use of an analogous orthotropic grid
4. The use of an analogous flat plate with moment distribution
5. Model analysis
6. Successive approximation with a computer program

The first two methods are covered quite well by Gaus and Sbarounis in the February 1959 *Journal of the Structural Division* of the ASCE.

The analogous orthotropic grid method is presented in Timoshenko and Woinowski-Krieger's *Theory of Plates and Shells.* The analogous plate method is presented in excellent form by Coy in his *Structural Analysis of Space Frame Roofs*, parts A and B.

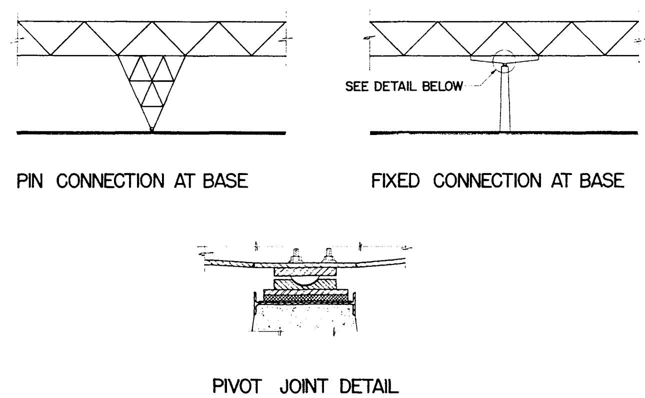

Figure 12

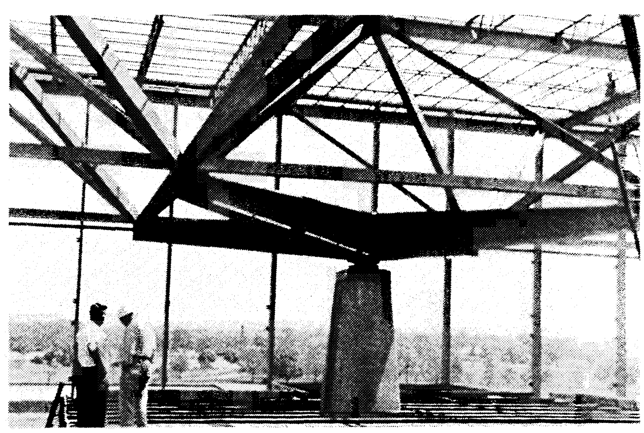

Figure 13

With regard to the use of structural model analysis it is the author's opinion that the degree of precision required and the scarcity of sufficiently qualified model analysis technicians tends to make this approach dangerous.

The use of the computer is strongly recommended for a successive approximation analysis. Although the programs available assume pin joints, it is our feeling that this is on the conservative side for large space frame lattices. The important consideration with any computer program is to completely understand what the program is doing so as not to be overcome and confused with its output or any portion thereof.

CONCLUSION

The staff of the Engineers Collaborative has had the opportunity of being involved in many unusual types of structures, including several which others have described as "significant" space frame structures. Analysis, design, preparation of design drawings and specifications, shop drawing checking and construction supervision can not be done by one or two people. The author wishes to recognize some of the men who worked on these problems: Mike Gaus, John A. Sbarounis, Winston Lau, Nick Gouvis, Arthur G. Jones, Stephen Tang, James E. Ambrose, John B. Hackler, and John H. Lee.

REVISED PAINTING SPECIFICATIONS

Steel Structures Painting Manual, Volume 2, Systems and Specifications

Steel Structures Painting Council, 4400 Fifth Avenue, Pittsburgh, Pa. 15213
Price: $7.50

This newly revised edition of Volume 2, a complete guide and index for the painting of bridges, buildings, tanks, towers, piping and other kinds of steel structures, should be of particular interest to readers of this Journal. Based on ten years of practical experience and upon exhaustive tests at 20 exposure sites, this revision includes the following new features:

SSPC Paint Systems Specifications now permit an entire painting system for most steel painting problems to be easily selected, including surface preparation, paint application procedure, primer, intermediate coat and finish coat.

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The Paint Application Specifications now provide for use of catalyzed paints, airless spray, hot spray, conventional spray, brushing, rolling, painting welds, contact surfaces, stripping and other special stipulations.

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Supplements scheduled for the next few years can readily be integrated in the new Manual, including special performance specifications now being verified on epoxies, coal-tar epoxies, zinc-rich and other coatings for which no completely suitable public specifications are presently available.