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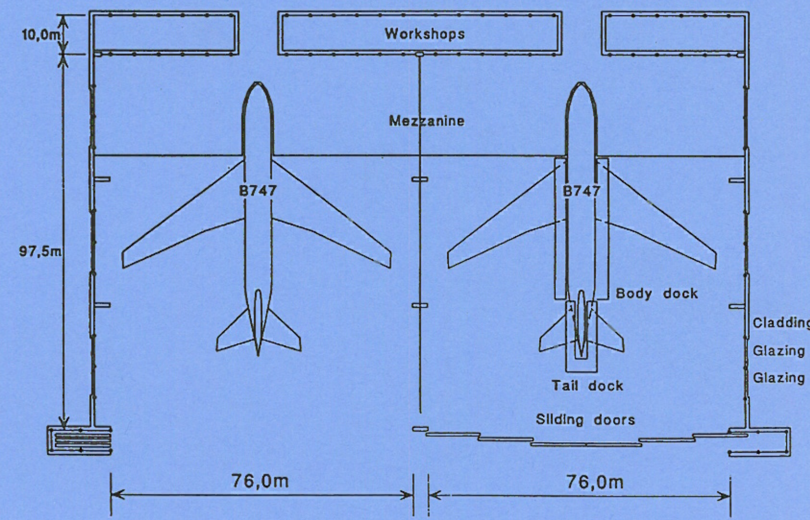

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OVERSIGT OVER ESDEP-MATERIALETS INDHOLD:

CONTENTS:

ESDEP WG 1B
STEEL CONSTRUCTION
INTRODUCTION TO DESIGN

Lecture 1B.5.2: Introduction to Design of Special Industrial Buildings

OBJECTIVE/SCOPE

To outline the principal features of the design of special industrial buildings.

PREREQUISITES

None.

RELATED LECTURES

Lecture 1B.5.1: Introduction to the Design of Simple Industrial Buildings

SUMMARY

Special industrial buildings are of two kinds - those which are of unusual construction and those which are designed for a special industry. Several features, such as handling methods, maintenance and fire protection, are briefly discussed. Examples of special buildings, e.g. power stations, hangers, are presented.

1. TYPES OF SPECIAL INDUSTRIAL BUILDINGS

Special industrial buildings are of two kinds - those which are of unusual construction and those which are designed for a special industry. The main characteristic of such buildings is that they are invariably designed for a particular purpose or process, and are consequently virtually impossible to adapt for another kind of use.

Among the former are industrial buildings which, for reasons of prestige rather than economy, utilise unusual structural forms which provide architectural expression and thereby contribute to the visual quality of the building. Because buildings of this kind are unique they cannot be considered generically. Some examples are briefly described later in this lecture.

Among buildings designed for specific industries are heavy engineering works, aircraft hangars, power stations, process plants, steel rolling mills and breweries. Many of these buildings have similar features which are considered in principle below.

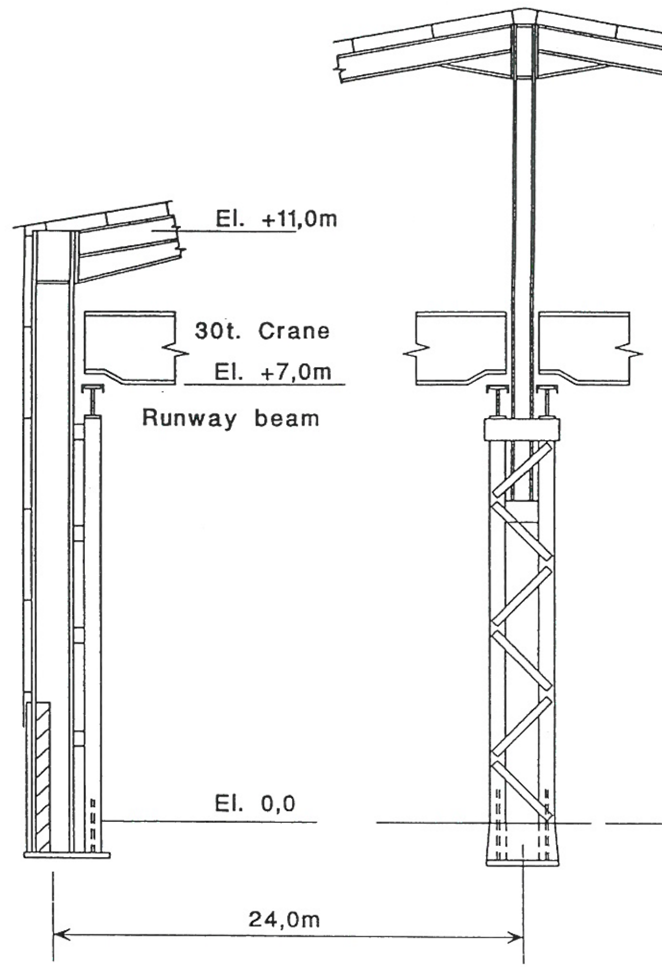


Figure 1 Compound columns

2. HANDLING METHODS

Overhead cranes with capacities of 10 tonnes and more are a characteristic of heavy engineering works and power stations. They require the support of compound columns and runway beams to carry the vertical and surge loads (Figure 1). Light overhead cranes with capacities of 1 to 5 tonnes, are a characteristic of aircraft hangars and light industries. They can be attached to the roof structure and be designed for multiple supports for wide coverage, or they can be arranged to transfer laterally from bay to bay (Figure 2). Roof flexibility may become important for roof-mounted cranes used for assembly.

Some years ago, so-called NoRail cranes were developed.

The NoRail crane concept inverts the overhead crane principle. Short rails are mounted in the endtrucks of the crane. These rails run along a series of stationary wheels. The rails are designed to be somewhat longer than the maximum distance between three adjacent support points, so that the crane is always supported by at least two wheels on each side (Figure 3). As a result of this design, the long conventional crane track becomes superfluous. The benefits of this innovative design arise both in cost savings (up to 20%) on the steel structure of the building and in material handling. Crane travel "tracks" that cross each other are feasible.

Conveyors can be either floor or roof mounted. Conveyors for assembly purposes may carry appreciable weights, and are of necessity suspended from the roof (Figure 4). Power roller conveyors are also used for transport of bulky items and are usually floor mounted.

As a result of advances in design, motorised floor transport vehicles including fork-lift and pallet trucks are now very common. The main influence they have on design is on the floor quality and on headroom.

Automated pallet stacking by fork-lift trucks of specialised design may require very stringent control of fabrication and erection of the stacking racks. The racks may be incorporated in the structure of the building (Figure 5).

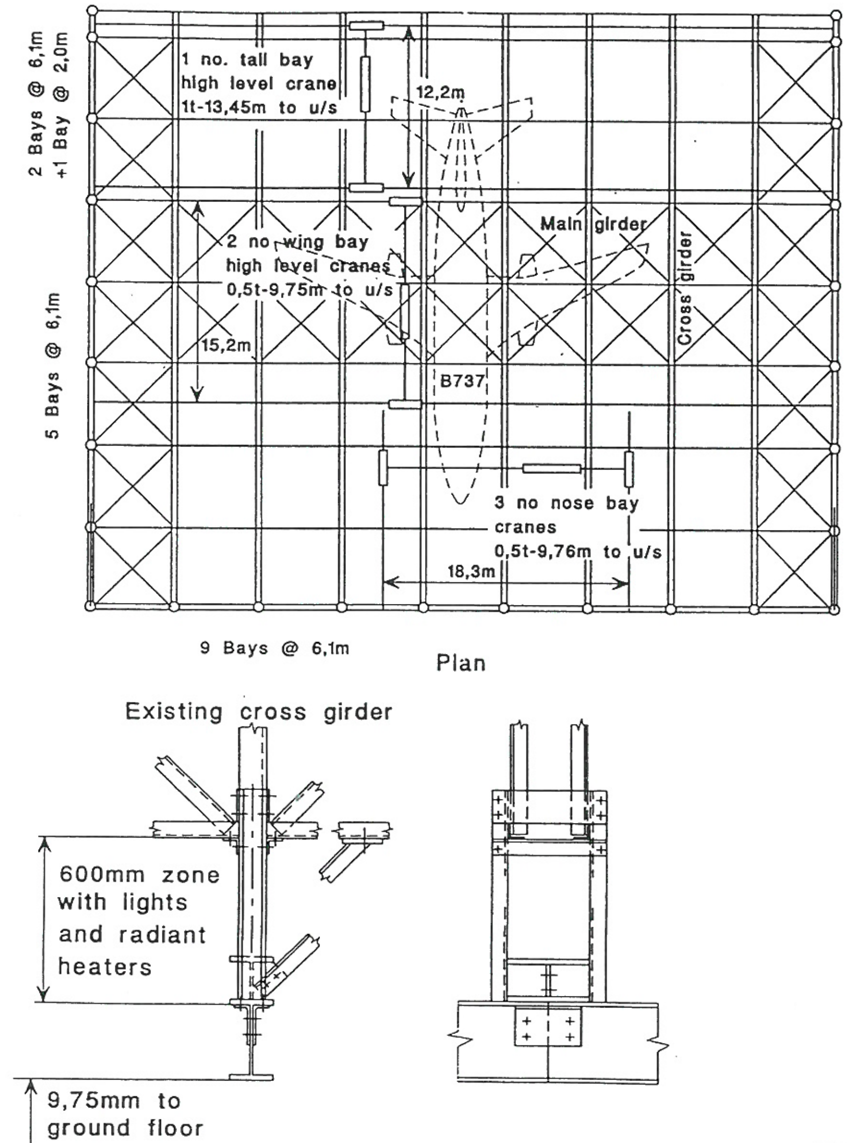


Figure 2 Underslung crane installation in existing hanger

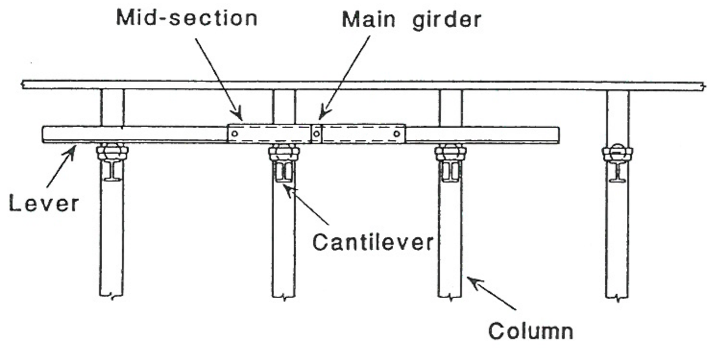


Figure 3 NoRail crane

Lecture 1B.5.2

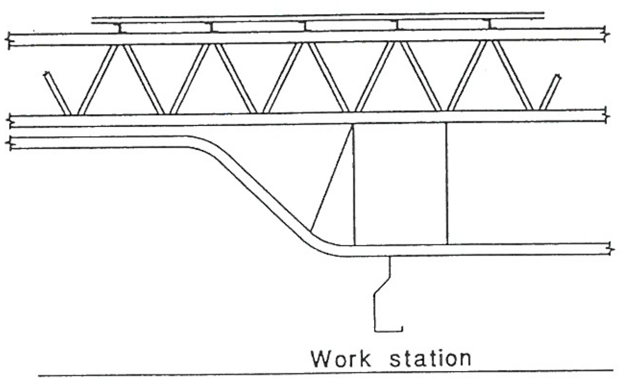


Figure 4 Conveyor system

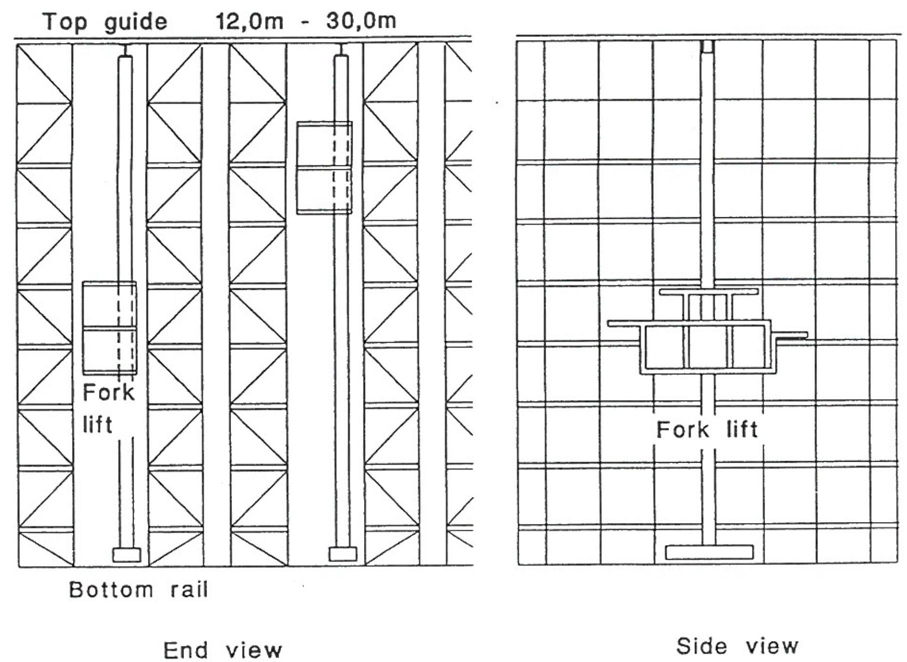


Figure 5 Palletised rack storage

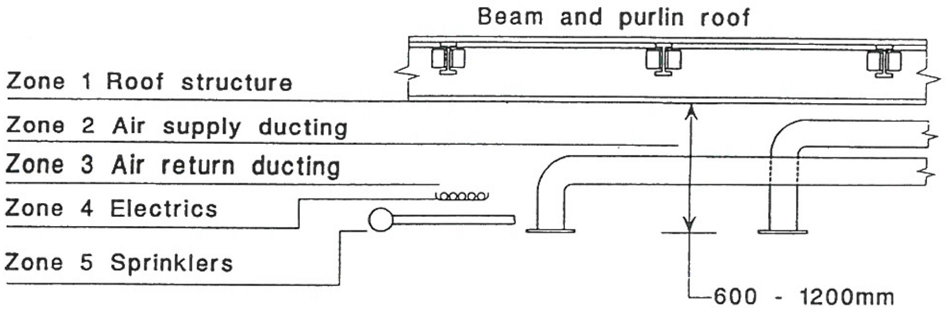
Lecture 1B.5.2

3. DAYLIGHTING

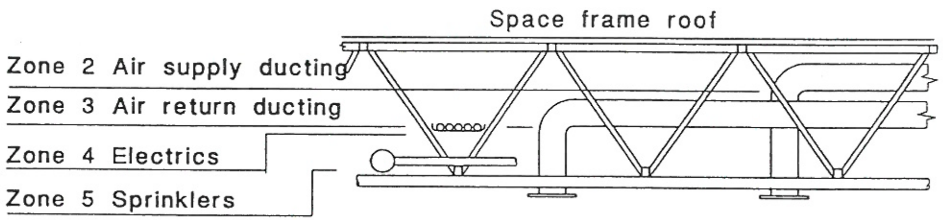
Few industries now have particular needs in respect of daylighting, since shift work is often provided for. Sidewall and roof daylighting is usually described as a percentage of the plan area, 5% giving sufficient light for bulk storage, 20% for a working process. Some artificial lighting is usually employed to establish a consistent high level of illumination, daylighting may be provided for visual comfort or for architectural effect.

4. SERVICES

The amount of services can vary in different parts of a building, from an exacting standard of air conditioning appropriate to a "clean room" to extensive process ductwork. The support and passage of services can be facilitated or hindered by the roof construction (Figure 6). The heating of high single-storey structures is always a problem, particularly when fire safety places stringent control on the temperature of the heat source. Inevitably provisions for cranage, lighting, heating and services such as air and electric power, will conflict. They each influence the structural design. Sometimes, if services are particularly extensive, it is advantageous to use a structural form which provides abundant support for services.



(a) Services placed below roof structure



(b) Services placed through roof structure

Figure 6 Services at roof structure

5. SPECIAL ROOF LOADING

Whilst it is usual in advanced factory units to allow in the design of the roof a nominal overall loading for services and a single point load on the main members, this provision may not be sufficient for special buildings. Roof loading may be determined by provision for future developments of the process for which the building is designed, or for developments in handling methods or access platforms designed for improved productivity. These provisions may cause major loads on the roof. Whilst it is not possible to take into account every possible development which can influence the building design without incurring large additional cost, it is much cheaper to incorporate surplus strength in a building at the design stage than to add additional strength after completion, particularly if intensive use of the building would conflict with the strengthening operation. The ability of the structure to laterally distribute local loads may influence the choice of structure. Space frames, for example, have exceptional capabilities in this respect.

6. MAINTENANCE

Every material used in construction has a limited life, which can usually be extended by appropriate maintenance. Maintenance is likely to be particularly important in special buildings. The design of the building should allow suitable access for the maintenance required. Maintenance may conflict with the planned usage of the building, which can easily occur if usage is intensive, as, for instance, if maintenance requires dismantling or opening up, or if radiography requires areas cleared for safety.

Roof maintenance is particularly important. The possible results of overflow due to rainwater outlets being blocked, either by process emissions or by snow or hail needs to be considered in assessing the merits of the roof design, the routes for rainwater disposal and the maintenance necessary. The deterioration of the roof covering due to weather or to aggressive effluent also needs consideration.

7. FIRE PROTECTION

Due to the characteristics of the process to be carried out in a special building, it may require exceptional measures in respect of fire and explosion prevention, and in fire protection and damage limitation. Sprinkler installations of exceptional capacity may be required, as well as carbon-dioxide injection.

Dust explosion is a risk in processes dependent on the transport of finely divided powders by conveyor or air duct. Controlling the results of an explosion is often achieved by strategically placed blow-out panels. Gas explosions can be far more destructive and difficult to control.

8. SOME EXAMPLES OF SPECIAL BUILDINGS

8.1 Coal-Fired Power Stations

A typical medium-sized power station (Figures 7 and 8) consists of a 38,6m span turbine hall, flanking a 13m span bunker bay beside a 31,5m span boiler house and 12m wide air heater building. The height of the turbine hall is typically 30m, determined by the servicing requirements of the turbines and generators. The height of the bunker bay, which stores several hours fuel, and that of the boiler house are similar, determined by the height of the boiler and the size of the fuel mill below, and is typically 60m. The length of the building depends on the number of generators installed, each having its own boiler.

This type of power station is constructed almost entirely of structural steelwork and steel cladding. Steel construction is chosen because the completion of the boiler house is always on the critical path of the execution schedule. The execution of the boiler frame, designed to suit the boiler and from which the boiler is suspended, is central to the schedule. The stanchions of the boiler frame, often six in number, are typically compound H-section, carrying up to 1000 tonnes each, and the boiler is suspended from heavy plate girders spanning across the stanchions. The external steelwork to the boiler house is relatively light, being mainly supported by the boiler frame which also braces the building.

In the bunker bay, which is also a steel structure, are large feed bunkers of 600 tonnes capacity constructed of steel plate, supported at high level, to which fuel is supplied by conveyors. There is a fire and explosion hazard in the feed conveyors and the ductwork connecting the bunker to the fuel mill and the latter to the boiler. Sprinkler and carbon-dioxide fire protection is therefore required in this part of the plant, and fire protection is also applied to the steelwork.

In the turbine hall the generator sets are supported 10m above floor with condensers fitted below. Due to the weight of the generator sets the supporting structure, which is usually of steel but may be of concrete, is of heavy construction. To carry out maintenance of the generator sets a 100 tonne overhead crane travelling the length of the hall is provided, requiring heavy compound sidewall stanchions to support the runway beams. The roof structure is of light lattice girders except where additional strength is required to facilitate the installation of the crane.

Provision for extension of the turbine hall can be made, but extension of the boiler house depends on the choice of boiler, so that the ease of joining to existing steelwork has to be relied upon.

Maintenance of the generating plant is an important consideration in the design of a power station. Maintenance of the building is reasonably straightforward, since generation does not create aggressive conditions or waste. Corrosion is not a major problem, so that it is adequate to shot-blast and coat the steelwork.

The construction of power stations of this type displays the versatility of steel, its use varying from heavy steelwork for the support of plant to light roof steelwork and sheeting. Allied to this versatility is speed of execution on site,

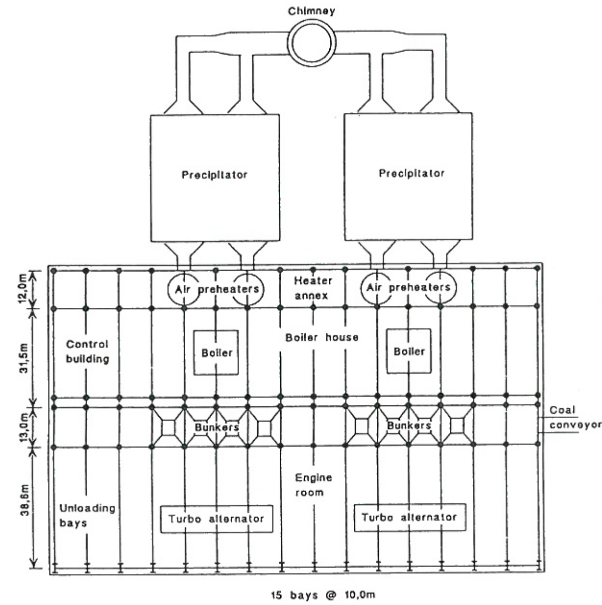


Figure 7 Coal fired power station

Lecture 1B.5.2

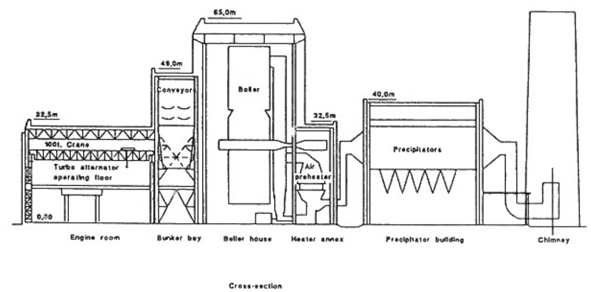


Figure 8 Coal fired power station: cross-section

Lecture 1B.5.2

which off-site fabrication allows. It is therefore understandable that steel is used almost exclusively in this field of application.

8.2 Aircraft Maintenance Hangar

A typical hangar bay for the maintenance of Boeing 747 aircraft (Figures 9, 10 and 11) is 76m wide and 97,5m long, and the hangar may consist of one, two or three bays. The maximum clear height is usually 23,5m to allow clearance over the 20m high tail fin of the aircraft, but only 17m is required over the body and main wings. The roof may therefore have two levels, the height in the tail area being 23,5m, the remaining area 17m. The two-level roof restricts the attitude of the aircraft to nose-first, whereas a full-height hangar allows either nose-first or tail-first attitude. At the rear of the hangar is the 2-3 storey workshop and administration block, 10m deep and the same width as the hangar. The roof slope is usually small to avoid excessive height, utilising either an insulated roof membrane on metal decking or insulated two-layer cladding. The roof structure is usually comprised of lattice trusses, girder or portal frames, but double-layer grid space frames have also been used.

The main door is usually 21m high, and can be a sliding-folding or slab-sliding design. The full opening width required is 80m. If bunching space for the doors overlaps the door opening the bay width is increased correspondingly. Some hangar doors are only 14m high with a 7m high tail gate, or they may have a vertically folding 21m high centre section.

Whilst some smaller hangars have been constructed in prestressed concrete, virtually all are now constructed in structural steelwork with insulated steel cladding.

Hangars are specialised for maintenance of one type of aircraft or a mix of types. Access to an aircraft, because of its shape and size, is a problem which is best solved by specially designed docking tailored to suit the particular aircraft. This arrangement enables a large workforce to carry out maintenance. Typically the docking consists of main wing docks, a tail dock and body dock. They are moved into place after the aircraft has been placed in a fixed position. Since aircraft are jacked up 1,5m for landing gear overhaul, it is usually necessary for the docks to have vertical adjustment. The use of wheel pits can make jacking unnecessary, but these add considerably to cost as well as adding to specialisation.

Unless they can be moved out of the hangar, docks occupy a large amount of floor space. They obstruct the placing and maintenance of other types of aircraft when not in use. Consequently tail docks and body docks are sometimes suspended from the hangar roof. Since tail docks weigh 12-50 tonnes and body docks 50-100 tonnes, provision for them must be incorporated in the roof design.

Hangars are usually provided with light overhead cranes covering the full area. They are used to handle dismantled parts up to 1 tonne weight. Isolated engine hoists up to 10 tonne capacity may also be provided. Alternatively the overhead cranes may be of 10 tonne capacity. Conflict can arise between cranes and

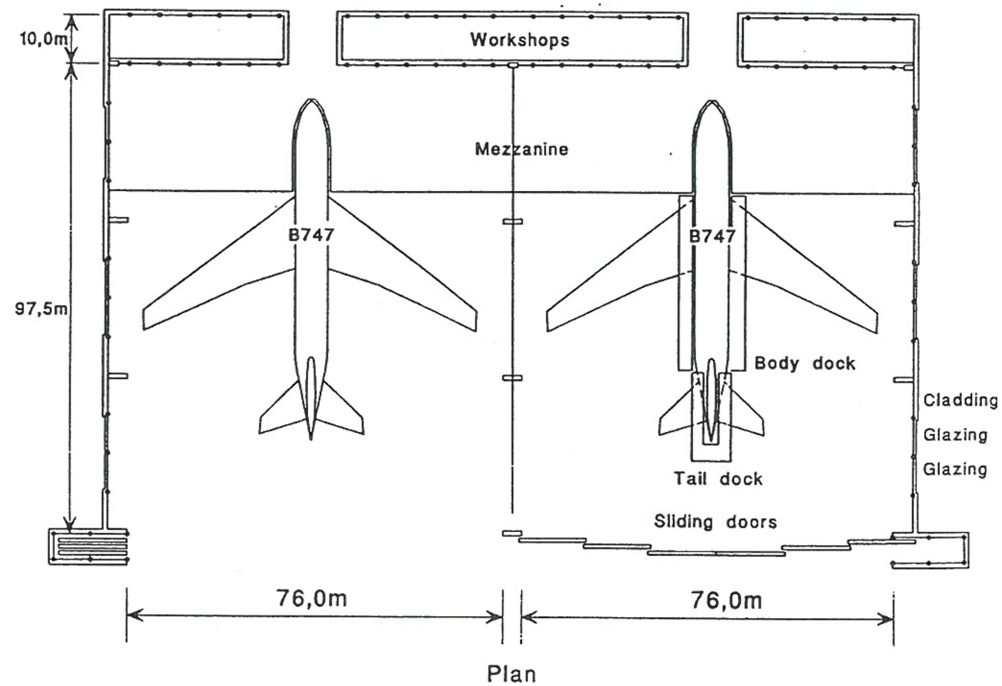
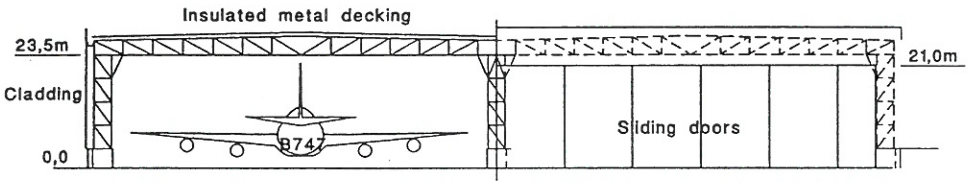


Figure 9 Two-bay aircraft hanger



Cross-section

Figure 10 Two-bay aircraft hanger

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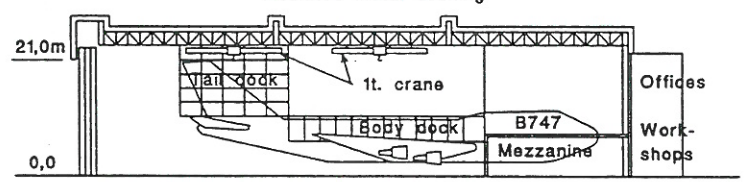

Longitudinal section

Figure 11 Two-bay aircraft hanger

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suspended docking. If a two-level roof is adopted, separate cranage is required in the tail bay.

Electric power, air and other services may be from roof-mounted motorised reels or in the floor. Heating is by embedded floor coils or high-power blowers suspended from the floor. Blowers are large units appropriate to the height of the hangar. Sprinklers may be installed, depending on the extent of the maintenance carried out and the safety procedures adopted regarding on-board fuel.

Except for roof maintenance, the maintenance requirements for a hangar are usually slight, since aggressive emissions are confined to drainage from the hangar floor where painting is carried out, or from cleaning or chemical process shops. Due to the large roof area and its height, and to the characteristically exposed environment of an airport, storm damage is always possible. Roof leaks can have very serious consequences, because of the high value of aircraft parts.

Developments in aircraft design and increased competition for contract maintenance make it necessary to allow for modifications to a hangar. The introduction of the 747 type and other wide-body aircraft compelled the extension of many of the hangers in use at that time. However, the intensive usage of a hangar and the strict fire and safety regulations applied when aircraft are inside makes modification difficult to carry out. Flexibility therefore needs to be allowed for at the design stage.

The superiority of structural steelwork for aircraft hangars is now well established. The speed of construction, suitability for large-span roofs, versatility for the mounting of various services and docks, and the adaptability for future development virtually exclude other structural materials.

8.3 Milk Powder Plant

A typical milk powder plant (Figures 12, 13 and 14) consists of a spray-drier tower 18m by 17m by 32m high with an external boiler house, a silo and packing plant annex 16m by 18m, and a storage warehouse for packaged powder 54m by 54m with 7m clear height for fork-lift transport and stacking.

The tower and annex are framed in structural steelwork, with composite concrete floor and steel cladding. The warehouse typically has multi-bay short-span portal frames carrying pressed steel purlins and asbestos cement or single-skin metal cladding.

The spray drier is a 10m dia. stainless steel drum 14m high, supported at several floors. Milk and hot air are injected at the top, and the dry milk powder collects in the hopper bottom. From there it is conveyed to the silos of the packing plant. The floors are lightly loaded except for ancillary plant and the spray drier, which in operation weighs 60 tonnes.

There is an appreciable explosion risk from the finely divided milk powder. Strong explosion ducting with an exterior blow-out panel, intended to control the

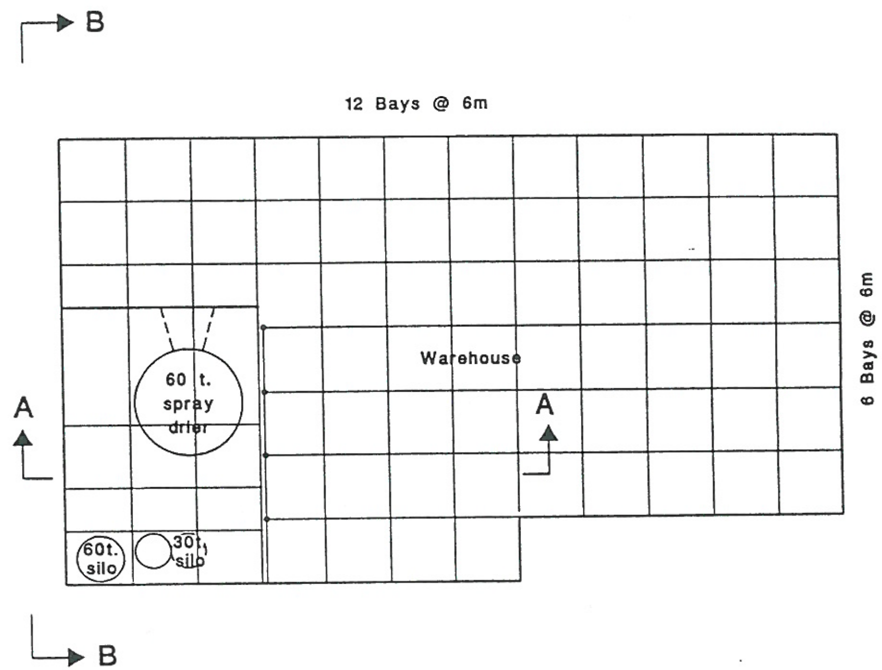
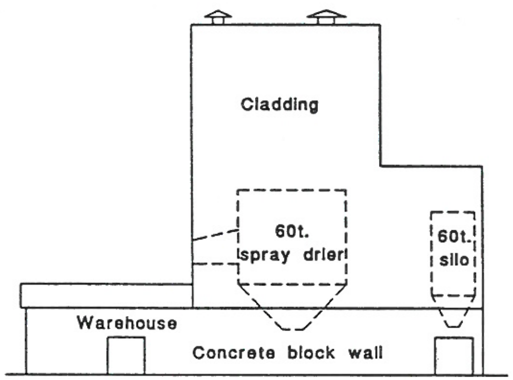
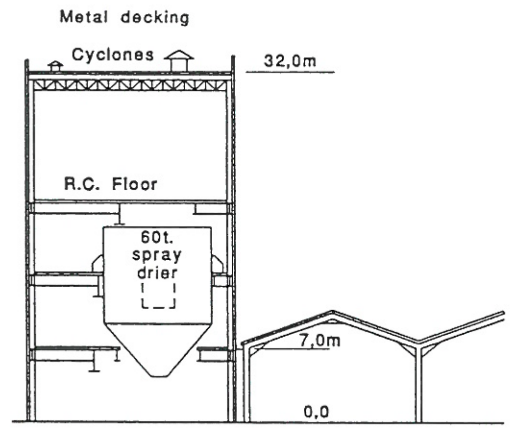


Figure 12 Milk powder plant



Elevation B-B

Figure 13 Milk powder plant



Section A-A

Figure 14 Milk powder plant

direction and result of an explosion, is incorporated in the drier, and provision for this facility is made in the tower steelwork.

The large amount of air injected in the process requires outlet cyclones to extract milk powder from the exhaust air. Even with regular maintenance, cyclones are never 100% efficient, so that some powder, which can accumulate quickly, escapes. Deposits of powder can cause problems with roof drainage, which therefore requires appropriate design. Milk powder contains lactic acid which is moderately aggressive particularly to flat roof coatings such as asphalt and felt. Consideration of the durability of the roof is therefore required.

Internally a biologically clean environment is required in order that the plant complies with process regulations. Easily cleanable surfaces are required internally. This requirement is best met by high quality internal sheeting. Avoidance of crevices which can cause lodgement of material, affects the choice and detailing of any steelwork exposed internally.

Competition in milk powder production requires that first cost and running cost are carefully controlled. Since development of driers occurs, a change of drier may be necessary involving major alterations to the tower. The use of structural steelwork and cladding facilitates cost control in both construction and modification.

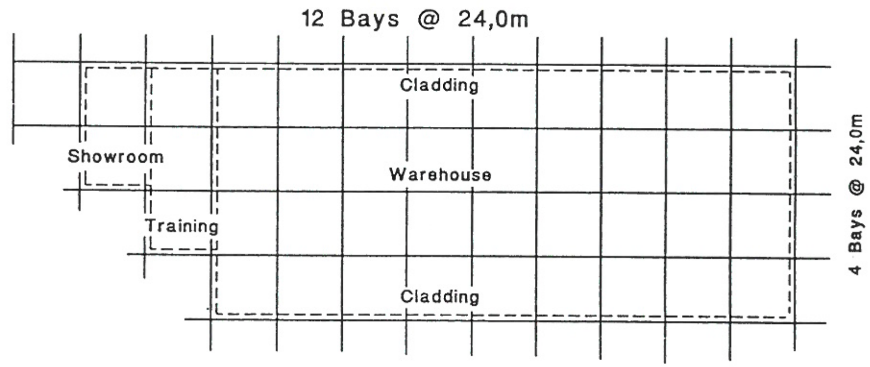
8.4 Industrial Complex

Some major industrial projects provide both the scale and the opportunity for adopting unusual structural forms which have particular advantages. A good example of an unusual structure form is the Renault Parts Distribution Centre in Swindon (Figures 15, 16 and 17).

The requirement was for a single-storey building of 25.000 sq.m containing a warehouse, training school, showroom and office, with provision for 50% expansion. To suit the storage arrangements for the warehouse a 24m x 24m bay was adopted, with 8m internal height, with 2,8% roof lighting and sidewall glazing in some areas. The main area is 4 bays wide and 9 bays long, with an additional 6 bays at one end.

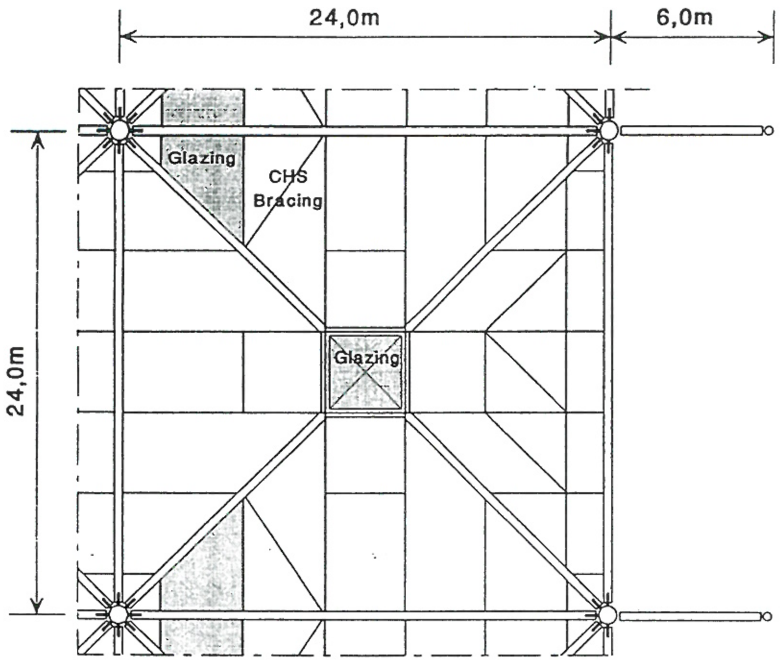
The structure consists of skeleton portal frames on both rectangular and diagonal axes. The main verticals are 16m high 457mm dia. circular hollow sections with rod stiffeners. The roof members are simple trusses formed from shaped I-beams cambered 1,4m stiffened on the underside with rod bracing and short tubular verticals. Continuity between the main verticals and the trusses is established by rod bracing connecting the heads of the main verticals to the quarter-points of the trusses. Whilst the internal verticals are balanced by trusses on each side, the perimeter verticals, which have transverse and diagonal trusses on one side only, are balanced by ground anchors bracing short beam members connected to the verticals at the same level as the trusses.

Macalloy bars are used for the rod stiffeners to the main verticals, and Fe 510 steel is used for the main rod bracing. The rods are connected to the main verticals by purpose-made cast-iron eyes pinned to lugs welded to the



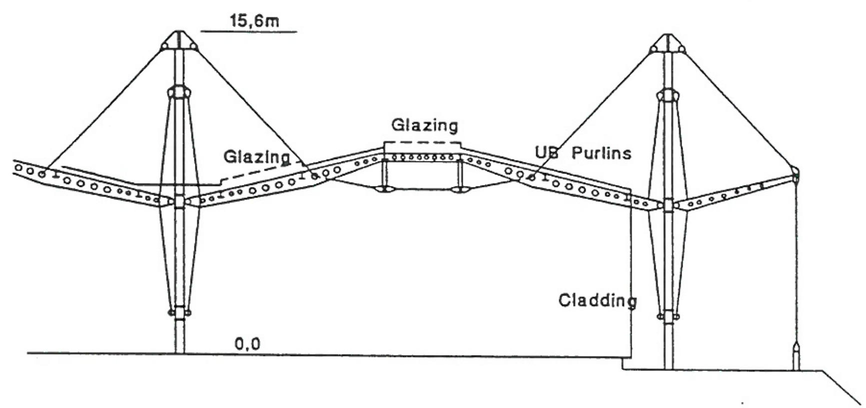
Layout plan

Figure 15 Renault Centre, Swindon



Part plan

Figure 16 Renault Centre, Swindon



Part section

Figure 17 Renault Centre, Swindon

457mm dia. hollow sections, and to the trusses through sleeves set into the beam sections.

In each bay the trusses are cambered to a central 4m x 4m dome rooflight. The roofing consists of an insulated membrane on metal decking, which is carried on purlins between the trusses. Valleys formed by the cambered trusses are drained by downpipes incorporated in the main verticals. Both main vertical and bracing rods pass through the roof covering.

The overall appearance is unusual, resembling a large marquee due to the tent-like profiles of the cambered trusses and the main verticals and bracing rods protruding through the roof.

9. CONCLUDING SUMMARY

- Special structures are needed for some industries. They may also be provided for reasons of prestige.
- Cranes and conveyors carry appreciable weights and may be suspended from the roof.
- If services are extensive, it is advantageous to use a structural form which provides abundant support.
- Speed of construction, suitability for large spans, versatility for the mounting of services and adaptability all favour the use of structural steelwork for industrial buildings.

ESDEP WG 14
STRUCTURAL SYSTEMS: BUILDINGS

Lecture 14.5: Space Structure Systems

OBJECTIVE/SCOPE

To describe different types of spatial truss systems, and the design parameters to be considered. To give guidance on initial sizing and on analysis methods. To describe fabrication and erection procedures.

PREREQUISITES

Lecture 1B.3: Background to Loadings
Lecture 6.3: Elastic Instability Modes
Lecture 7.12: Trusses and Lattice Girders

RELATED LECTURES

Lectures 13: Tubular Structures
Lecture 14.6: Special Single Storey Structures

SUMMARY

The lecture provides an historical background and an overview of different types of spatial truss systems: double-layer grids, barrel vaults and domes. Design parameters are introduced and some rules for initial sizing are described. The principles of different methods of analysis are given. The lecture concludes by describing aspects of fabrication and erection particular to these structures.

1. INTRODUCTION

1.1 Definitions

For this lecture, trusses are defined as structural systems in which the members are interlinked so that they are only subject to axial compressive or tensile forces.

This definition assumes that no action is applied directly onto the members. All loads are applied to the joints which are known as 'nodes'. In case it is impossible to guarantee the coincidence of member axis, the bending effect resulting from this must be evaluated. It is particularly important to ensure that the axes of the members coincide (Figure 1). Only perfect pins could completely ensure compliance with this loading condition. The technological construction of assemblies deviates to some extent from this theoretical situation and, in effect, is one of the main difficulties associated with these structural systems.

The lecture is concerned mainly with truss systems for roofs, which span in two directions (termed 'space structures'). Other arrangements are possible, such as continuous systems, based on the Vierendeel girder (Figure 2) in which diagonal bracing members are unnecessary because the bending behaviour is predominant, rather than the axial one; the resulting voids can be used to accommodate mechanical and electrical services.

1.2 Historical Background

Until the 1960s, almost all truss systems were two-dimensional. They had developed from timber roofs, which themselves had evolved from a basic triangular arrangement to more complex shapes (Figure 3). The need to lighten long tie beams and reduce bending stresses (Figure 3a) had led to the introduction of a suspender (Figure 3b). A similar concern to reduce bending of the rafters led to the introduction of diagonal members (Figure 3c). By dividing the suspended member in two, the familiar arrangement of Figure 3d was obtained.

The use of metal became dominant in the 19th century for all types of structures except domestic buildings. Articulated systems (Figure 4) commonly used for roofs of railway stations were perfect examples of the two-dimensional triangulated system. The second half of the 19th century was characterised by some remarkable achievements, for example the Garabit viaduct in France (Figure 5) and the Maria Pia bridge in Oporto, both designed by Eiffel.

Although spatial systems were proposed early in the 20th century, their use in practice has arisen from the more recent development of computer methods for analysis, the functional need for spaces free of columns and from demands of architectural appearance.

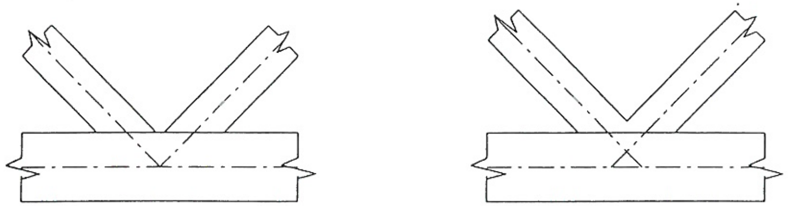


Figure 1 Coincidence of member axes

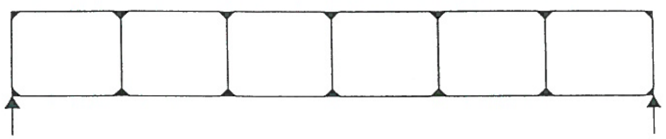


Figure 2 Vierendeel girder

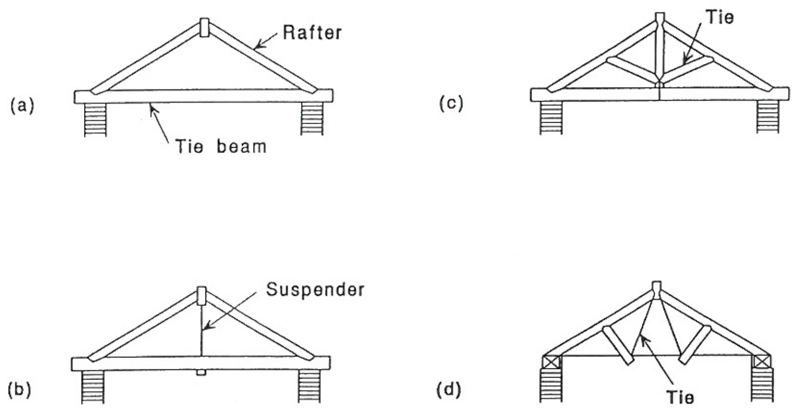


Figure 3 Evolution of construction of timber roof trusses

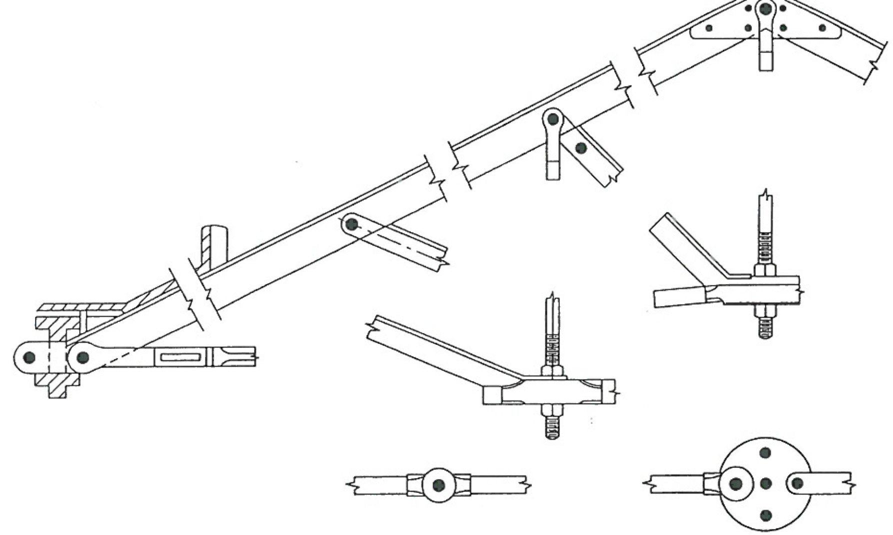


Figure 4 Euston Station details

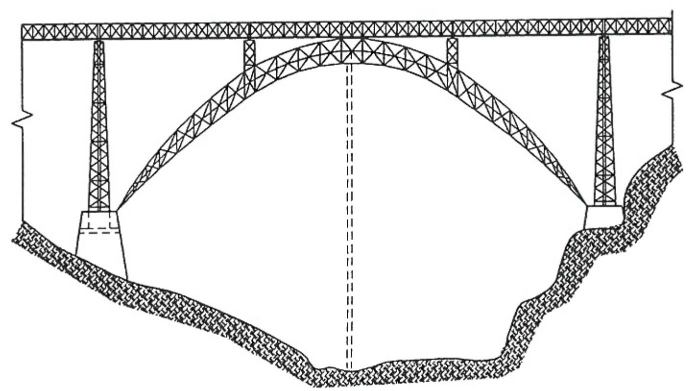


Figure 5 Viaduc de Garabit - ligne de Marvejols à Neussargues (Cantal)

1.3 Different Types of System

1.3.1 Introduction

Different types of spatial truss systems are normally classified according to their general shape. The following may therefore be distinguished [1]:

- two dimensional grids
- cylindrical vaults
- domes.

In each case it is advisable to distinguish between single and double or even triple layer grids. The number of layers depends on the span. A third characteristic lies in the geometry chosen for the system of members in the layers and possibly in the composition of the bracing of the layers.

1.3.2 Two dimensional grids

1.3.2.1 Single layer grids

These grids are mentioned only as a reminder that these systems are beam grillages which work in bending and torsion, rather than under axial compression and tension.

Depending on the directions assigned to the members, grids may be identified in two, or three directions (Figure 6). Grids in two diagonal directions are more rigid (beams follow the direction of the principal stresses of the equivalent plates) and are widely used. Utilisation is restricted to about 10m of span.

1.3.2.2 Double layer grids

These grids comprise two systems of members on two parallel levels (upper and bottom layer). Both these systems are interlinked by bracing members (web members) (Figure 7). Two types of double layer grids may be distinguished (Figure 8):

- lattice grids where there are always top and bottom chords in the same vertical plane (Figure 8a).

- spatial grids, made up of triangular based pyramids, square or hexagonal (Figure 8b). Two kinds may be identified: one where the layer geometries are identical though displaced (offset grids), and the other where the layer geometries are different (differential grids).

These systems are suitable for spans up to 100m. For greater spans, it is necessary to incorporate triple layer grids, to avoid long members otherwise necessary with the increased depth.

The size of the constituent modules depends on several factors, principally: span, load, cladding system, type of node, transportation and erection facilities.

For spans of between 30 and 40m, member lengths of about 1,5m to 3m are acceptable.

The advantages of double layer grids are numerous:

- they are three dimensional structures which can withstand loads from any direction.

- they are hyperstatic, and buckling of some compression members does not cause the whole to collapse as has been demonstrated by mathematical models and experiments.

- their rigidity minimises deflections.

- they have a very good fire resistance.

- their composition allows factory pre-fabrication in modular elements, which are easily transported. Fabrication precision ensures ease of assembly and erection.

- they allow a wide choice of support positions owing to modular construction.

- the space between the two layers may be used to install electricity, electrical and thermal piping, etc.

- installation is carried out by bolting and may be done whatever the atmospheric conditions.

- they provide indisputable aesthetic qualities.

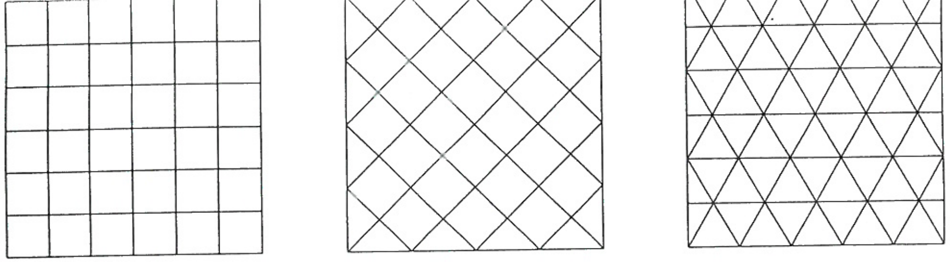
From an economic point of view, it is important to have a minimum number of nodes. It is therefore necessary to compromise between this criteria and those determined by the choice of module sizes.

1.3.3 Cylindrical vaults

In the history of construction, cylindrical vaults appear as an evolution of arches. The use of metal has enabled construction to be carried out with factory prefabricated elements which may be assembled on site. The first example to recall is the Crystal Palace which was erected by Joseph Paxton for the Great Exhibition in 1851.

This shape has proved to be suitable for roofs of halls, railway stations and sports facilities, e.g. in-door tennis courts.

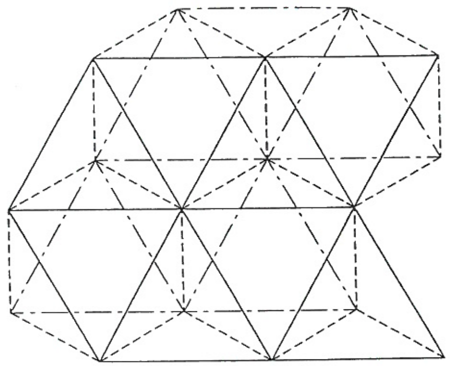
Maximum efficiency may be attained for shapes with rectangular surfaces and a length/width ratio of between 1 and 2. The optimum shape (rise/span ratio) is in the region of 0,15 to 0,20 (Figure 9).



(a) Square : two directions (b) Diagonal : two directions (c) Three directions

Figure 6 Simple layer grids

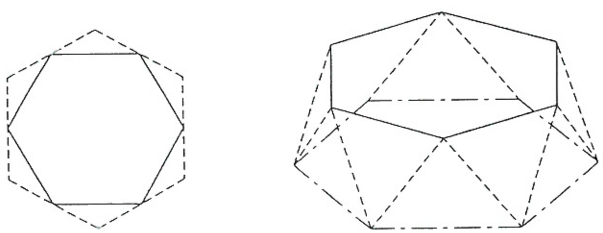
Lecture 14.5



—— Upper layer
- - - - Bracing layer
— · — Bottom layer

Plan

(a)



(b)

Figure 7 Double layer grid formation

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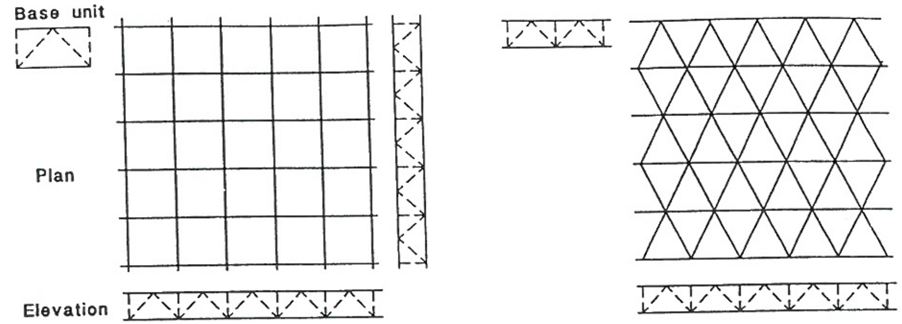
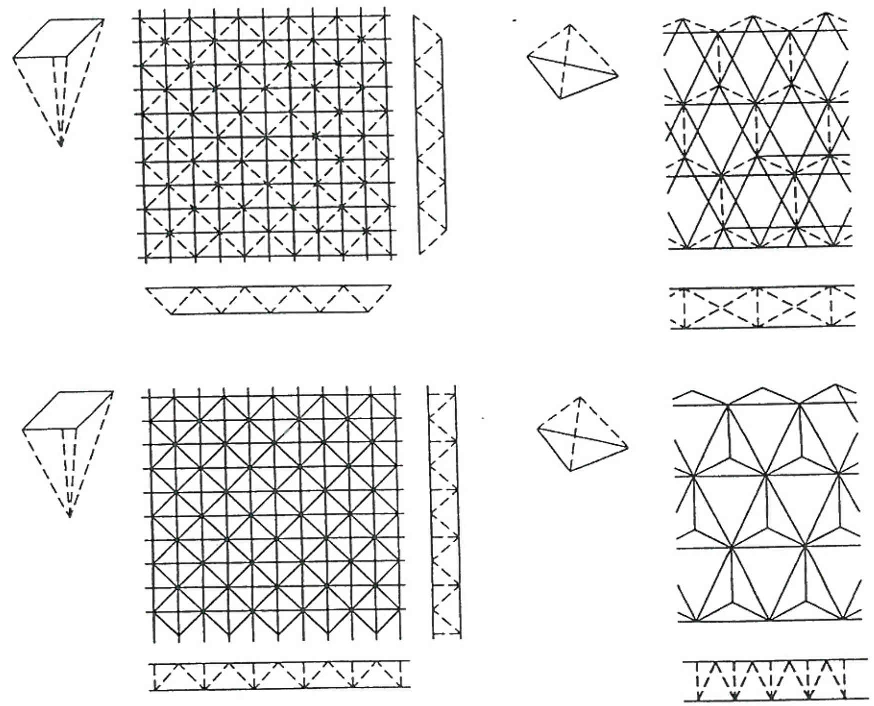


(a) Lattice girders



(b) Space grids

Figure 8 Double layer grid types

Lecture 14.5

Several layer geometries are possible (Figure 10). In practice, three directional systems offer the most advantages. They may usually be analysed by assuming pin-jointed behaviour for the nodes. This assumption does not hold, however, for some systems where bending rigidity must be taken into account, e.g. for a vault composed of prefabricated elements rigidly joined by high strength bolts.

The sensitivity of these systems to asymmetrical actions, in particular to wind, should not be underestimated. Such actions can even bring about force reversal in the members, which is an additional reason for choosing a three directional geometry in which the length of all elements is identical.

In the same way, choice of support conditions along the boundaries influences force distribution.

Economical spans for single layer vaults are in the region of 20m. Spans may be increased by inserting diagonal elements. They reach 60m for double layer systems, in some cases even more. Appropriate weights for double layer systems vary between 0,13 and 0,25 kN/m^2 depending on the intended shape, support conditions and on the geometry of the sheets (for a uniform load of between 0,75 and 1,50 kN/m^2).

1.3.4 Domes

Domes constitute one of the most ancient forms of construction. However big or small their size, the outline of the two-dimensional support is normally circular on plan.

Skeletal dome structures can be classified into several categories depending on the orientation and position of principal members. The four more popular types are: ribbed domes, Schwedler domes, three-way grid domes and parallel lamella domes (Figure 11).

Domes are of special interest to architects and engineers because they provide the maximum enclosed volume for minimum surface area. In the last 25 years construction with steel sections has largely replaced reinforced concrete. The first two examples concern single and double layer systems. These systems permit spans of about 40m and more than 100m respectively. Some double layer solutions have encouraged 'record' spans of more than 200m.

The accurate analysis of domes has only been possible with the introduction of computers which have allowed an accurate study of elastic behaviour. It is important to note that large span single layer domes subjected to asymmetrical actions can experience global instability effects. In addition to consideration of local buckling of compression elements, specific attention should be drawn to possible global 'snap-through' buckling (Figure 12). The action of wind is not very well known; application of factored actions to a non-critical load case does not necessarily cover the most unfavourable situation and recognition of an appropriate excess weight does not necessarily bring about the most unfavourable situation.

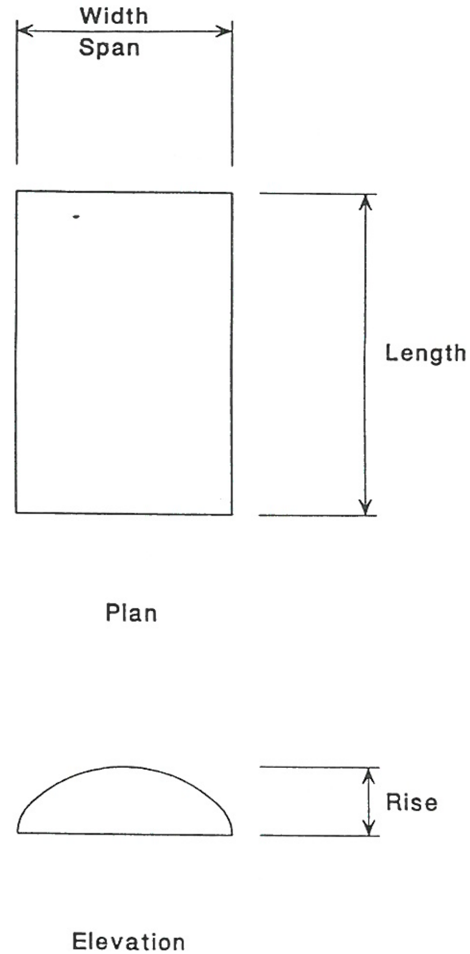


Figure 9 Cylindrical vault

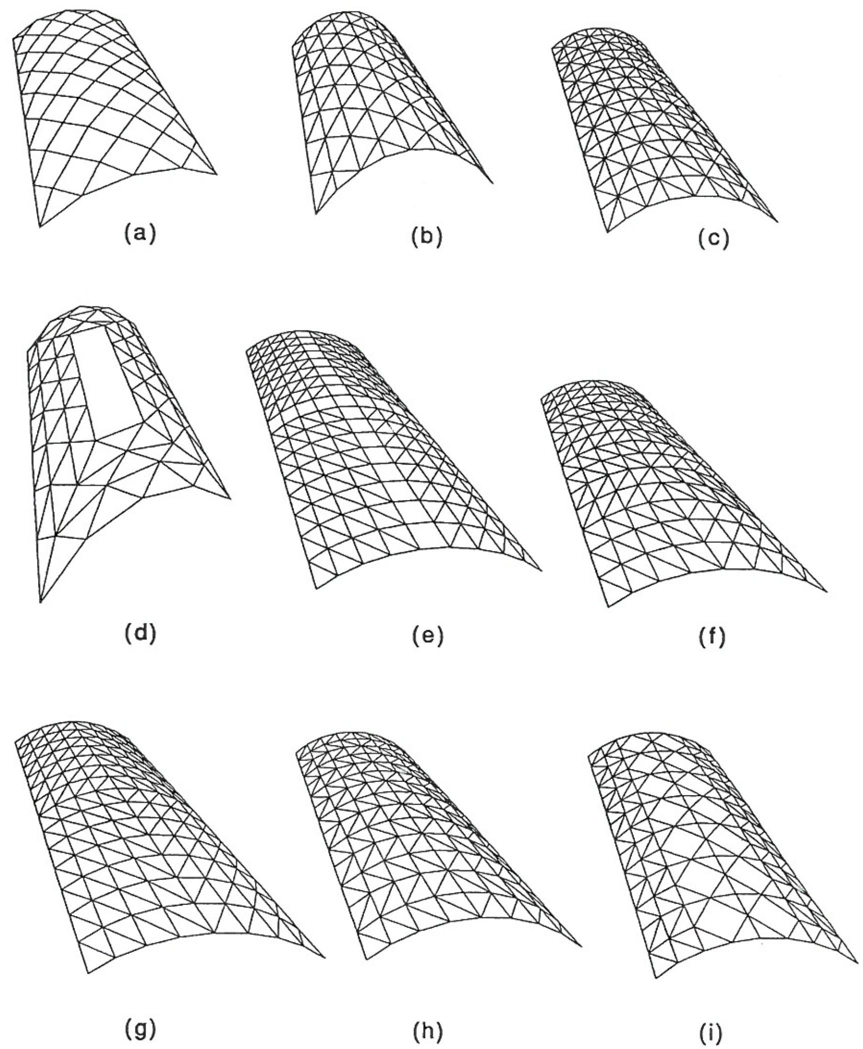


Figure 10 Cylindrical vault configurations

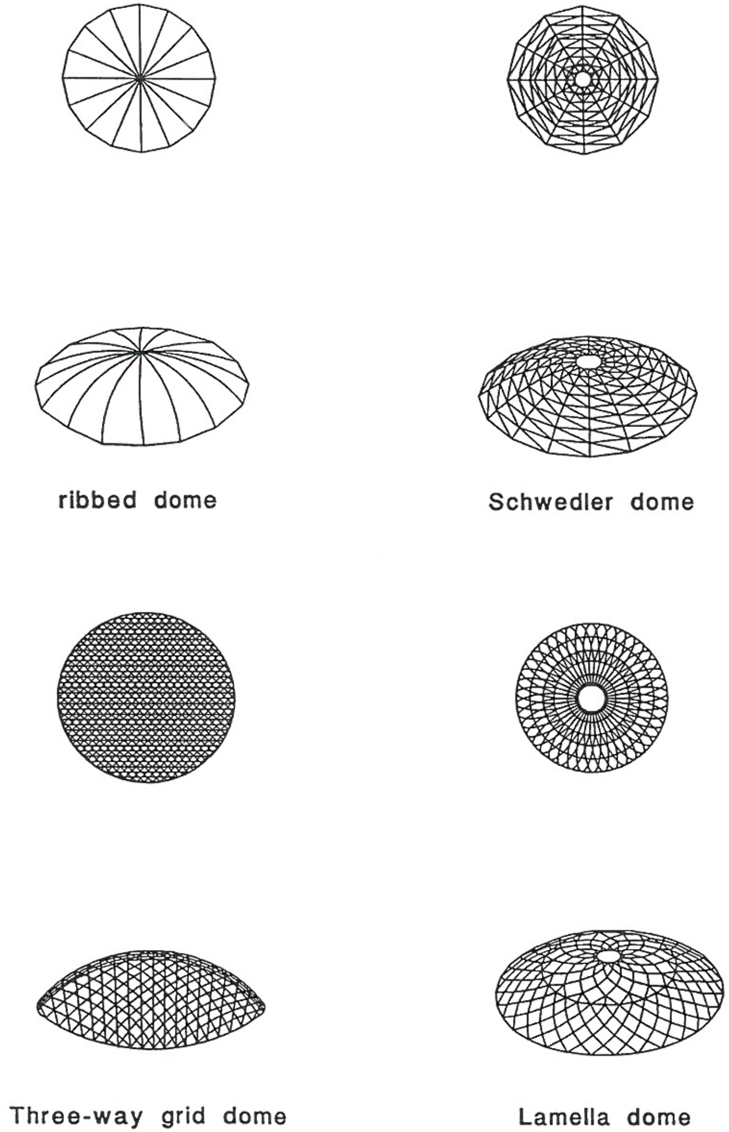


Figure 11 Dome configurations

From the physical point of view, it is important to stress the difference in behaviour of domes with respect to cylindrical vaults. The sensitivity to asymmetrical actions and the resistance to global buckling phenomena is strictly connected to the rigidity of the geometrical shape. For cylindrical vaults, their surface has a single curvature, so it can be developed on plan. In contrast, domes, having a double curvature, resist any actions by virtue of the shape itself.

Definition of the geometric arrangement of elements, whether for a single or a double layer construction, is a difficult problem to resolve. Research is aimed at using fewer different lengths for the members. Moreover, it is important to check that the polygons defined are as similar as possible, in order to facilitate cladding. Fuller designed one of the first big developments: the dome of the American Pavilion in Montreal (1967) which is composed of two interconnected layers and constructed with welded tubular sections; the resulting structure is 5/8 of a total sphere, with a diameter of 75m.

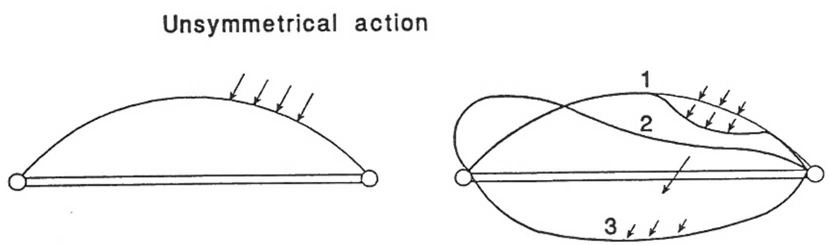


Figure 12 Snap through buckling for domes

Lecture 14.5

2. DESIGN OF SPATIAL TRUSS SYSTEMS

2.1 Conceptual design

At the conceptual design stage it is important to define the geometry of the structure, given that determining support positions is an important factor in the strength and rigidity of the system.

Geometric design parameters are:

- the overall shape; flat shape or assembly of small flat shapes (so called "polyhedric surfaces"), curved surfaces (generally positive double curved surfaces; one of the curvatures may be equal to zero, i.e. the case of cylindrical vaults). For curved surfaces, a rise/span ratio should be fixed to satisfy both mechanical and architectural criteria, e.g. in order to avoid domes which are too shallow.
- geometry of the cladding supports.
- the number of layers; structural depth/span ratio influences the weight, strength and cost.
- frequency of mesh, i.e. number of geometric elements for a given length.

The choices made directly influence the number of members converging on each node and on the connecting angles between these members; these two parameters determine the feasibility of the nodes. Too many elements meeting at different angles and lack of repetition are hindrances to construction efficiency. The choice of spacing of grids should be related to geometric connections, particularly the connection of the two-dimensional sides of a polyhedric surface.

Choice of support conditions does not pose any specific problems, but it does affect behaviour.

It should be noted that, because of the low weight of these structures, the weight of the equipment supported by the structure should not be negligible. Similarly, actions resulting from the method of construction should be carefully examined. The effects of concentrated actions and of partially distributed loads, which are greater when the total loading is not symmetrically distributed, should be examined. Except in specific cases, dynamic actions can be replaced by enhanced static actions.

2.2 Design Method

Every project is undoubtedly an individual case. Nevertheless, it is possible to establish sequential steps in the development of the design. Most importantly it should be noted that, for a shape to be covered in any pre-determined way, two distinct methods can be used to define the general surface:

- either the overall surface geometry is defined a priori: a geometric division must then be made, e.g. a geodesic division for domes.

- or the generating module is laid as before and multiplication of this module provides the final geometry.

Once the overall geometry has been established, the designer must decide on the number of layers. This design depends basically on the free span, and also on the geometric distribution of the members in and between the layers. Choice of frequency of mesh is important for reasons of resistance and cost, as well as for aesthetic reasons. Choice in the geometry of the network of members directly influences behaviour of the systems. For example, in the case of double layer grids, an examination of different geometric arrangements has confirmed the importance of adopting an arrangement where directions of the members in the two layers are set at 45°.

It is therefore possible to examine the structural behaviour under appropriate combinations of actions. Choice of support conditions has a big influence on the distribution of internal forces and size of the deflections. The possible use of multi-point supports is an important advantage of spatial trusses. Definition of the areas of the cross-sections of the elements may lead to a process of optimisation suitable for the design model.

2.3 Initial Sizing

It is possible to provide further information in the case of double layer two dimensional grids. The curves in Figure 13 illustrate the variation in weight suitable for different structural depths with the slenderness ratio for seven different geometries (Figures 14-20). Calculations were made for the case of a square outline with peripheral supports under each node, assuming a uniformly distributed load. In addition, the following data have been assumed:

- take a depth/span ratio of 1:15 in relation to the free span where there is a working load of 1,50 kN/m^2;.

- consider a self weight of about 0,15 to 0,20 kg/m^2 for spans up to 30m.

It is also important to consider the relation between the size of the mesh element and depth of the grid.

2.4 Choice of the Structural System

As soon as the overall shape of the structure is defined, it is necessary to choose the structural system.

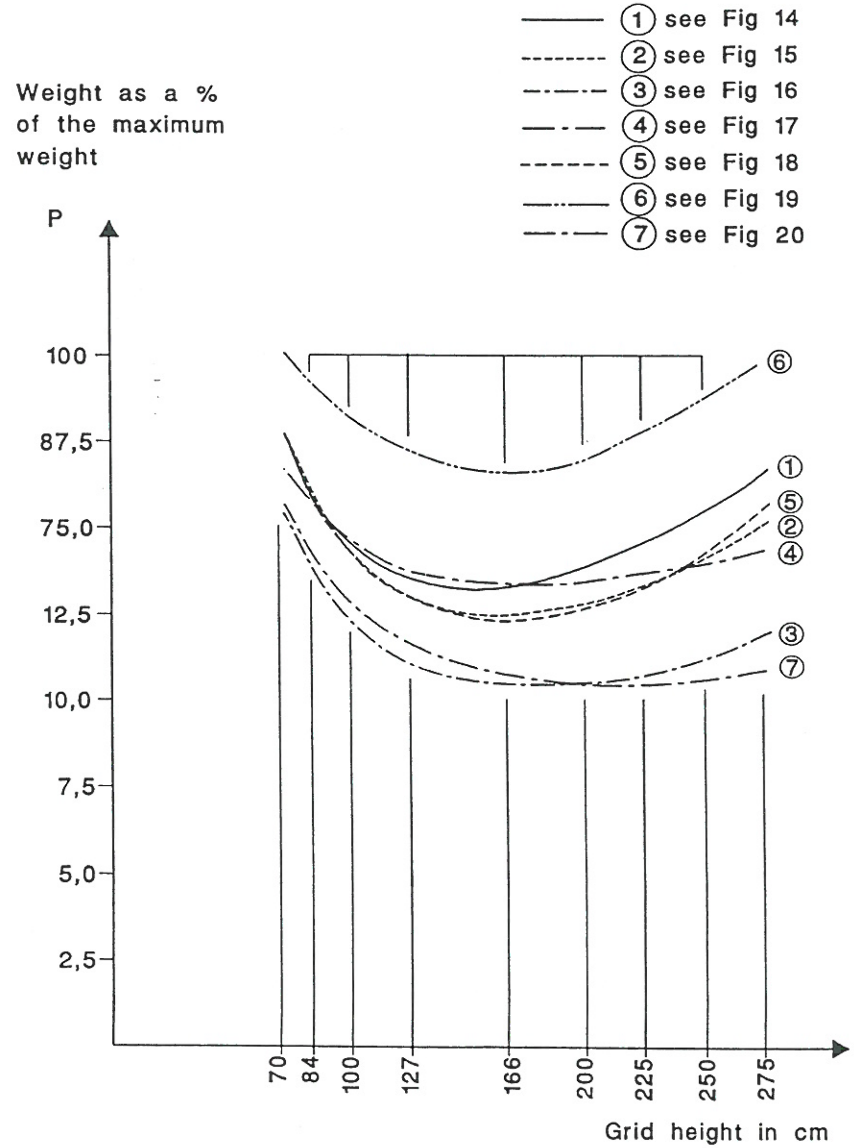


Figure 13 Comparison in terms of weight for different configurations

A wide variety of structural systems is available, which can fulfil the geometrical requirements of the design. However, they cannot all provide the load carrying capacity which is required to resist the most unfavourable design loading conditions.

The sizing of the cross-section of members is a task of the designer, whereas the producer of the structural system must guarantee that the node-member combination should be able to give a joint of full strength type. Only by means of appropriate qualification procedures can this guarantee be realised.

2.5 Qualification Procedure

The qualification procedure should demonstrate that the system for the space structure is based on a basic node - member joint which is a full strength type. The demonstration can be done by calculation and by tests.

On the one hand, it is possible to model the node-member joint by means of finite element techniques. However, it is not prudent to rely solely on these numerical results.

On the other hand, only experimental evidence can fully demonstrate the actual behaviour of the system.

For these reasons, the qualification procedure should be based mainly on laboratory tests, the results validating the calculations. A suggested procedure could be based on the following phases:

- monoaxial tests on the node;
- monoaxial tests on the member-to-node connection;
- bending and shear tests on full-scale structural units;
- bending tests, monotonic and cyclic, in the elastic range (including temperature effects) up to collapse on full-scale prototypes.

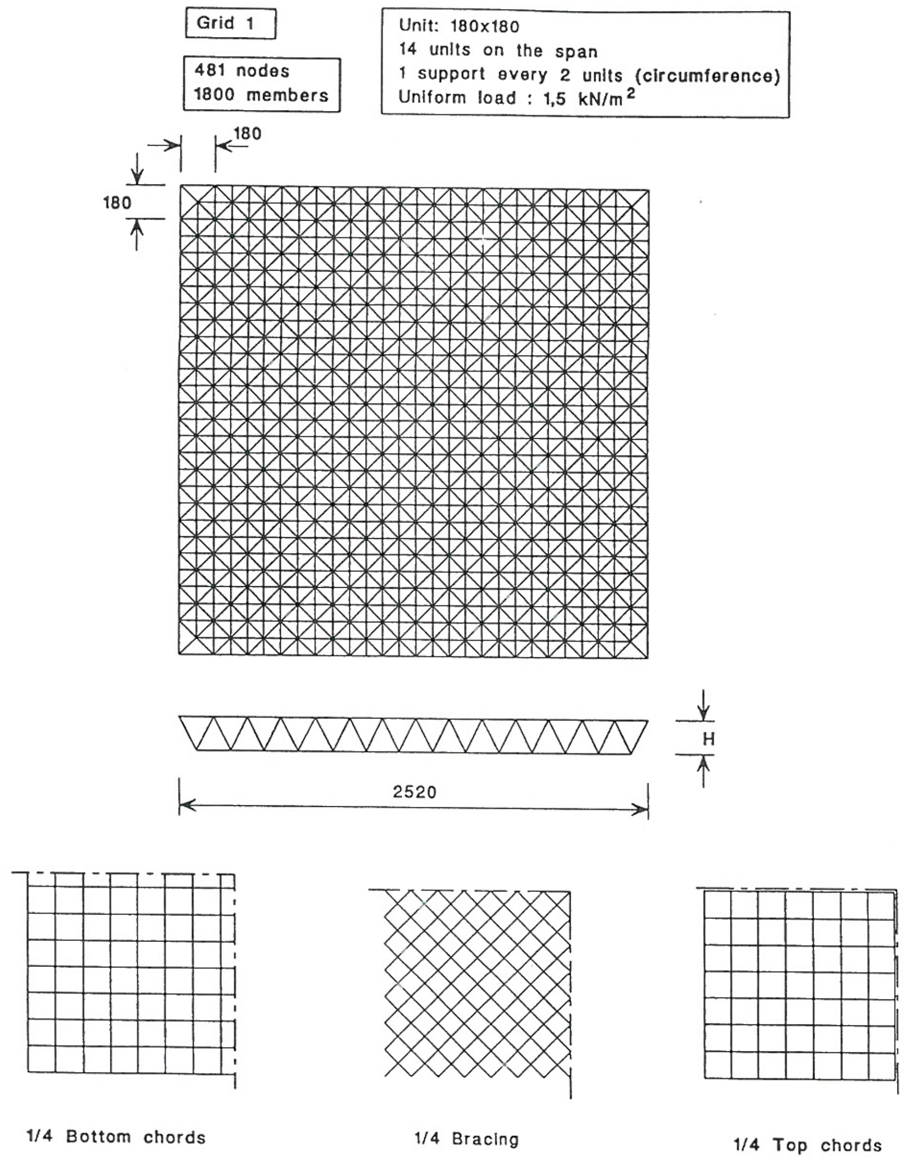


Figure 14

3 ANALYSIS OF SPACE TRUSS SYSTEMS

3.1 Different Analysis Methods

The objective of an analysis of truss systems is to determine the values of the variables necessary for sizing purposes and for those required to size their supports. The variables generally required may be:

- compressive and tensile forces in the members in the system.

- node displacements.

- values of support reactions.

The study has to be made for several cases of actions and combinations of actions. The most unfavourable cases are used as the basis for design.

Depending on the type of problem being examined, it is not necessary to determine all unknown quantities. The methods of analysis available can be classified as follows:

i. **Method of joints**: applicable to two-dimensional systems which are internally isostatic (Figure 21), for which the reactions have previously been determined. It is possible to determine the forces in all members. They can be determined by analytical or graphical methods (see Figures 22 and 23).

ii. **Method of sections**: in certain cases, this method can give direct access to internal forces in a limited number of members selected by the designer. It is used, for example, for preliminary design work in order to assess the maximum forces in a triangulated system (Figure 24).

iii. **Displacement method**: This is the most general method. It is applicable to two-dimensional and spatial systems, isostatic or hyperstatic. It gives values for all unknown quantities: internal forces, displacements, reactions.

3.2 Design Assumptions

In addition to use of principles which are common to all structures in respect of actions and their combinations, Eurocode 3 [2] allows a perfectly pin-jointed model for global analysis.

Furthermore, it is considered that all actions are applied at the systems nodes. It is sufficient to assume linear-elastic response.

Calculations in the elasto-plastic range can be done by using simplified models of the members such as is shown in Figure 25. The results show that a

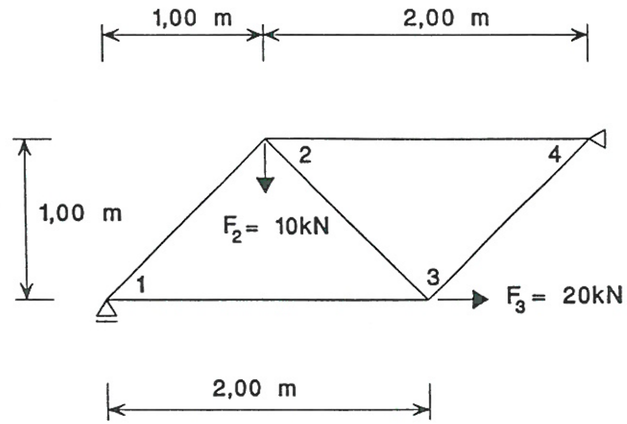


Figure 21 Two-dimensional schedule

Lecture 14.5

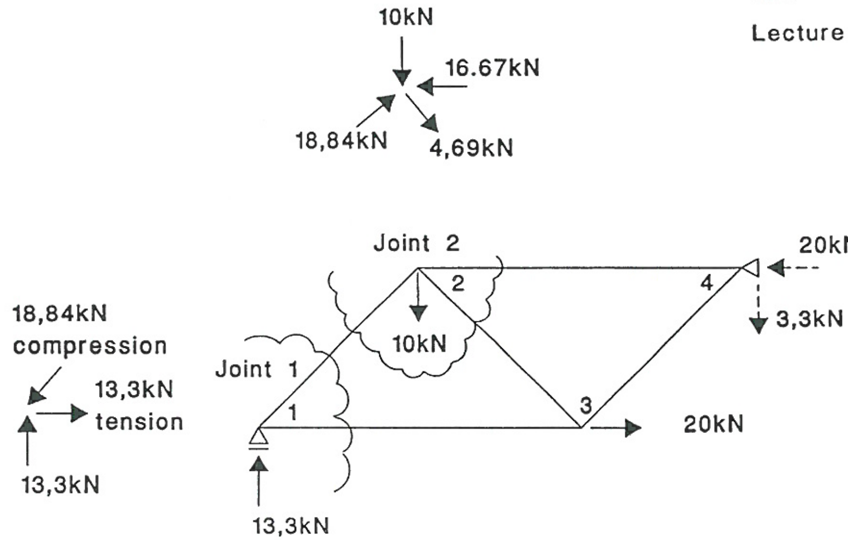


Figure 22 Method of joints - analytical

Lecture 14.5

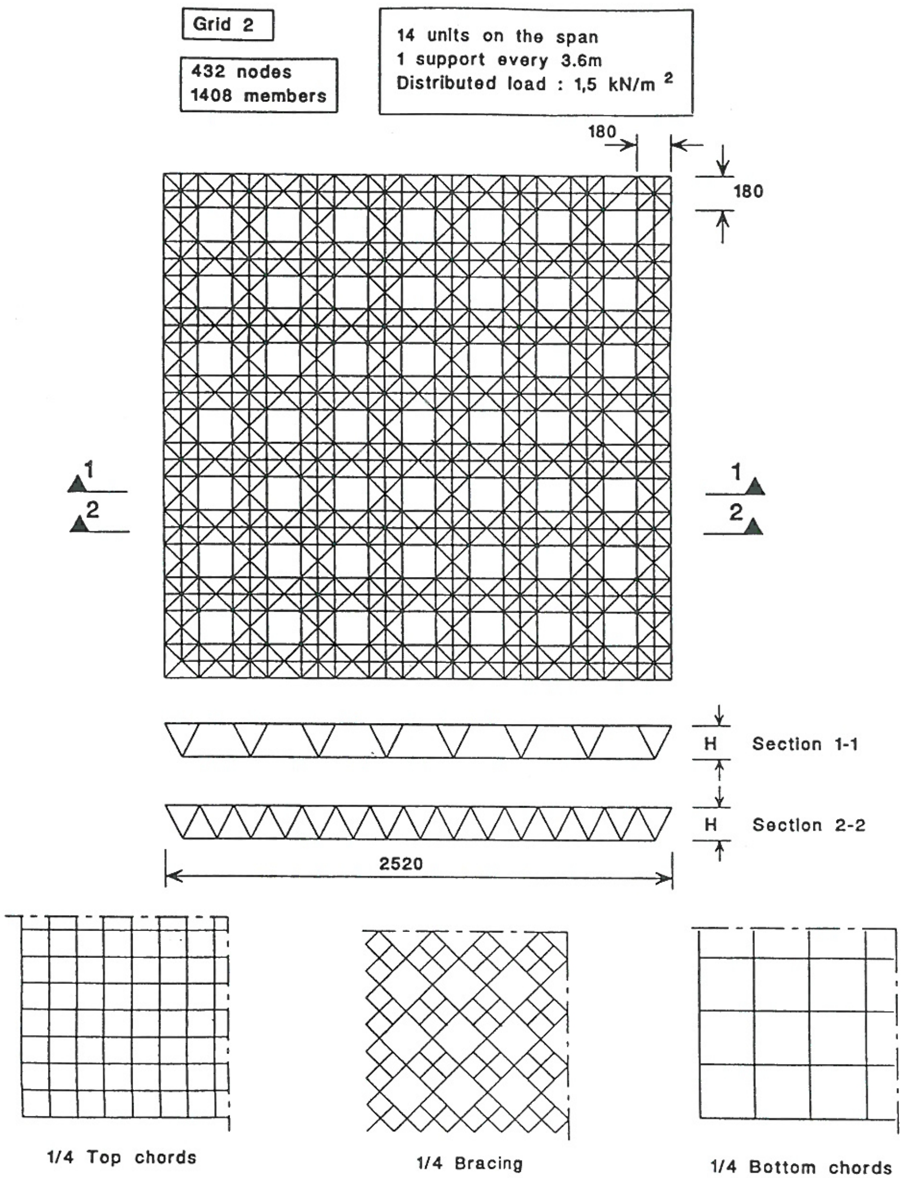


Figure 15

Lecture 14.5

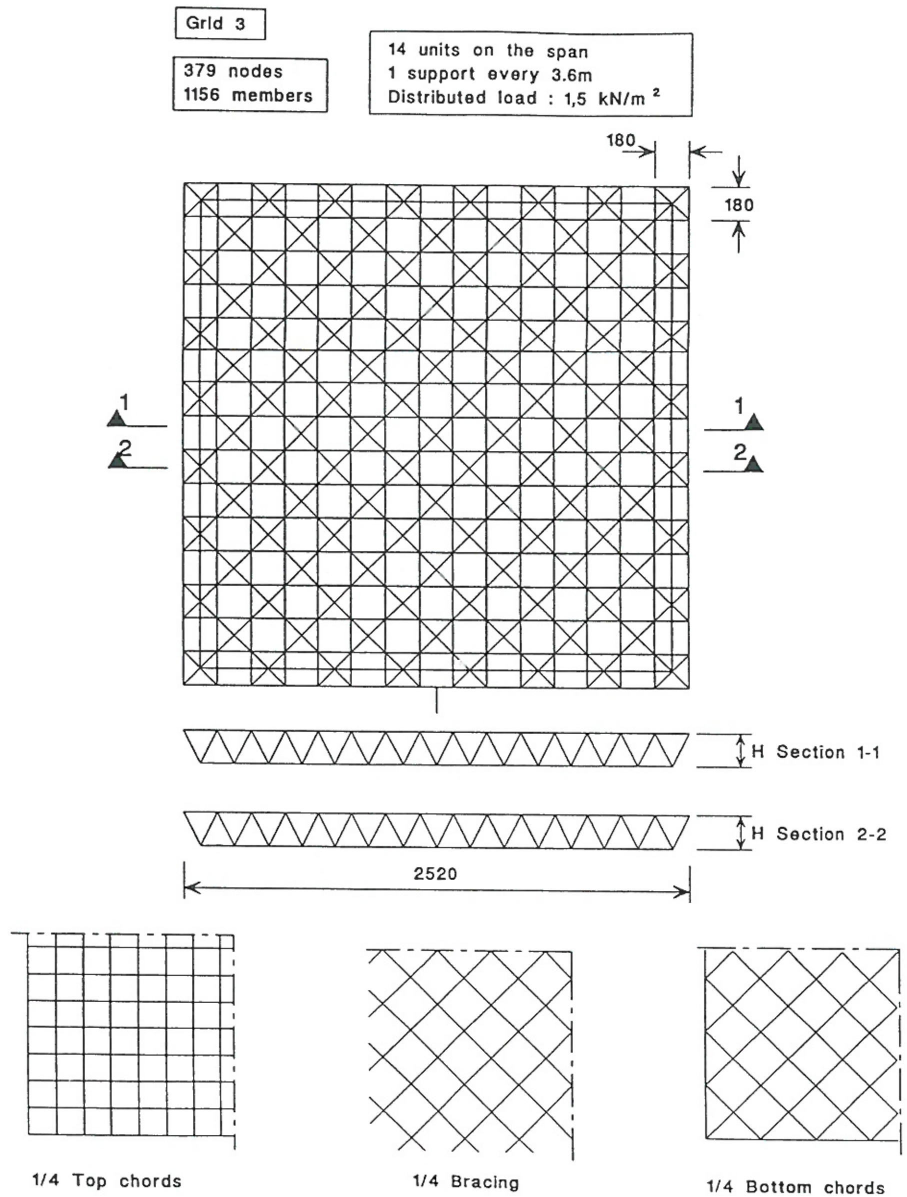


Figure 16

Lecture 14.5

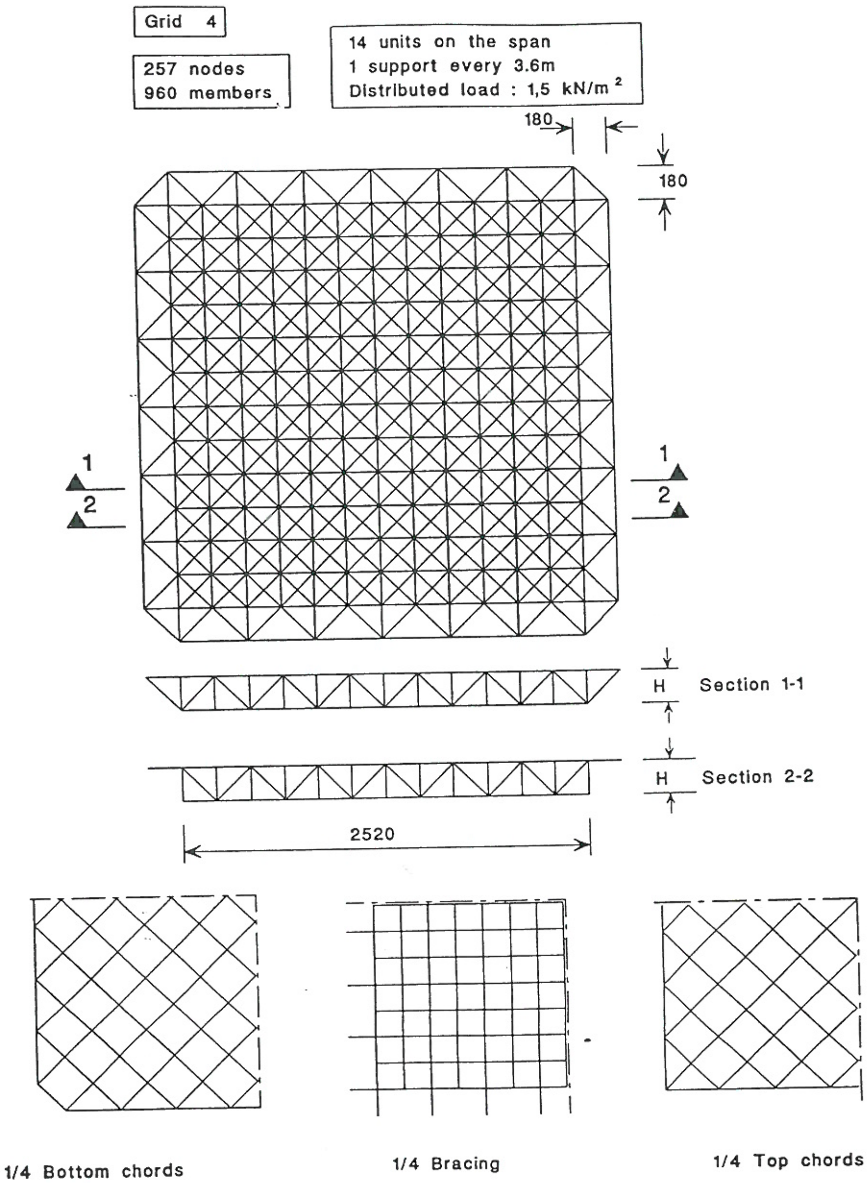


Figure 17

Lecture 14

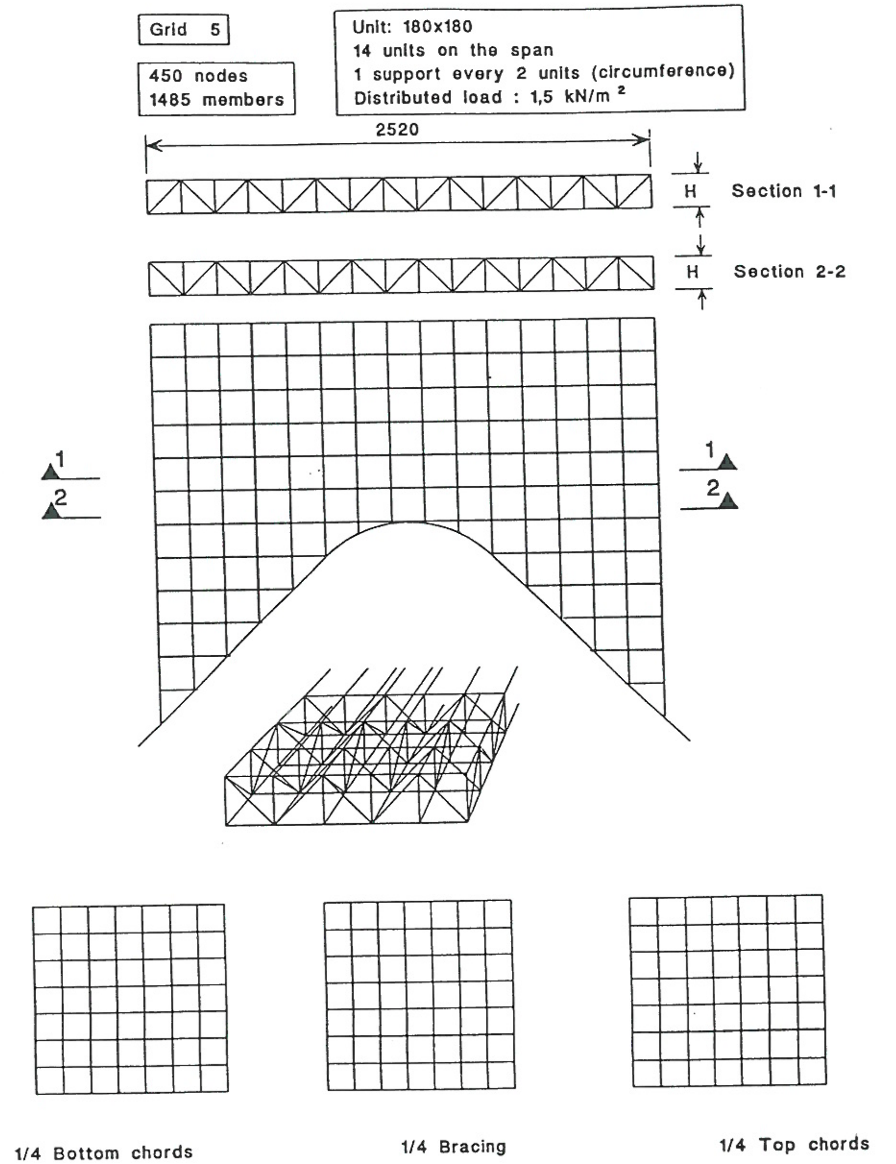


Figure 18

Lecture 14.5

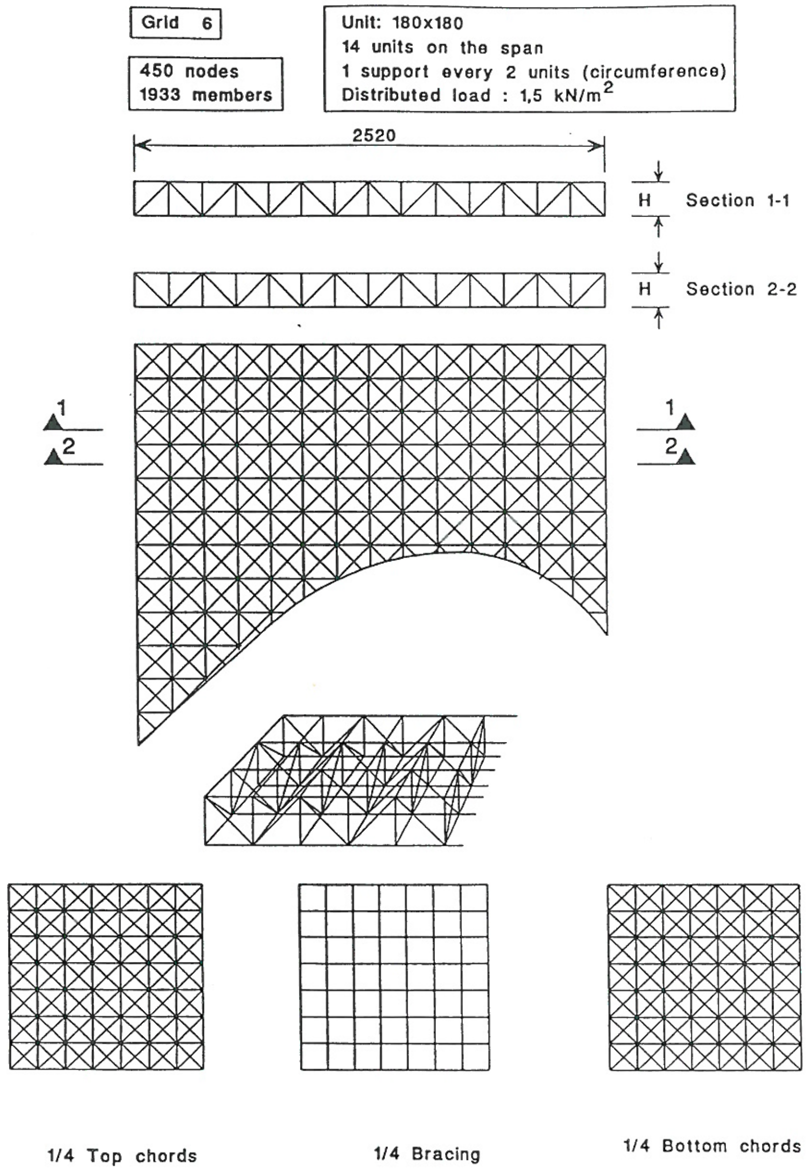


Figure 19

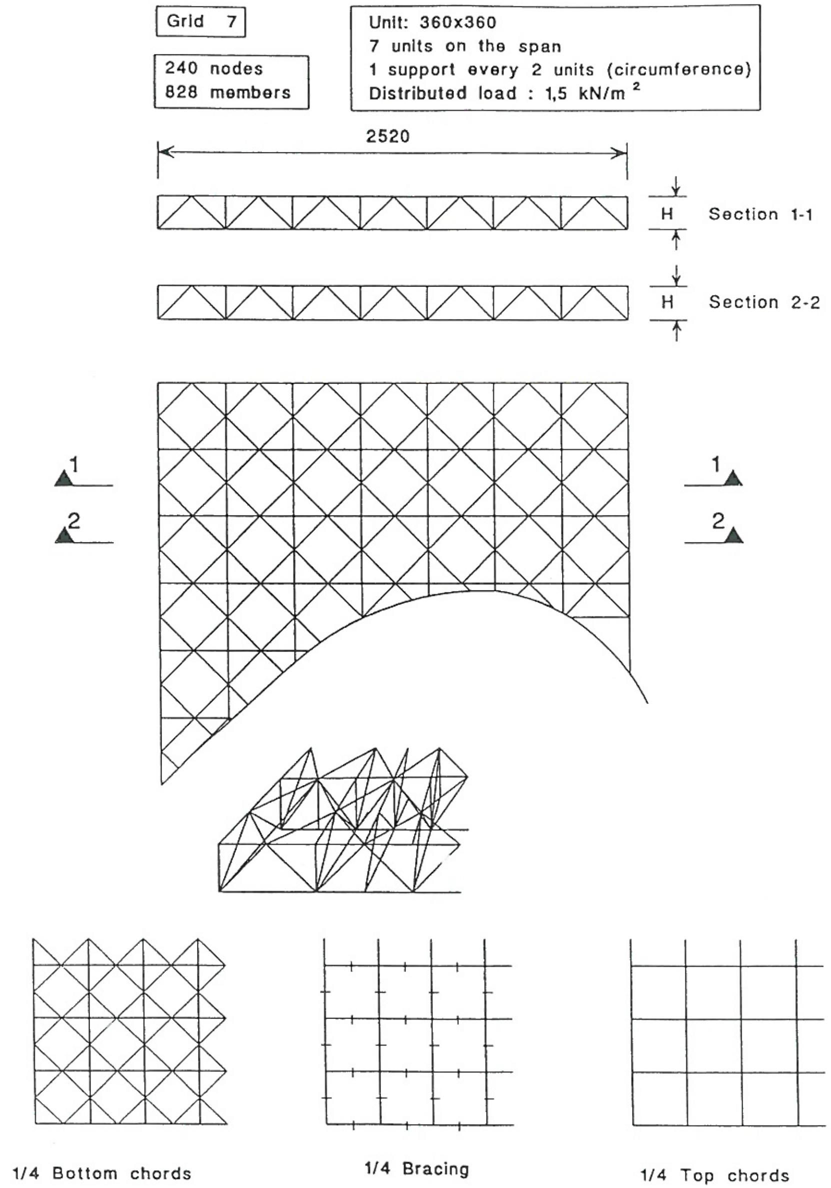


Figure 20

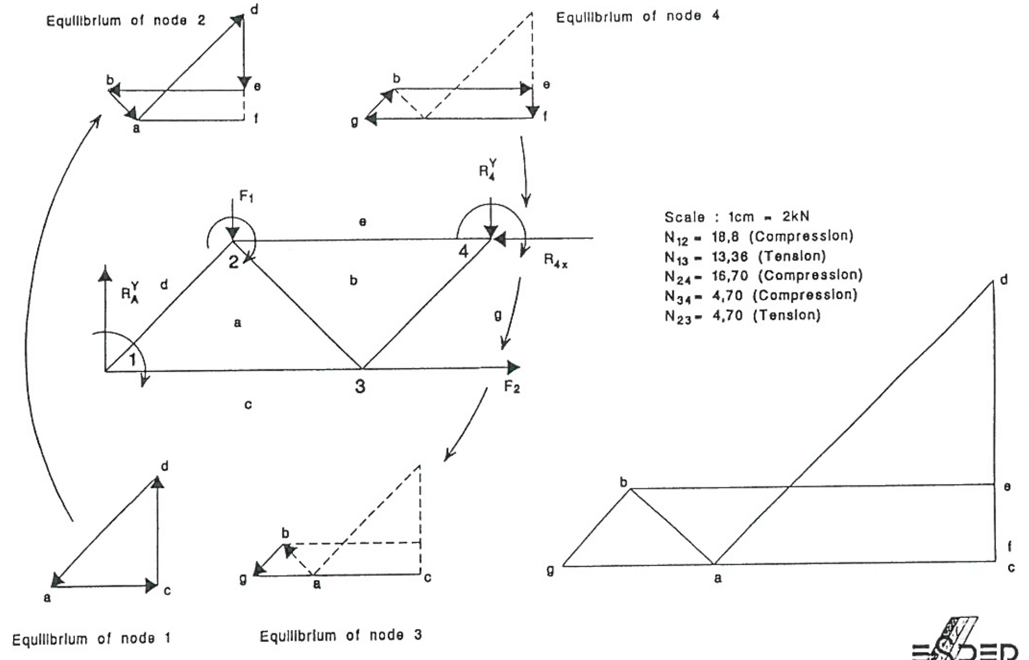


Figure 23 Method of joints - graphical

Lecture 14.5

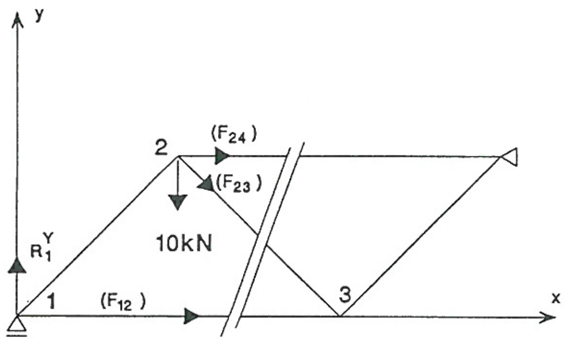


Figure 24 Method of sections

Lecture 14.5

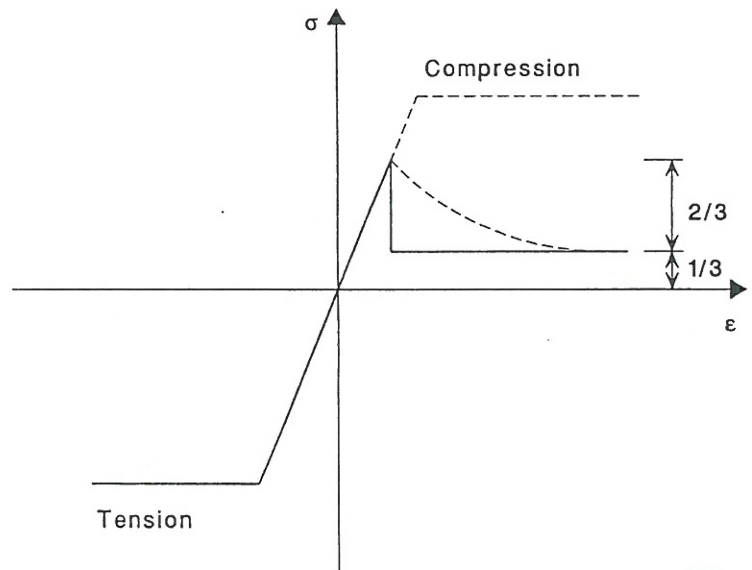


Figure 25 Member behaviour

Lecture 14.5

reasonably stable response of compression elements (after the ultimate load of a member is reached) can be achieved and the forces transferred to the adjacent members from that which has reached its ultimate resistance.

The methods described in the following paragraphs are all based on the application of equilibrium to the forces applied by the members to the nodes. The pin-jointed analysis used for triangulated systems simplifies the formulae for equilibrium: in general, the three equations relating to translational equilibrium are necessary.

3.3 Limit of Validity of the Methods Described

Attention is drawn to the fact that the methods described do not take account of:

- any non-coincidence of the axes of the members forming a node.
- flexure induced by the application of external actions but along the length of members, rather than at nodes.
- secondary bending moments due to the effective rigidity of the joints which no longer corresponds with the assumption of pin-jointed behaviour.
- non-linearity due to geometry and/or to material.

It is therefore necessary to evaluate carefully the significance of factors associated with failure to ensure the appropriateness of the assumed models, either by making additional calculations, or if applicable, by using more detailed calculation models, e.g. the generalised displacements model which takes account of stiffness in flexure and in torsion, when the given joint is far from the pin-jointed assumption.

3.4 Displacement Method

This is the most general method applicable in all cases of space structures. The behaviour of the materials is assumed to be elastic and linear.

The principle of the method lies in resolving a system of linear equilibrium equations as follows:

$$[K]\ \{D\} = \{F\}$$

where

[K] is the structure stiffness matrix.
{D} is the unknown displacement vector.
{F} is the vector of known actions.

The components of {D} corresponding to the fixed supports are zero. The corresponding equations, the second components of which are the reactions, do not therefore appear in the corresponding system of linear equations. Determination of the displacements enables the internal forces to be calculated. This method is normally implemented by computer software.

4. FABRICATION OF SPACE TRUSSES

4.1 Introduction

The very nature of spatial trusses encourages research into maximum standardisation, linked to individual component manufacture, and requires particular attention to problems of precision.

It is advantageous to design spatial trusses with a minimum number of different members; the same criteria should be used for the nodes. It is common to use members with the same section sizes, independent of their different stress state due to their location in the structure. For tubular sections, it seems reasonable, however, to keep the same external diameter and to vary the wall thickness.

Installation methods may cause greater deformations and forces during erection than after completion. The designer should consider the erection phases when sizing the elements.

4.2 The Structural System

The structural system is characterized by the combination of three main components:

- member
- node
- connection

Members

Hollow sections are essential for a number of reasons. In particular, tubular sections are generally used because of their large and uniform radius of gyration.

Nodes

The dream of a 'universal' node has not yet been realised. Several parameters govern the design of nodes. Nodes can be connected mainly by welding, bolting or by special fabrication. Except where pretensioned bolts are used, bolted connections reduce the resistance of net-sections. Some authorities prefer welding for large spans, even if it is difficult to guarantee the quality of welds in site. One of the determining factors in the choice of nodes is the number of members to be assembled. Apart from structural influences on the node itself, this problem is linked to the way the members are connected to the node and to considerations of space and ease of installation. The regularity of geometry resulting from the node determines the entire geometry of the structure. Significant progress has been made in this area through computerisation linking design and fabrication. As a result, it is possible to fabricate nodes by varying the angles of incidence of the members.

Five 'kinds' of nodes may be identified, Figure 26:

- plate nodes (Figure 26a)
- folded nodes (Figure 26b)
- cast nodes (Figure 26c)
- nodes of extruded aluminium section (Figure 26d)
- special connections with spherical nodes (Figure 26e)

Plated and folded nodes are usually connected to the member ends by means of bolted connections, but also welding can be done.

The node is a critical element when evaluating the cost of spatial structures: one node for 2,5-3,0m^2 would seem to be an economical solution.

Connections

The node-to-member joining system determines how the ends of the members must be treated. Five processes may be described, for example, (Figure 27):

- Straight cutting (Figure 27b)
- Profiled cutting (Figure 27c)
- Squashed and drilled (Figure 27d)
- Addition of a connection plate (Figure 27e)
- Special fixing: threading, welding, or bolt crushing (Figure 27a)

It is clear that not all these systems give full strength joints.

4.3 Methods of Fabrication and Erection

Methods used may be listed in three categories:

a. Erection of separate members, each one lifted into position and connected to the work already assembled.

b. Erection of sub-assemblies: this is an intermediate stage whereby the members are connected in sub-assemblies, either in the factory or on site. The sub-assemblies are lifted into final position and connected to the work already assembled.

c. Lifting of the whole space structure, which is assembled on the ground on site. Various methods may be considered ranging from the use of vertical construction parts as lifting masts to cranes.

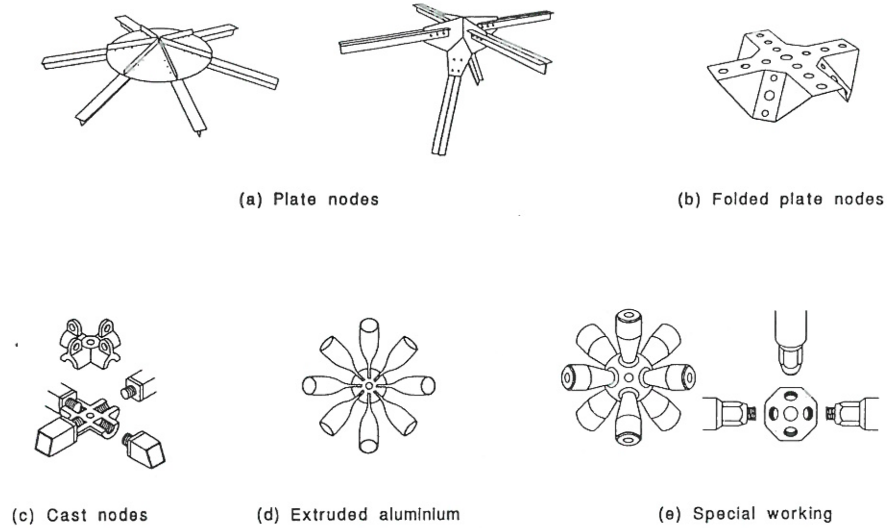


Figure 26 Different types of nodes

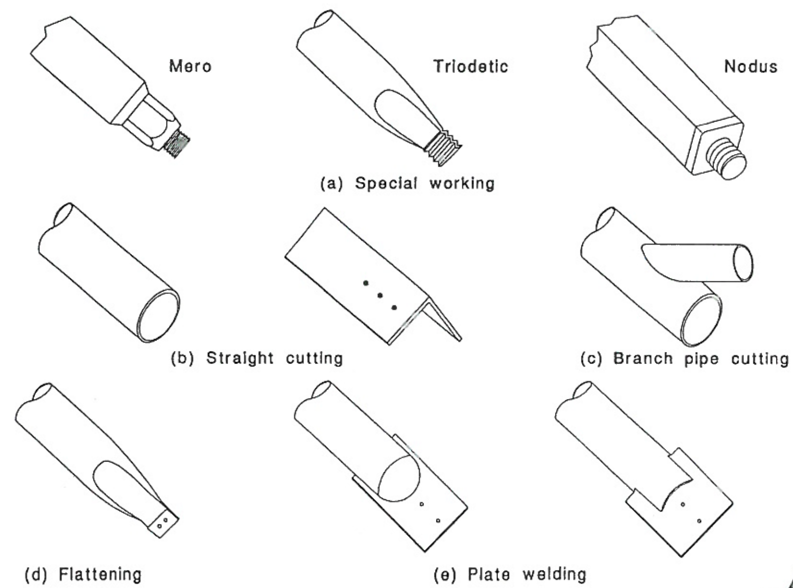


Figure 27 Different treatments of member ends

The choice of one of these three methods depends on:

- the nature of the project in terms of type of structure and size.
- operational conditions: actual layout of the site, available means of lifting, transport costs, experience, etc.
- safety.

In a) and b) it is essential to predict the need for any temporary supports that may be necessary where the structure achieves stability only when it is complete. The many phases in erection should be carefully examined so as to avoid intermediate structural behaviour which is less favourable than that for the final state of the structure.

Lifting the whole space frame has the following advantages:

- the greater part of the work is carried out on the ground, thus aiding control of the operation, especially the making of welded joints.
- the use of heavy hoisting machinery is required for a shorter period, which may reduce final costs.
- in some cases, the structure with other equipment attached may be lifted together.

Hoisting is a critical stage in erection. Lifting points should be carefully examined. Lifting should be carried out in the best meteorological conditions and certainly not in wind. Once in position, the structure should be connected to the work already erected. Precise regulating devices should be planned in advance in order to facilitate connection and fixing. The lifting stage can be a determining factor in the design of the structure.

A new approach has been used in Barcelona for the erection of the dome of the Olympic Palace. It involves the fabrication of the dome on the ground in five portions which are temporarily pinned to each other (Figure 28). The central portion is then lifted and the remaining segments of the dome locked in the final position.

Different methods of execution may be considered depending on the type of structure and place of installation.

For example, it is possible to carry out assembly using a launching method borrowed from bridge construction, etc. There is no limit to the list of solutions. The erection method chosen depends on the imagination and know-how of the designer within a particular context.

5. CONCLUDING SUMMARY

- The lecture has been concerned with space roof structures in which members are subject to internal axial forces.
- The structures may take the form of:
  - two-dimensional grids.
  - cylindrical vaults.
  - domes.
- Unsymmetrical patterns of loading must be considered, including combinations of actions that may arise during erection.
- The displacement method of analysis, implemented by computer software, is the most convenient approach to determine internal forces, displacements of nodes and support reactions.
- Repetition of nodes and members permits the use of standard components and reduces the costs associated with design, detailing and fabrication.

Assembly on the ground with temporarily pinned joints

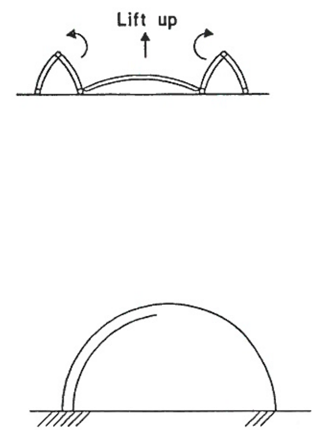


Figure 28 Novel method of dome erection

Lecture 1

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ESDEP WG 14
STRUCTURAL SYSTEMS: BUILDINGS

Lecture 14.6: Special Single Storey Structures

OBJECTIVE/SCOPE

To describe less known forms of single storey steel structures, including cable and tension structures.

PREREQUISITES

None

RELATED LECTURES

Lecture 14.5:	Space Structure Systems
Lecture 15C.4:	Guyed Masts

SUMMARY

The lecture examines some less-common structural systems, not described earlier. Variants on standard forms used for industrial buildings are briefly described. Cable-stayed and tension structures are described in more detail.

1. INTRODUCTION

1.1 General

Most single storey structures are portal or trussed frames, Figure 1. These structural forms have been shown to provide economic and effective solutions for routine construction. However, alongside these commonplace forms a wide range of single storey structures have been developed for special applications.

Although the number of special structures is relatively small, they encompass a wide range of structural forms. Many of these structures result from the need to cover large areas, typically for sports, exhibition, industrial or commercial purposes. As such, they are characterised by large uninterrupted spans and relatively light imposed loading. They are frequently lightweight, reflecting their designers' intention to maximise structural efficiency. Structural considerations are, therefore, to the fore in determining their architectural form.

The lightness of these structures requires close attention to be paid to aspects of structural action and response that are not significant for more conventional structures. Attention has to be given to dynamic and fatigue effects, to load reversal and to uplift. In this latter regard, elements primarily designed for tension may need to be braced to resist compressive forces, and elements may require pretensioning to ensure that stiffness is maintained under reversible or dynamic loading. Members and connections may require detailing for fatigue. Temperature effects and second order effects relating to cable extension or relaxation may also need to be considered.

It is convenient to categorise these forms in terms of their structural systems. These include direct-force systems and mixed systems combining flexural elements with major direct-force components, in addition to systems which exploit the potential of curved or folded forms. In some cases, the foundations play a more complex role than usual in equilibrating the force system and in limiting deformations.

It is proposed in this lecture to outline some of the more common systems encountered. Additional material on some of these topics will be found in Lecture 14.5 on Space Frames, and Lecture 15C.4 on Guyed Masts and Towers. The slides on special single-storey structures provide a wide range of examples of the application of the principles outlined in this Lecture.

The Lecture commenced with a brief review of some non standard approaches which have been used for industrial buildings in the past - the saw tooth roof, the umbrella and butterfly roof and the arch roof. The dome is also reviewed.

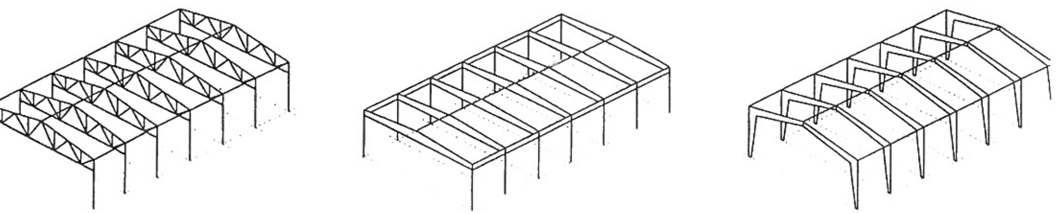


Figure 1 Truss & column and portal frame system

1.2 Safety

With large-span structures, the consequences of failure are more likely to be catastrophic than is the case for normal structures and safety therefore merits particularly careful consideration at all stages of the life of the structure. The more severe consequences of failure can be attributed partly to the large scale and partly to the lack of structural redundancy encountered in many of the systems adopted for large-span structures. Increased redundancy offers the prospect of greater stiffness and strength, and the alternative load paths associated with redundancy offer a reduced likelihood of total collapse. Against this must be balanced the capacity of statically determinate structures to accommodate secondary effects (shrinkage, creep, relaxation, temperature changes, settlement, lack of fit, etc.) without distress.

Special structures require critical assessment of the applicability of standard loading codes, particularly in regard to wind. Construction techniques, materials and detailing may be non-conventional and introduce additional uncertainties. Non-linear effects - relating to material or geometrical factors - may have a bearing on the behaviour and detailing of the structure. Inspection during construction, and periodic inspection and maintenance of the structure in service, assume added importance.

2. AN OUTLINE OF OLDER TYPES OF SPECIAL SINGLE STOREY STRUCTURES

Some of the more interesting types of older, single-storey steel structures are briefly described below. These types have been used extensively in the past, and although their use today is rather limited because of the development of other systems, they still have a significant interest for the designer. In certain cases they can still provide satisfactory solutions from the viewpoints of functionality, economy and aesthetics.

2.1 The Saw-tooth Roof

This structural system (Figure 2) was frequently used in the past, mainly for industrial buildings. Its use is now rather limited because its main advantage of uniform daylighting into the building, is achieved by new roofing products that can provide daylighting efficiently through flat roofs, or by artificial lighting. The saw-tooth roof is costly to construct, wasteful of heat, and requires many internal gutters.

2.2 The Umbrella and Butterfly Roofs

These two structural systems (Figure 3) are conceptually very similar to the saw-tooth roof, and they are used to cover large floor areas with a minimum of internal columns. It is still common to use this construction in portal frames with, for example, alternate valley columns omitted (Figure 4).

2.3 Arched Roofs

Arches have long been recognized as being highly efficient and economic structural systems for covering large spans in buildings (Figure 5).

The main problem of an arch is the thrust developed at its supports. The thrust can be of considerable magnitude for long spans. It increases as the ratio of rise to span becomes smaller and can be resisted only by appropriate foundations, or by an arch tie.

Portal frames benefit from arching action, which has led to the use of tied frames (Figure 6). Care has to be taken to design the rafter against the considerable compressive force that arises. Also the tie member must not buckle if wind uplift causes reversal of force in this member.

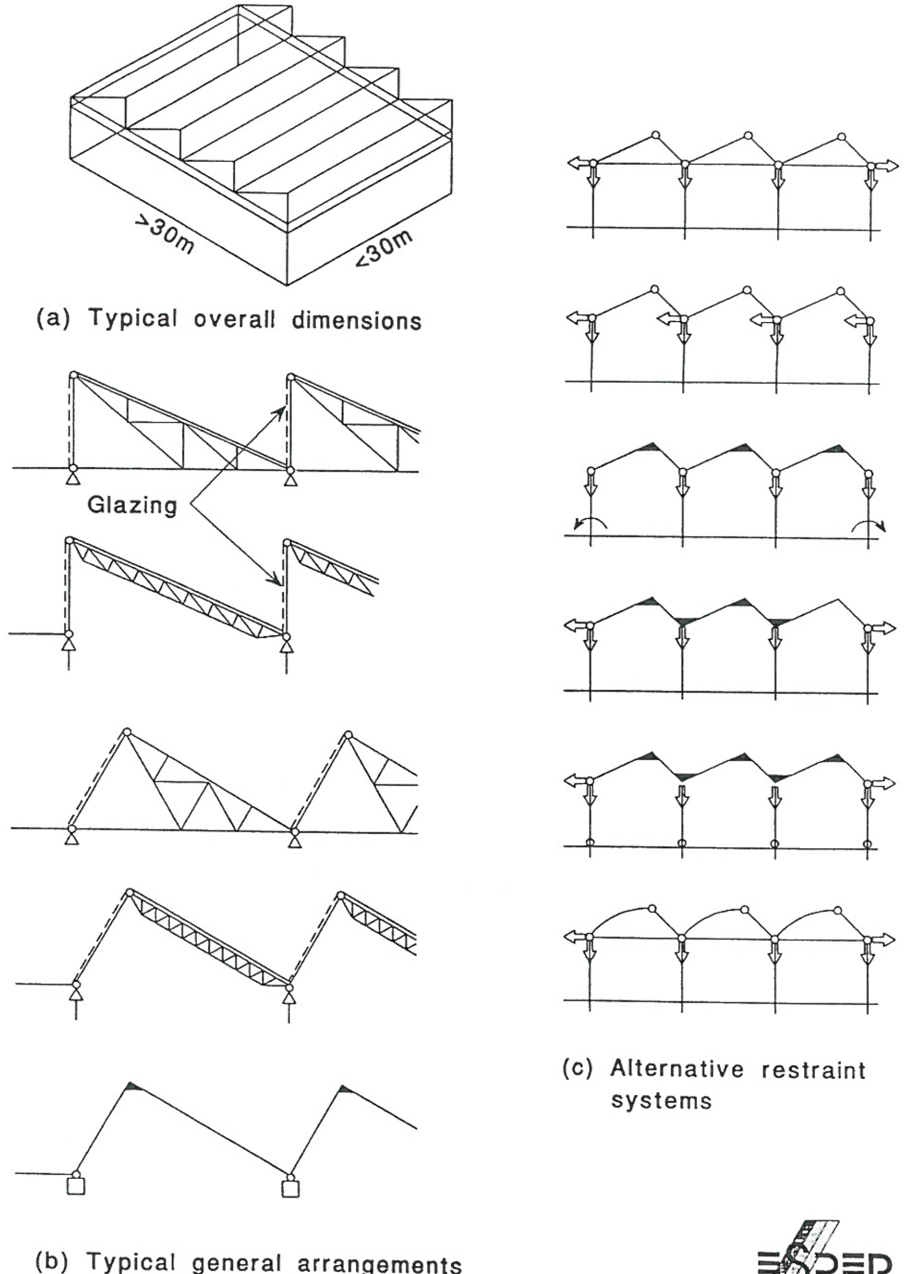


Figure 2 Saw-tooth roof system

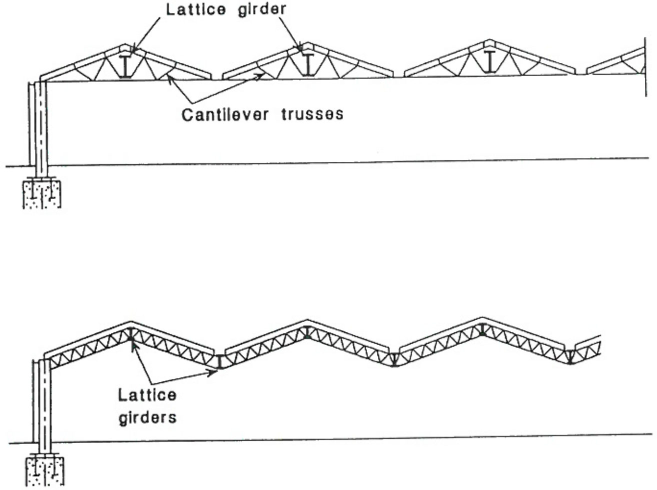


Figure 3 Umbrella and butterfly roof systems

Lecture 14.6

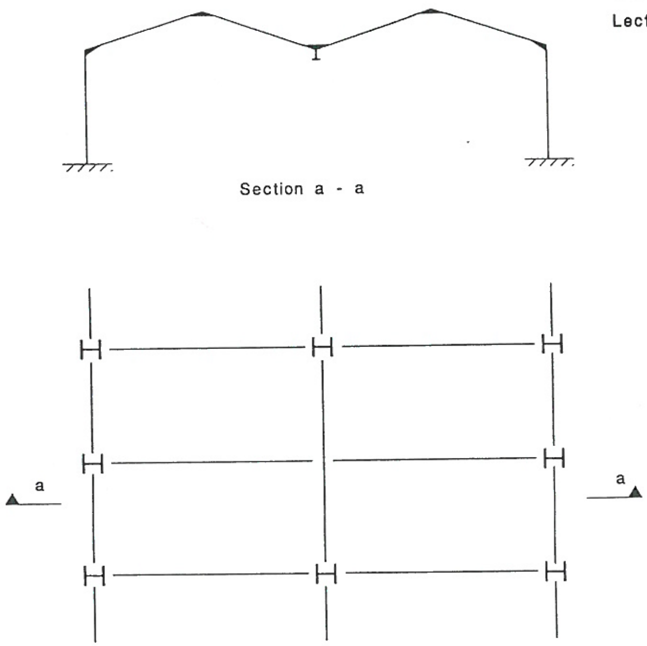


Figure 4 Portal frame with alternative valley columns omitted

Lecture 14.6

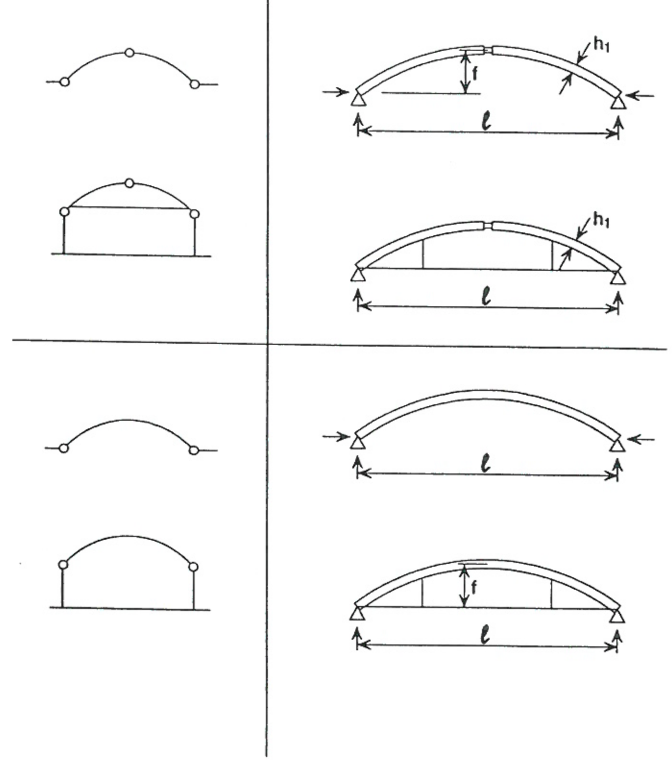


Figure 5 Arched roof systems

Lecture 14.6

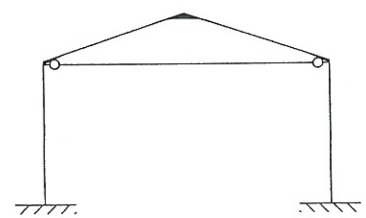


Figure 6 Tied portal frame

Lecture 14.7

2.4 Prestressed Frames

Prestressing provides an option, if uncommon, for creating a more favourable regime of bending moments in large span frames, as well as for deflection control (Figure 7).

The additional compressive forces must obviously be taken into consideration in determining the bracing requirements of members.

2.5 Domes

Domes are three-dimensional structures used to cover buildings with a circular floor plan. Their cross-section may be spherical, elliptical, parabolic, etc. Steel framed domes have been extensively used for public assembly buildings such as auditoria, arenas, gymnasia, exhibition halls, etc.

There are many types of dome (Figure 8). A common type is the ribbed spherical dome which consists of main meridional ribs and horizontal (parallel) rings. The rings decrease in diameter going from the base to the top of the dome, concluding with the compression ring at the top. This arrangement, whereby the ribs are stopped at the compression ring instead of being continuous over the pole, is used practically in all dome designs. It has three advantages. Firstly, it makes possible the connection of all ribs converging to the same point. Secondly, a large opening is formed which can be used for lighting and ventilation. Thirdly, the structure becomes statically determinate.

Apart from the compression ring at the top, the primary system of ribs and rings, may be subjected to further modifications. For example, auxiliary members, such as purlins and diagonal bracings may be added in the plane of the roof surface. The bracings are usually placed in alternate bays. They are an important aid during erection, which is usually done by using only one scaffold tower at the centre of the dome.

Another modification concerns the connections between the ribs and rings. Theoretically they should be pinned but in practice they are moment resisting. Finally, the geometry of the dome is usually modified, 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.

This type of dome is essentially a direct force structure, its individual members being subjected primarily to axial tension or compression (membrane action).

For symmetrical loading, the global analysis can be made by simple statics (Figure 9). However this method is invalid for analysis under unsymmetrical loading. The most important unsymmetrical loading is wind pressure.

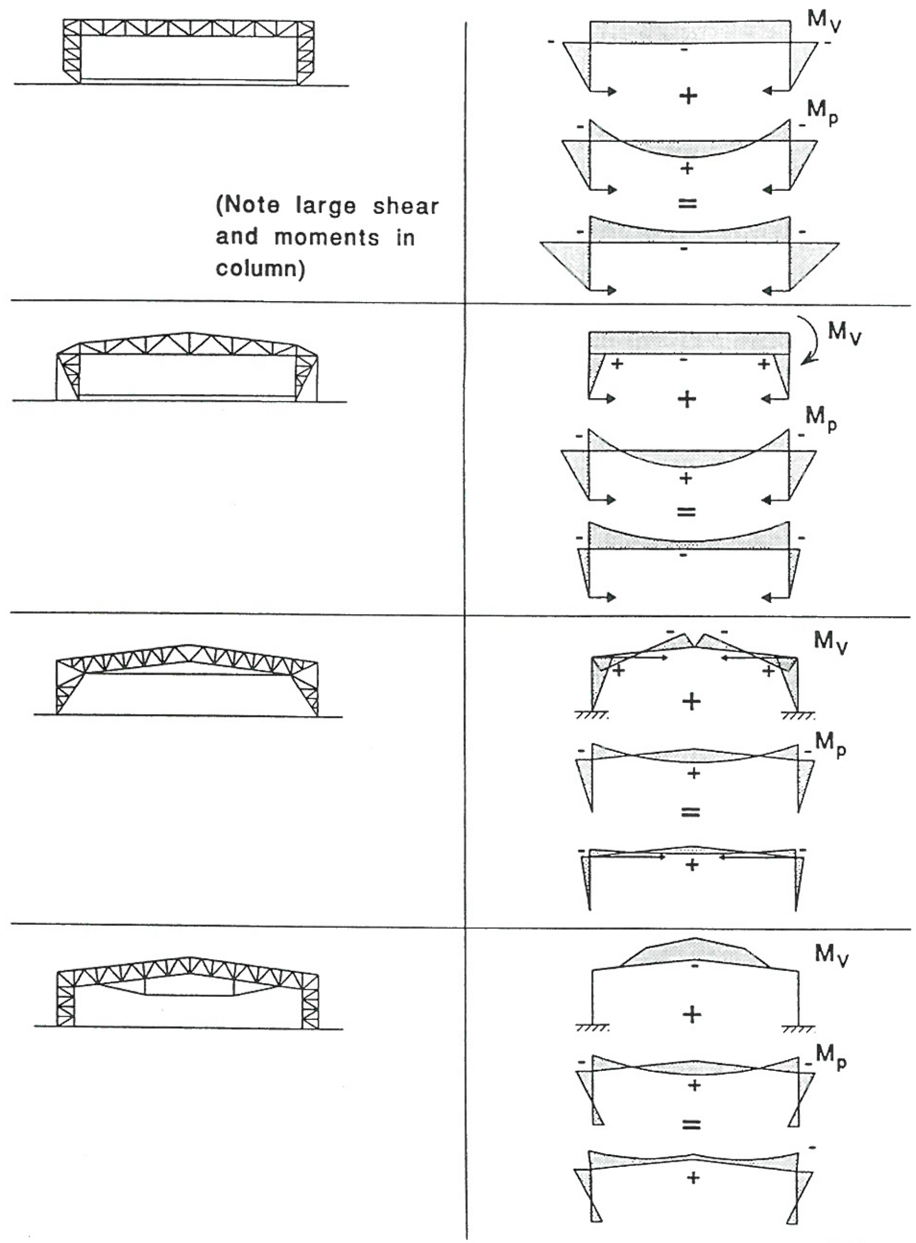


Figure 7 Prestressing of frames-some possibilities

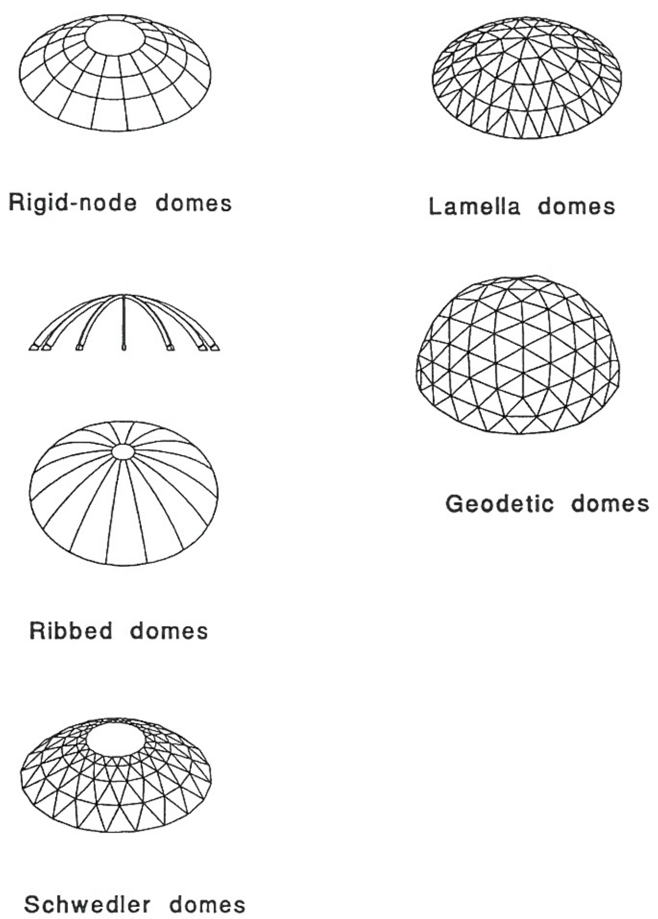


Figure 8 Steel framed domes

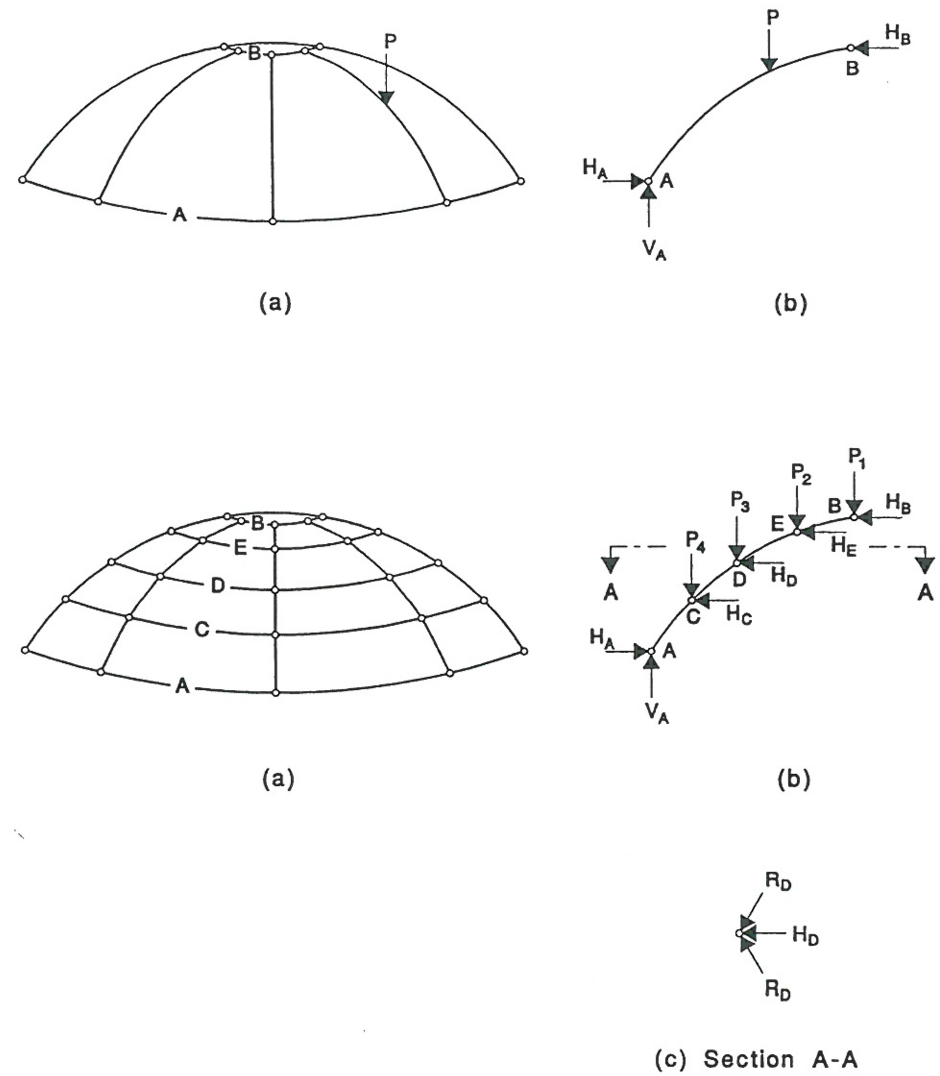
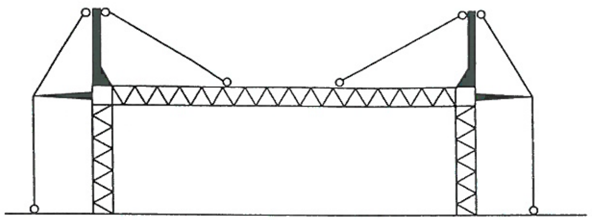
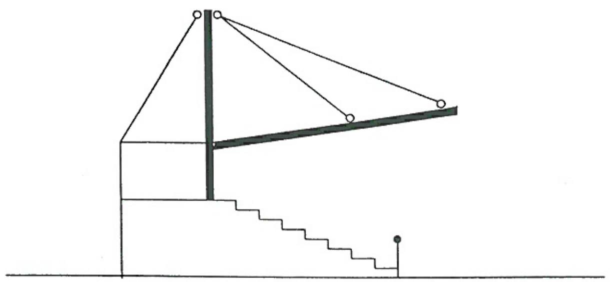


Figure 9 Ribbed spherical dome - analysis for symmetrical loading.

Economy depends primarily upon the rise-to-span ratio of the dome, and the number of ribs and rings to be used. A rise-to-span ratio around 0,13, i.e. when the spherical radius is equal to the dome diameter, provides the maximum structural economy according to some authors.



Cable-stayed roof of a building



Cable-stayed roof for a grandstand

Figure 10 Examples of cable-stayed roofs

Lecture 14.6

3. CABLE AND TENSION STRUCTURES

These structural systems play an interesting role in modern construction and are examined in detail below.

3.1 General

High strength steel cables have been used extensively over the past twenty five years for space roof structures.

There are two different possibilities when using steel cables in roof structures.

The first possibility, consists of using the cables only for suspension of the main roof structure, which can be either conventional, e.g. beams, cantilevers, etc., or a space frame. In this case, the main roof structure, instead of being supported, is actually suspended from steel cables above the roof, which transmit the tensile forces to appropriate anchorages (Figure 10). They are cable-stayed roofs.

There are many examples of this type of construction used as industrial buildings where the roof structure, either as a single or as a double cantilever, is suspended from cables, which in turn are anchored on robust pylons above the roof level.

In this type of construction, the cables behave as simple suspension elements, while the roof structure itself behaves like a normal load resisting unit, subject to moments, shears, and other kinds of action effect. It is expected that the suspending elements remain in tension, even under wind uplift, due to the dead weight of the roof.

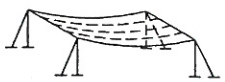
The second possibility is represented by those roof structures where the steel cables are effective members of the roof structure itself, and not just conveyors of forces from the structure to the anchorages. In this type of construction (tension structures), the cables themselves resist the various external loads. Their particular behaviour has deeply influenced the structural forms used and has imposed new methods of execution.

Tension structures may be categorised as:

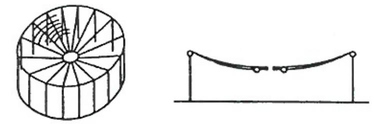
(a) Single-layer cable systems (Figure 11a)
(b) Double-layer prestressed cable truss systems (Figure 11b)
(c) Prestressed tensile membrane systems (Figure 12)

Tension structures are used to cover stadia, arenas, swimming pools, recreation halls and other buildings where a large area for public assembly and exceptional aesthetic effect are required simultaneously.

There are some particular problems associated with these cable-stayed and tension roof structures.

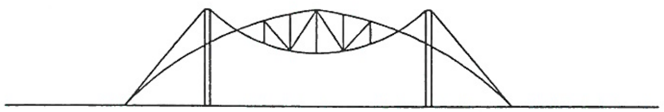


Parallel cable system

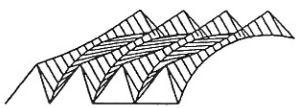


Radial cable system

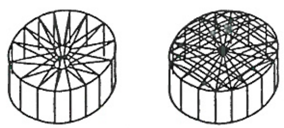
(a) Single layer cable systems



Planar cable truss



Cable space truss



Radial cable truss

(b) Double layer prestressed cable systems

Figure 11 Single and double-layer cable systems

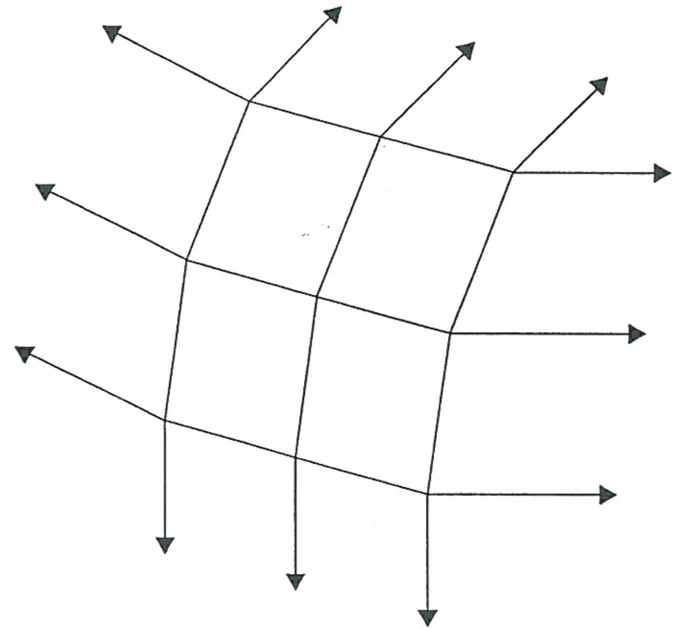


Figure 12 Prestressed tensile membrane system- an anticlastic cable net

A first problem derives from the fact that the cable is flexible. It assumes a shape compatible with the applied loads whilst architectural and building requirements demand that the structure has a definite form. Any deviations from that form due to the action of the applied loads, must be kept to a minimum. To meet this requirement, a pretension must be introduced into the structure, which must be compatible with the desired shape, and when combined with the applied loads, must maintain the deformation between specified limits. Design may therefore involve use of mathematical 'form-finding' procedures, implemented by appropriate software.

Another feature of these structures is their geometrically non-linear behaviour. Deformations play an essential role in the analysis and the principle of superposition of effects is not valid.

Finally, an important problem associated with these structures is their sensitivity to aerodynamic instability, e.g. flutter. This sensitivity imposes special requirements on the design and the constructional details of these systems, particularly those which use membranes made of lightweight fabric as cladding.

The requirements of stiffness under transverse loading and anchorage are major form determinants for cable structures, and these are examined in the following sections.

3.2 Stiffness Under Transverse Loading

Single cable structures are characterised by their flexibility, Figure 13. They require stiffening to prevent a change of shape with each variation in load and to make them capable of resisting uplift due to wind, Figure 14. Gusty winds can produce oscillations, unless damping is provided to the structure.

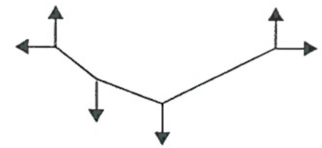
The principal methods of providing stability are the following:

(i) Additional permanent load supported on, or suspended from, the roof, sufficient to neutralise the effects of asymmetrical variable actions or uplift Figure 14a).

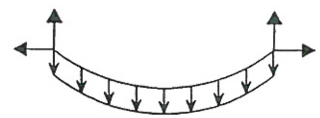
This arrangement has the drawback that it eliminates the lightweight nature of the structure, adding significant cost to the entire structure.

(ii) Rigid members acting as beams, where permanent load may not be adequate to counteract uplift forces completely, but where there is sufficient flexural rigidity to deal with the nett uplift forces, whilst availing of cables to help resist effects of gravity loading (Figure 14b).

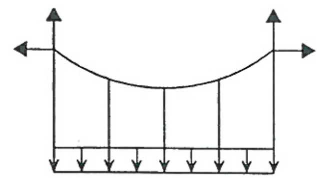
(iii) Rigid surfaces behaving as inverted shells or vaults, where uplift forces are countered by the in-plane compressive rigidity of the structure (Figure 14c).



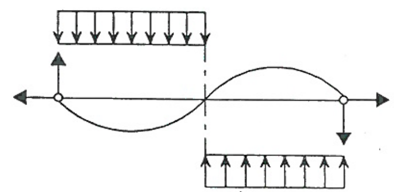
Concentrated loads (Polygonal from)



Self weight (Catenary from)

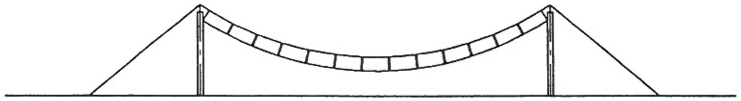


Uniformly distributed vertical load
(Square parabolic from)

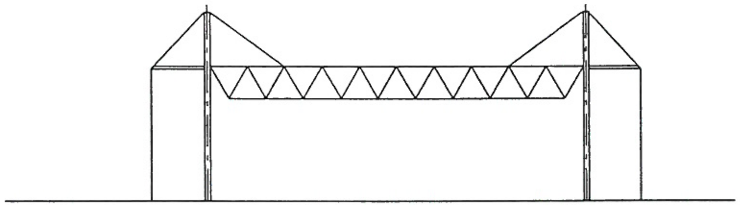


Assymetric loading with uplift

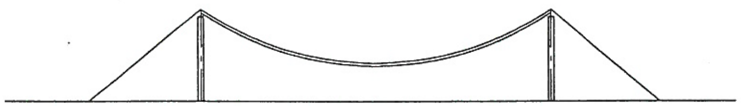
Figure 13 Single cable : Load/ shape relations



(a) Permanent load



(b) Rigid roof structure



(c) Construction as rigid arch or shell

Figure 14 Cable stability : plane systems

(iv) Secondary cables prestressing the main cables so that these remain in tension under all conditions of load. Such prestressing can take a variety of forms:

- a stayed (guyed) arrangement, wherein the main cable is stayed to other elements or to the ground, as in the case of guyed trusses (Figure 14d).

- A planar arrangement of suspension and stabilising cables, with opposite curvatures cables, Figure 14e. This structure reacts elastically to all changes of shape provoked by the externally applied loads. This principle can be extended to permit creation of space trusses, or structures of revolution.

- An orthogonal or diagonal arrangement of suspension and stabilising cables, with opposite curvatures, forming an anticlastic (saddle-shaped) surface, Figure 14f and 15.

Figures 14 and 15 show the application of these general principles to cable and cable-stayed systems, whilst Figure 16 details the structural actions of prestressed cable truss systems. Accurately defined, a cable truss system has a triangulated structural form which increases stiffness, particularly under non-symmetric loading. However, the term is also frequently applied to the cables with opposite curvature shown in Figure 14e.

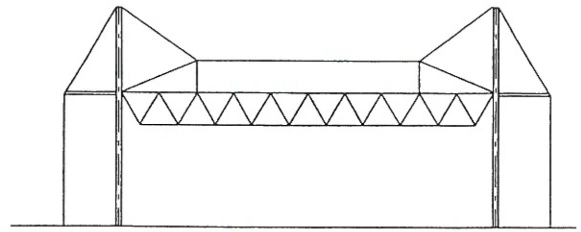
The orthogonal or diagonal arrangement of anticlastic cables shown in Figure 15 can also be extended to the conical form shown in Figure 17. The increasing use of horizontal ring cables, from Figure 17a to 17c enhances stiffness against asymmetric loading. Because of the difficulty of anchoring a large number of cables at a point, the top is usually flattened as shown in Figure 17d.

The use of anticlastic cable nets is further enhanced by the use of internal arches, Figure 18. The use of conical forms can be extended to create exciting doubly curved surfaces by the use of multiple high points and/or interior anchorages, Figure 19. The Pavilion of the Federal Republic of Germany, Expo 1967, Montreal by Frei Alto and Rolf Gutbred was an outstanding example of the former.

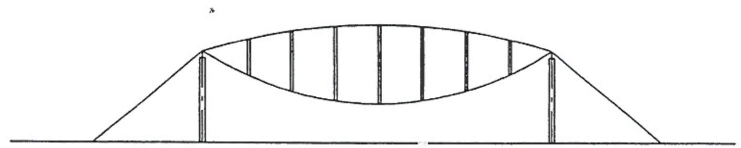
3.3 Anchorage

Cable stayed structures generate a requirement for the anchoring of tension forces. Some of the commoner solutions are:

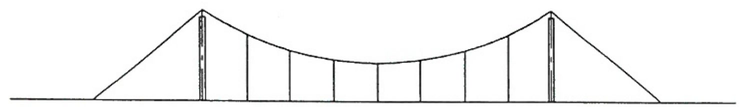
(i) Vertical and horizontal reactions provided by axially loaded elements - stayed columns used with ground anchors (Figure 20a).



(d) Additional staying

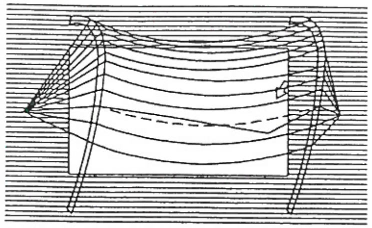


(e) Prestressing by cable with opposite curvature

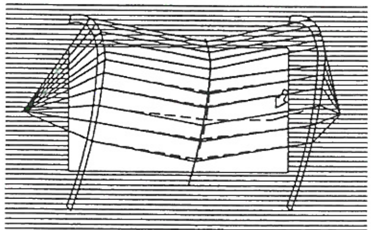


(f) Staying with transverse cables to ground or to another part of the structure

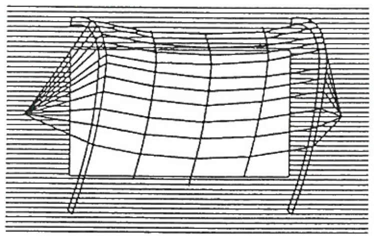
Figure 14(cont'd) Cable stability : plane systems



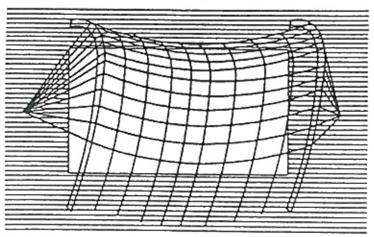
Single load causes major deflection that remains localized to the cable under load



Transverse stabilization cable stresses suspension cable and resists deflection

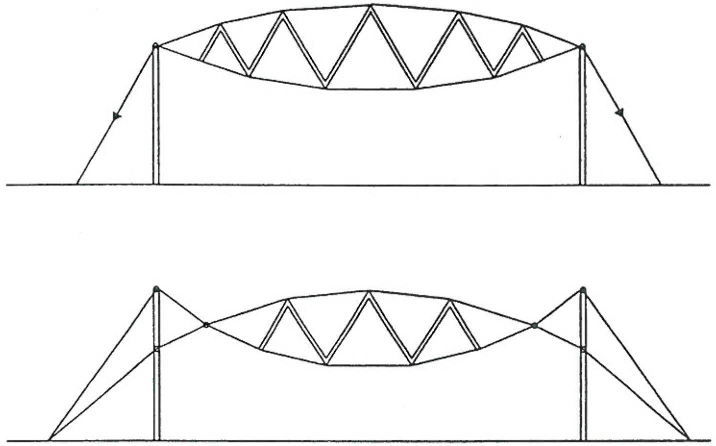


Increase of stabilization cables strengthens resistance against point loads

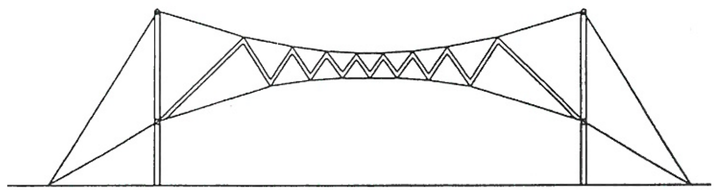


All the cable are participating in the mechanism of resisting single load deflection

Figure 15 Cable stability : anticlastic cable nets



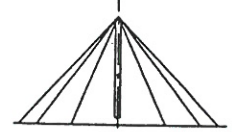
(a) Cable trusses with diagonal struts



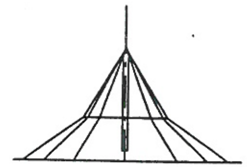
(b) Cable trusses with diagonal ties

Figure 16 Cable stability : cable trusses

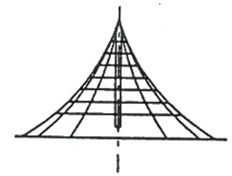
Lecture 14.6



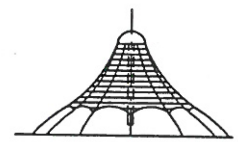
(a)



(b)



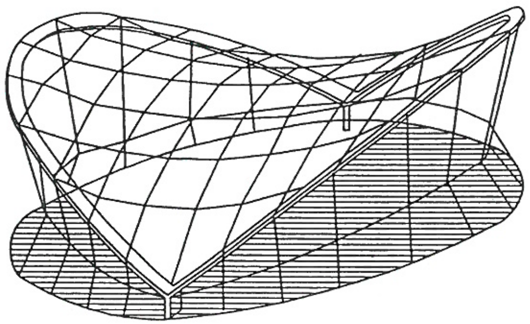
(c)



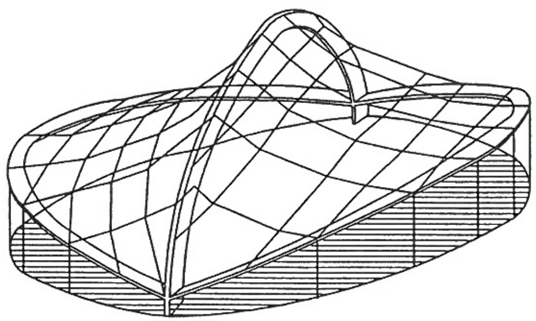
(d)

Figure 17 Cable stability : conical membrane

Lecture 14.6



2 boundary arches with common base points



2 boundary arches with one central arch

Figure 18 Anticlastic cable nets with boundary arches

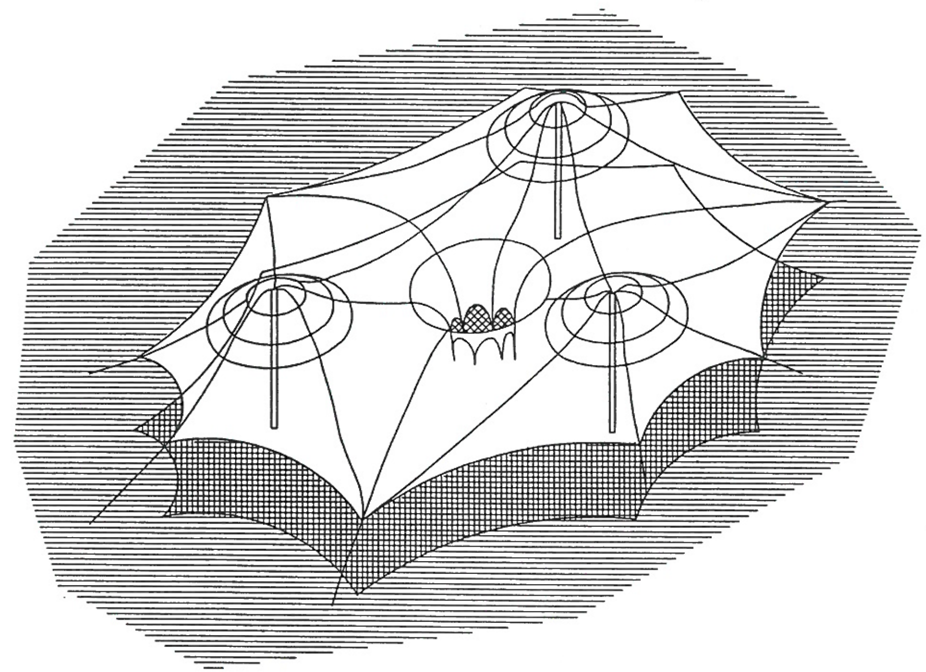
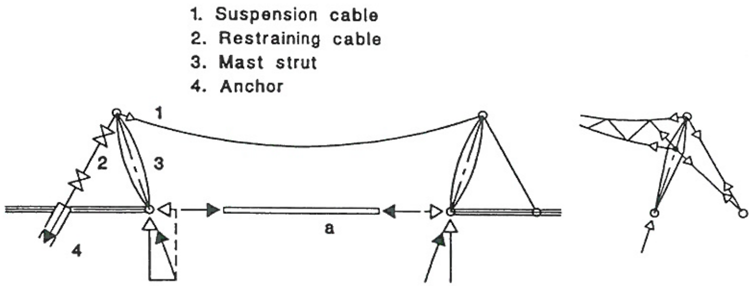
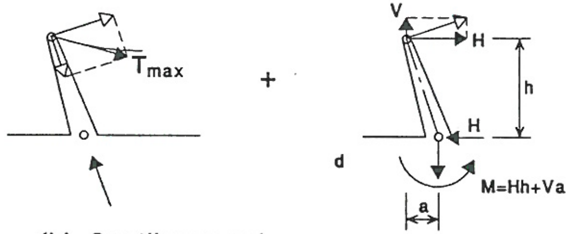


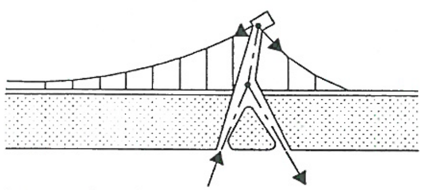
Figure 19 Complex tent system with multiple interior supports and internal anchorage



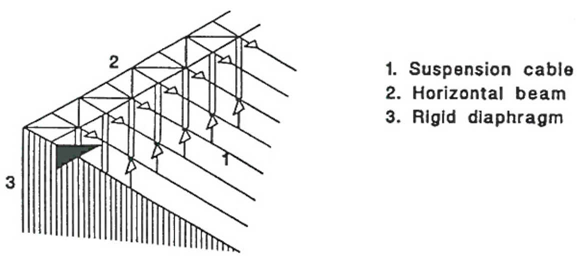
(a) Guyed masts



(b) Cantilever columns



(c) Legged columns



(d) Horizontal beams

Figure 20 Cable anchorage systems

Lecture 14.6

(ii) Vertical and horizontal reactions provided by flexural elements i.e. cantilever columns (Figure 20b) or legged columns (Figure 20c).

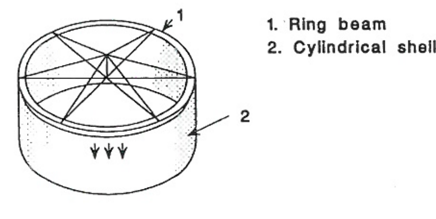
(iii) Vertical columns acting with horizontally loaded edge beams which transfer horizontal reactions to rigid diaphragms (Figure 20d).

(iv) Inclined walls, or vertical cylindrically curved walls (Figure 21a).

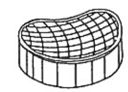
(v) Form-related boundary shapes, creating, in some cases, a closed self-equilibrating system of tension and compressive forces and requiring no tension ground anchors (Figure 21b).

The magnitude of forces in stayed columns and in diagonal stay restraining cables is reduced by inclining the columns. In some symmetrical structures lateral thrust is balanced by means of struts at foundation level.

Some tension anchorage possibilities are illustrated in Figure 22.



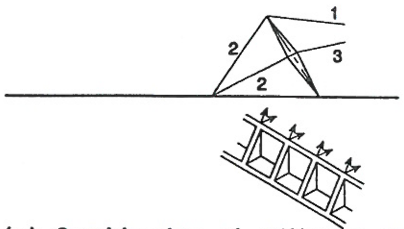
(a) Cylindrical walls



(b) Form - related boundary shape

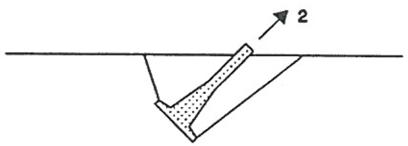
Figure 21 Cable anchorage systems - 2

Lecture 14.6

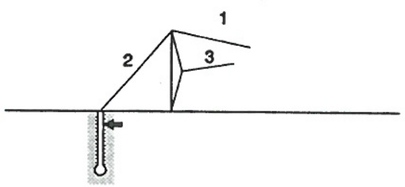


1. Suspension cable
2. Restraining cable
3. Stabilising cable

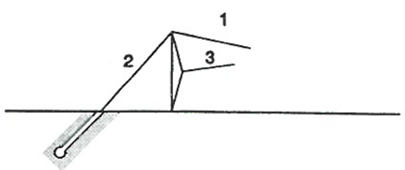
(a) Combination of stiffened plates (Counterfort walls) and soil loading



(b) Combination of massive foundation and soil loading



(c) Vertical tension pile with horizontal reaction component



(d) Tension pile collinear with restraining cable

Figure 22 Tension anchorage alternatives

4. ADDITIONAL SPECIAL STRUCTURE CATEGORIES

4.1 Hangars

Cantilever construction is used extensively for large hangars, the column free interior and facade permitting the required ease of access and flexibility of use. Sliding doors around the perimeter are vertically supported on rollers at ground level and laterally supported by the roof structure. The deflection of the cantilever roof structure must be allowed for on the design of the doors.

Single or double cantilever systems may be used. In the single cantilever system, substantial foundations must be provided to counteract the overturning moment. With symmetrical double cantilever systems, the permanent loads on either side of the central block balance each other. The central block which frequently contains offices and circulation areas, may be used to counteract the effects of unsymmetrical loading.

In regard to the composition of the cantilever, roof structures may be categorised as follows:

(a) Pure cantilever systems, formed of varying-depth trusses, girders, folded plates and shells (Figure 23a).

(b) Cable supported structures, supporting any of the above structures types (Figure 23b).

In addition to gravity loading and temperature effects, the roof structure is exposed to wind loading on its upper and lower surfaces. In the case of cable-stayed structures, particular attention must be paid to uplift.

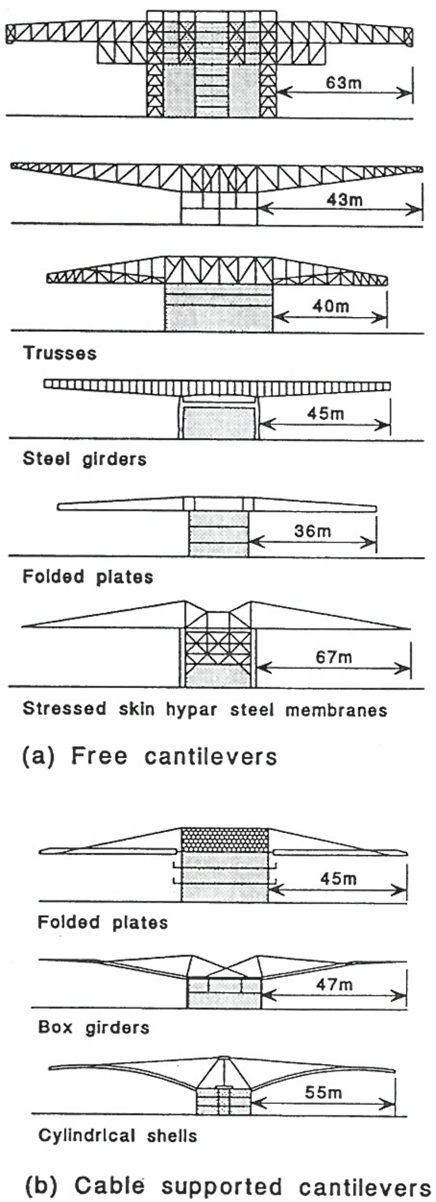


Figure 23 Large span cantilever roof structures

5. CONCLUDING SUMMARY

- Standard industrial forms can be varied to provide structures capable of spanning considerable distances.
- Curved forms, arches or domes, provide further possibilities.
- Cable staying extends the spanning possibilities of conventional trusses or truss frames.
- Tensile structures open up a large repertoire of dynamic structural possibilities for medium to large-span structures.
- Tensile structures may be planar or anticlastic, membrane or cable net structures.

6. ADDITIONAL READING

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