AIRSHIP-HANGAR NEAR BERLIN

Design and analysis for ARUP´s biggest internal space today

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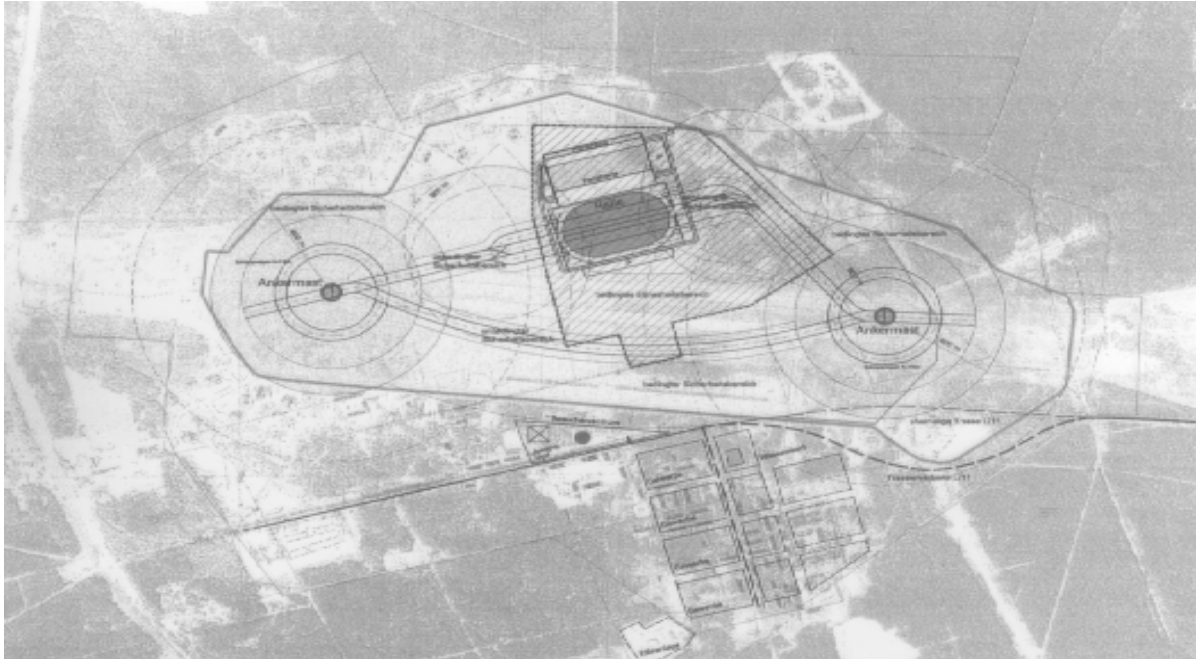
0.   Introduction

In the middle of 1997 CargoLifter AG commissioned the design for an airship hangar in Germany to house two new airships. The hangar is needed for production and maintenance of a new generation of airships. These CL160 airships are so-called "blimps" - airships filled with helium without skeleton. The airships are designed to lift and transport goods of up to 160 tons on long distance haul.

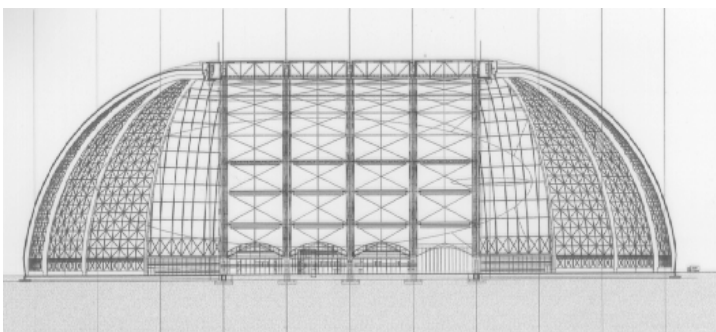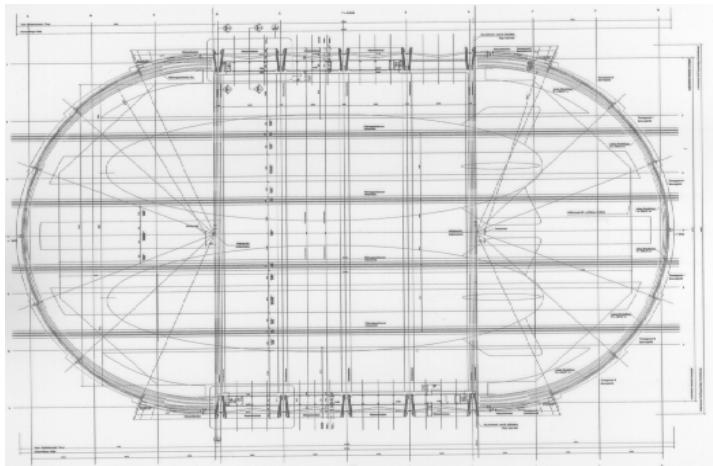The site is an airfield in Brand, approx. 50 km south of Berlin, formerly used by Soviet troops.
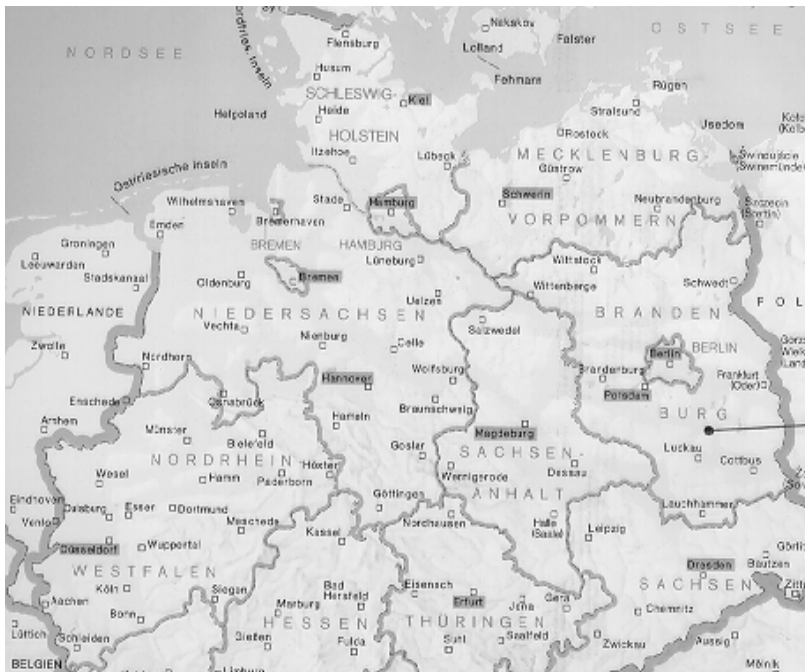
BRAND

Fig.1: Map of Northern Germany

The whole project comprises the hangar - the "Werfthalle" - itself, a number of subsidiary buildings for the production of components as well as a visitors centre. (Fig. 2)

Fig.2: Site plan with hangar, subsidiary buildings and position of anchor masts (SIAT)

1. Building Concept / Architecture

Despite the fact that it will be an industrial building, the architecture of the CargoLifter hangar is of considerable importance, both in functional and visual terms, as it has been designed by the architects SIAT from Munich.

The airship hangar has to fulfill various very specific requirements: it has to house two airships at a time, it must provide appropriate working conditions (min. internal temperature 17°C 2m above ground, avoid drafts, guarantee sufficient lighting) and allow for light cranage under the roof construction.

The total building is 360m long, 210m wide and 107m high. A total covered area of 63.000m^2 - completely free of columns - would allow for eight football pitches. The shape is oriented closely on the clearance diagram for two airships.

Fig. 3: Plan and longitudinal section (SIAT)

Sliding doors which rotate at both ends of the building allow the airships to be hauled in and out under appropriate wind conditions.

Two-storey high concrete "naves" along both sides of the production floor provide staff facilities for 250 employees, labs and offices for a further 75 staff.

A colour scheme has been specifically chosen for a building of this size. Two main criteria had to be satisfied: good visibility for aircraft, and colours which will not disturb migratory birds on their flight North/South. Furthermore the colour scheme is of great importance for the corporate image of the client.

Heating is provided by under-floor heating as well as radial panels hanging from the steel arches in the side areas.

2.   Stuctural Concept

The structural concept, developed by ARUP GmbH in Düsseldorf office together with Ove Arup & Partners in London, distinguishes two main parts of the building. The central part is of a cylindrical shape consisting of five steel arches at 35m centres - each of the four bays being covered with a textile fabric - and the sliding doors at both ends.
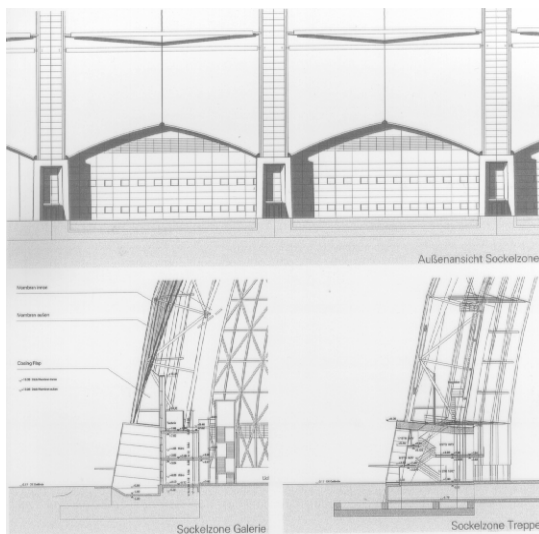
The arches spring off concrete plinths which act as a covered entrance as well, providing safety against snow-avalanches coming from the fabric roof. (Fig. 4).

These bases are founded on large concrete pad footings designed to limit the settlement to 30mm and to avoid sliding due to horizontal wind loads and thrusts from the arches. No additional tension member ties the bases together. The steel arches have a clear glass roof between their top cords at 3.4m centres to allow daylight into the building.

Fig. 4: Entrances at arch-bases (SIAT)

The steel arches have cross bracing between them internally and props on the outside to avoid overall torsional buckling of the arches. Thus, the steel arches provide a very stiff framework against horizontal thrust from the sliding doors.

At both ends of the building are the sliding doors which consist of two fixed and six moving elements each. (Fig. 5).

Both doors form a semi-circle in plan and a quarter-segment of a circle in elevation. Three moving elements slide under one fixed element. Each shell-shaped element is fixed to a hinge at the top of the end arch and guided horizontally by rails, both in tangential and radial direction, at
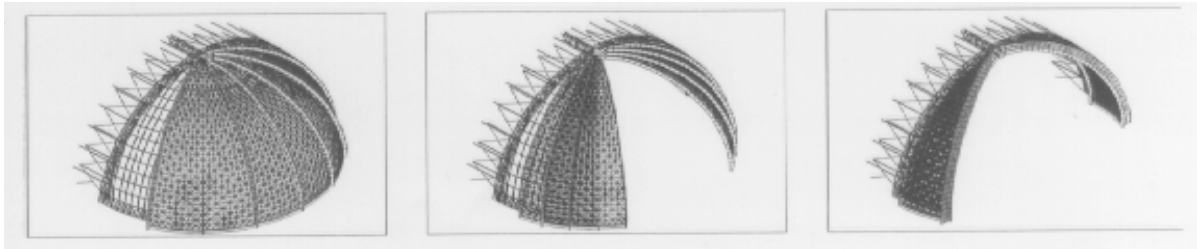
the bottom. Each sliding door element has two motor drives at both ends at ground level.
Fig. 5: Sliding doors, opening (SIAT)

The floor is constructed as a concrete slab, using road construction techniques. The side buildings are simple flat-slab-on-column concrete structures with an 8.1m x 5.6m grid.

3.   Loading Assumptions

Initially all the known and assumed loading data has been assembled and gathered in one document. This document has been sent to and checked by the entire project team, including the architects and the checking engineer. After receiving approval for all the loadings the "Loading Assumptions Report" formed a very important base for the whole engineering team carrying out the calculations.

For the loading a distinction has been made between dead load, live load and special loads. The dead load contains the weight of all the building elements. The snowload considered is based on the German standards using a base value of 0.75 kN/m², which can be reduced depending on the slope of the surface. The IFI Institute of the Fachhochschule Aachen carried out a wind tunnel test in order to determine the wind loading. Regarding the symmetrical shape of the ´Werfthalle´ it was considered sufficient to analyse only three wind-directions: 0°, 45° and 90°. Apart from the wind pressure coefficients (C_p-values), directly applicable wind-loadings per defined area in kN/m², at a wind-speed of 44.9 m/s were obtained. The distribution of the C_p-values differs considerably from similar examples in literature. The crosswind (0°)C_p-values along the middle arch are shown in Fig.6 and for comparison Fig.7 presents those according to Cook [1] for a cylindrical building.
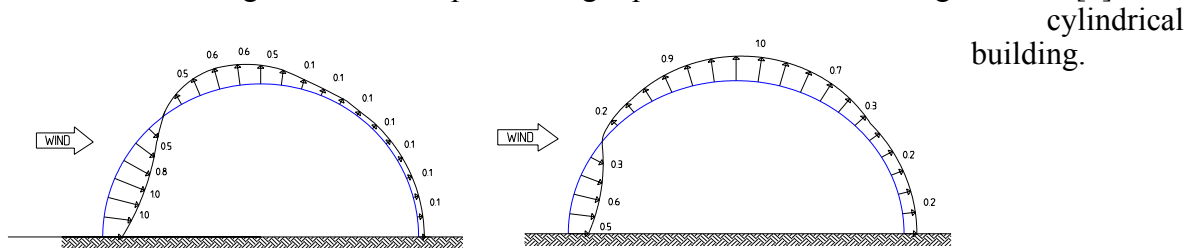

Fig. 6: Wind profile from wind tunnel test   Fig. 7: Wind profile from [1]

Wind tunnel tests were carried out for the closed-door case only. This is justified, as the doors of the hangar will only be opened at a maximum wind velocity of 10m/s. Hand-calculations were sufficient to realise that the open door case would not be a design load case. In addition to the snow and wind loading, internal crane loading, using dynamic-factors up to 1.96, were considered in the analysis.

As special loads a ±45°C temperature variation was applied to the external steelwork and a ±10°C to the internal steelwork. Further, a load due to ice of 30 mm thickness (0.21 kN/m²) was applied on the external steel. The influence of foundation settlements (50 mm) have also been analysed. For this size of structure, the effects of foundation settlements were negligible, as would be reasonably expected.

4　　The Cylindrical Part of the Building

4.1　　The Main Steel Arches

The cylindrical part of the building is formed by 5 steel truss-arches, placed at 35m centres. The arches have a structural height of 8m and span over 225m. The top chords are at 3.441m centres and the bottom chords at 2.0m centre. The chords of the truss-arch are brace-connected to each other, with the exception of the two bottom-chords; these are connected by only straight members, forming a Vierendeel-system. (Fig. 8)
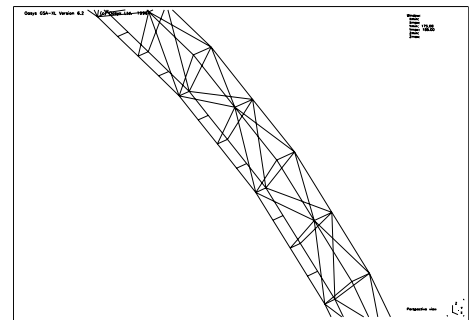

Fig.8: Main steel arches, typical system

Although the arches are always referred to as being the cylindrical part of the building, the arches are not perfectly circular but polygonised. Seventeen straight segments, each with a length of about 18m, form one arch. The wind-bracing is connected to the bottom chords at each intersection of two arch-segments. At the same intersection between the top chords, external props restrain any torsion in the arches, induced by the eccentrically connected membrane.

At their ridge the arches are longitudinally connected by a four chord, 8m deep truss, similar to the structure of the arches. This ridge beam enables the connection of the membrane and the valley cable at the top, and takes up the large compression force between the two end arches, generated by the doors.

At their bases, at +8.85m above ground, the arches are encastred in the foundations. Due to fixing the arch-ends approximately 10% of the steel was saved in the arches.

Tubular hollow sections were used as structural elements for the arches because of their high torsional resistance and their good buckling performance. The chords have an outside diameter of 559mm, the diagonals and the bottom straights of 355mm and the side and top straights of 273mm.

The building has been analysed in three parts: the membrane, the cylindrical part and the doors. The cylindrical part was analysed as one large model, containing the five arches, the internal windbracing and the external torsional bracing as structural elements. The membrane and door forces were applied as loads onto the structure. The deformations of the cylindrical part, calculated for the applied loads, have been taken into consideration in the analysis of the doors and the membrane. This enables a separate analysis of the cylindrical part to be justifiable.

The various sections of the arches have been grouped in such a way that the structure has double symmetry about the centre-lines. In one segment 10 groups have been defined (side brace, top brace, top chords, bottom chords, bottom straights etc.). The change in groups always takes place

per segment. Hence, in one segment the structural components may differ one from the other, but they are consistant in one group. An iterative analysis allowed to optimise the steel utilisation over the total structure in a sensible way. The cylindrical part has been designed by means of the Ove Arup non-linear structural program FABLON, also using the buckling option of GSA to determine the mode shapes. The lowest buckling load factor found was 4.765 for the diagonal wind loading case in combination with half the snow load on one side. (For further information on the buckling analysis refer to section 5)

The cylindrical part of the building covers an area of about 31.500m² and has a total weight of approx. 4,200 tonnes.

4.2 Fabric Roof

The building enclosure is achieved using a stressed membrane spanning between the trussed tubular arches in the warp direction and between the ridge truss and the edge cable attached to the arch bases in the fill direction.

Initial concern was raised in the ability to generate enough curvature in the membrane to limit stresses and deflection due to the large radius of the arches and their relatively small spacing. Initial analysis based on a form found surface with equal prestress in the warp and fill directions verified this. An additional constraint on the surface shape was to limit the deflection of the membrane under wind uplift so that it did not clash with the external bracing of the main structure.

By adopting a valley cable midway between the arches and by form finding the surface with an increased prestress in the fill direction, the warp span is decreased and the surface stiffened in the fill direction even though the overall curvature is decreased. The inclusion of a valley cable also had benefit when it was necessary, due to environmental reasons, to add a second internal membrane. By adjusting the prestress levels in the valley cables during form finding it was possible to develop surfaces that did not clash when loaded.

Early on in the project coated polyester was chosen as the preferred membrane material over the stiffer PTFE/glass chiefly on the basis of supply and cost. After the initial analysis, the polyester was shown to be the more onerous of the two for deflection of the membrane but at the greater benfit of producing much lower localised stresses. High localised stresses produced by PTFE/glass would have proven more complex to remove.
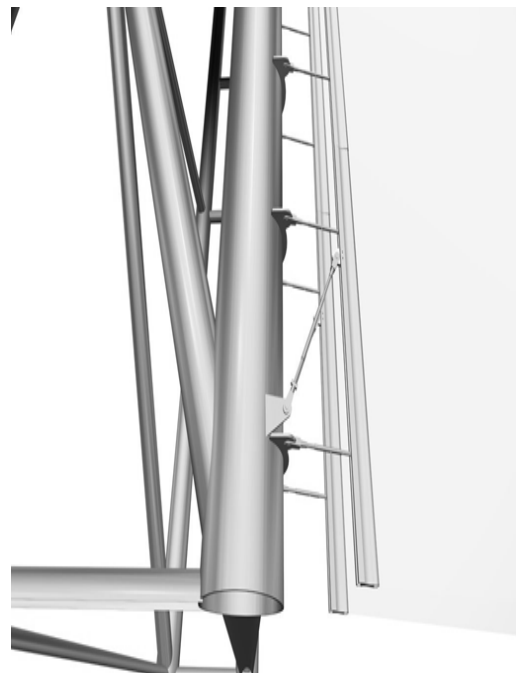

Fig 9: Membrane roof connection to steel arches

The development of robust details that are reasonably insensitive to construction tolerance is essential to the ease and speed of construction and prestressing of a structure of this size [2]. Initial details, in which the stressed fabric was directly attached to the tubular sections, were rejected in favour of an extruded aluminium luff groove. By doing so the details around the tube, that included glazing system and weather closure details, are greatly simplified.

The luff groove is held approximately 700mm away from the main structure by swinging links. The fabric edge contains a bolt rope that is fed along the luff groove and the membrane is stressed by pulling back on the luff groove and adjusting the swinging links. The swinging links also slot into a channel in the luff groove allowing for accurate alignment on site and their connection with the tubes are pinned to allow rotation as the membranedeflects under load. Diagonal links are added after stressing at the ends of each arch segment to remove shear forces.


Fig. 10: Connection detail

5. Sliding Doors

5.1 General (Geometry)

The hangar entrances are located at both ends of the cylindrically shaped main part of the building. They constist of a shell structure with a spherical surface. The quarter shells, thus formed, are subdivided into 8 parts (fig. 11). The 2 bays adjacent to the cylindrical part, are fixed and more or less continuously connected at one side to the end arches. The other six bays of the sphere form the moveable door segments. These "sliding doors" can move underneath the fixed door segment in the situation when an airship is being hauled into or out of the hangar.
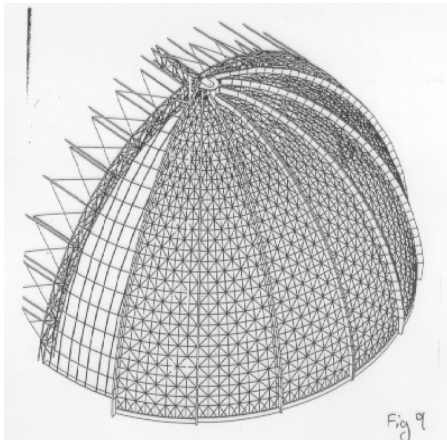
The sliding doors are each supported at 3 points. A common support for all 6 doors is the cantilevered "ridge point" at the end of the ridge beam. The different door radii, as well as the different support height for each door (necessary for the opening mechanism), required a complex detailing of the ridge point and the upper door parts (vertical and horizontal setbacks). The 2 supports at the bottom of each door contain the driving mechanism. The upper part of the concrete strip foundations include the necessary rail details to a) allow the doors to move in the desired direction during opening or closing and b) restrain the doors in the tangential and radial directions in the closed position.

Fig. 11 : Sliding doors structure in closed position without cladding, for clarity.

Symbols: In the following paragraphes these symbols are used typically:

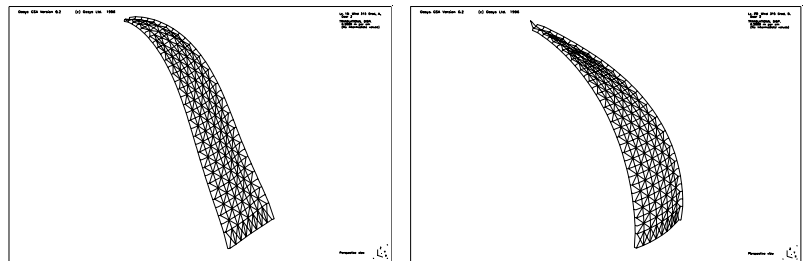|			
a:	Plate length	b:	Plate width
d:	Depth	w:	Section width
t:	Plate thickness	λ_{cr}:	Critical load factor

5.2   Structure

The enormous dimensions of the doors (arch length 168m, bottom width 42m) result in large weights. The door weight strongly influences the hangar costs: the upper door support reactions are collected in the ridge point, adding up to a point loading on the end arches. This point load has a large effect on the section dimensions in the arches. The cost of the foundations and the driving mechanism depend on the lower door support reactions which are mainly a result of the steel weight of the doors. Minimising the door tonnage was a main aim. Therefore various structural alternatives were investigated.

The lightest structure was achieved by adopting a shell principle: The inner part of a door segment consists of a spherically shaped frame structure, realised by identical horizontal, vertical and diagonal elements (DIN HE240A-sections), which are rigidly connected in their joints. The cladding (corrugated metal sheets) spans between the horizontal elements. Stiff boundaries are a prerequisite for activating the shell action. This led to the use of large rectangular hollow sections for the 2 side beams and the lower beam. The side beam dimensions are d x w = 3000 x 800 [mm]. The shell is eccentrically connected to the side beams: at one side to the top flange and at the other side to the lower flange. Hence, a horizontal section through a door shows an approximate Z-shape, allowing for the overlapping of doors underneath each other. At the bottom the shell joins the lower beam (d x w = 2300 x 800 [mm]) concentrically.

5.3   Design

Due to the hangar plan-symmetry the investigations could be restricted to 4 different doors (including the fixed door). The wind tunnel test produced 8 different wind profiles for each of the 4 doors. Together with dead weight of steel frame and cladding, snow loading (according to DIN 1055), temperature loading ($\Delta T_{shell} = \pm 10K$, $\Delta T_{outer\ edge\ beam} = -35K, +45K$) and movement of the ridge points, several hundreds of load combinations were generated. With the ARUP (linear) structural calculation program GSA, a first impression of governing load combinations was achieved. A distinction of 2 loading cases, producing the typical deformation shapes of an arch, could be made. The appropriate loading factors were determined by making use of those shapes. In Fig. 12 two governing wind profiles with their accompanying deflection shapes can be observed. Shape A shows a typical "gravity dominated" deflection , whereas shape B shows a typical "wind pressure dominated" deflection. In the latter situation the loadings acting approximately in gravity direction are multiplied with a loading factor 0.9.

Fig. 12: A) Gravity dominated   B) Wind pressure
         deflection   dominated deflection.

The German steel design code (DIN18800, part 2) prescribes a nonlinear analysis for structures with critical loading factors λ_{cr} less than 10, as linear analyses do not produce realistic section forces for such structures. The procedure is to superimpose scaled buckling shape deflections and the original geometry and perform the nonlinear analysis on the thus obtained imperfect structure. With a selection of governing loading combinations a nonlinear analysis of the doors was made. GSA allows for determination of the factors λ_{cr} and their buckling modes. The GSA-analyses produced minimal λ_{cr}-values of about 4.5 for the wind pressure dominated loading combinations. With the ARUP nonlinear calculation program FABLON the imperfect structures were nonlinearly analysed. FABLON recognises eventual instability of the structure (the development of the collapsing mechanism can be monitored on the screen), so that the derived stresses in a

"survived structure" can be directly used for comparison with allowable steel stresses (without slenderness limitations). Additionally, however, a lateral torsional buckling check for the single elements in the structure was required as FABLON does not account for this instability phenomenon. As a result of the nonlinear analyses, the shell was locally strengthened by applying HE240B (close to the supports) and HE500A (horizontal elements over lower beam) sections.
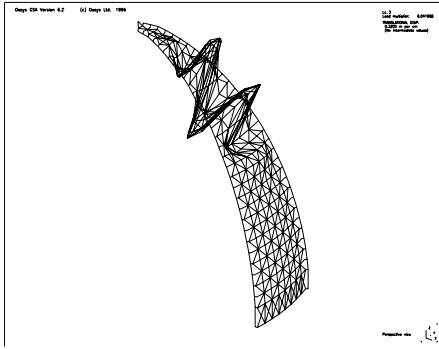

Fig. 13: Example for a local buckling mode (λ_{cr} = 5.98)

The main conclusions from the analysis can be summarized as follows:

- The buckling shapes, belonging to the lowest λ_{cr} –values, generally had a local nature and occurred in the shell. These lowest λ_{cr} –values are closely-spaced and have therefore similar buckling shapes.
- The lowest λ_{cr} –values were found for the wind pressure dominated loading combinations. For these cases the nonlinear calculations produced stress increases up to about 20%. The largest linear stresses, however, occurred for the gravity dominated loading combinations. Those were less influenced by nonlinear stress lincreases.
- The deformations of the lower part of the shell, relative to the edge beams, were in the order of 30cm. Even though the horizontally oriented elements are very flat, the presence of an initial arch height of about 2m does not give rise to a snap-through occurrence.

5.4   Details for plate buckling prevention

In order to minimize the weight of the edge beams, which comprise 70% of the door weight, the web thickness was chosen as small as possible (t=10mm). The side beams are subdivided in 33 straight parts, resulting in webs of b x a = 3000x4660 [mm]. Both, for defining fixed boundaries of the "buckling fields" and for transferring the local (eccentric) shell loads into side beam section forces, plates (t=8mm) are welded orthogonally between the different beam segments as internal stiffening plate. The combination of stiffeners (T-90 sections to DIN, one vertical and 2 no horizontal) and a local increase of the web thickness prevents the webs from buckling. The calculations are made by implementing the regulations from DIN 18800, part 3, and graph values from [3] in a spread-sheet program. The web beam flanges and the lower-edge beam are not sensitive to buckling due to their small b/t ratios.

6.   Concrete Elements, Foundations

In addition to the large amount of steel, large amounts of conrete are also used for the Werfthalle, about 20.000m³ in total. The main concrete elements are the foundations of the arches and the

doors, and the floorslab. Between the concrete bases of the arches a two storey high office area is planned.

The fairly good soil conditions at the site did not require complicated foundations. A maximum allowable soil pressure of 500 kN/m^2, with the assumption of a maximum settlement of 40mm, was used for the calculations of the door foundations. The groundwater level at -15m was not of importance so that sliding, stability, occurrence of tension and maximum soil pressure were used as design criteria.

A strip foundation, consisting of an one metre thick reinforced concrete slab and an upstanding block of 2.0m height was used for the sliding doors. The foundation geometry was mainly governed by the required rail pattern for moving the doors. The slab width varies from 6m to 10m (from the middle towards the fixed door segment). The large total length of the strips made use of doweled temperature/shrinkage joints necessary. These joints are put in the middle of one segment (covering one door segment).

Single slab foundations are used for the arches in the cylindrical part. The bottom level of the 2m thick foundation slab is at -5m. Two different slab dimensions can be distinguished, depending on the position in the "cylinder". For the end arches, where the large door loads are taken up, foundations of 12x26m are necessary. At the inner arches, the foundation sizes could be limited to 9x16m. The arches are based at +8.85m and connected to the foundation slab through 800mm thick concrete walls, which form the short side of the office zone. In order to minimise the foundation moment about the hangar's length axis, the centre line of the arch is placed excentrically on the slab. Use is made of the fact that the horizontal arch forces act in the outward direction: putting the slab perimeter outside of the hangar results in counteracting moments from horizontal and vertical arch forces.

The decks of the office area are designed as flat-slab-on-column. The 300mm thick concrete slabs span between the concrete bases of the arches over 3 rows of columns, with a maximum clear spann of 8.5m. The slabs act as diaphragm between the concrete bases, restraining the fassade against horizontal loading. The slabs are only connected to one base. An expansion joint connection to the other base allows for temperature movement of the slab.


7.   Organization

CargoLifter AG as the client commissioned SIAT Bauplanung und Ingenieurleistungen GmbH & Co. OHG with the planning and architectural design for the whole site in early 1997. After initial investigation with regard to contamination of the ground as well as environmental-impact studies the application for planning permission was submitted in March 1998, immediately followed by the application for air-traffic-permissions.

The structural design by Arup's was started in October 1997 and finished in June 1998. Site works started in May 1998 with the steel construction due to start in October 1998. The building is due to be completed in June 1999.

Total building cost for the hangar is estimated at DM 120 million.

8.   Conclusion
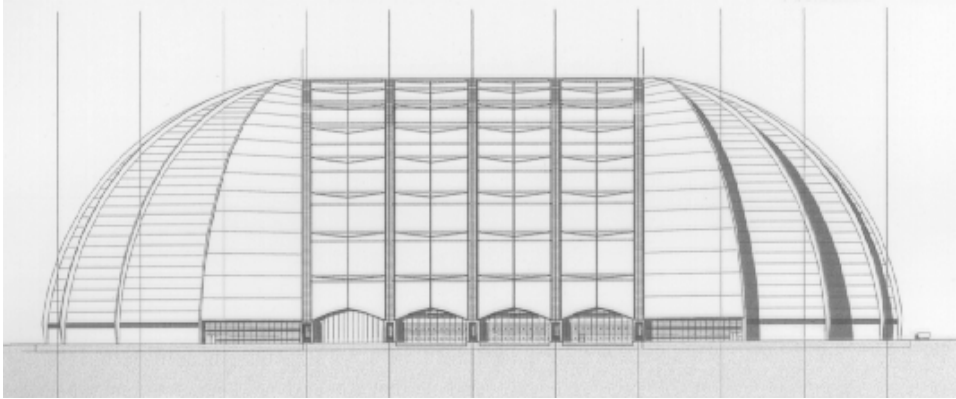
The very specific requirements of the airship-production for the CargoLifter AG have challenged SIAT-Architects and ARUP-Engineers to design an interesting lightweight building to high aesthetic standards.

The building, due to its size, is of course a huge challenge, however by using proven technologies and finding a tailor-made shape for its use, an economically viable solution has been found to suit the client's requirements.

Fig 14: Side elevation (Noth View) (SIAT)

References:

1:      Cook: The Designer´s guide to wind loading of building structures, Butterworths, London
2:      Bubner: membrane construction, connection details, Wehlmann, Essen, 1997
3:      Klöppel, Scheer: Beulwerttafeln ausgesteifter Rechteckplatten, Ernst und Sohn, Berlin, 1960

9.   Credits

Client: CargoLifter AG   Kreuzberger Ring 21
        65205 Wiesbaden   Building Services: Ingenieurbüro Klöffel
                                         Fliederstraße 1
                                         63486 Bruchköbel
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