THE ARUP JOURNAL

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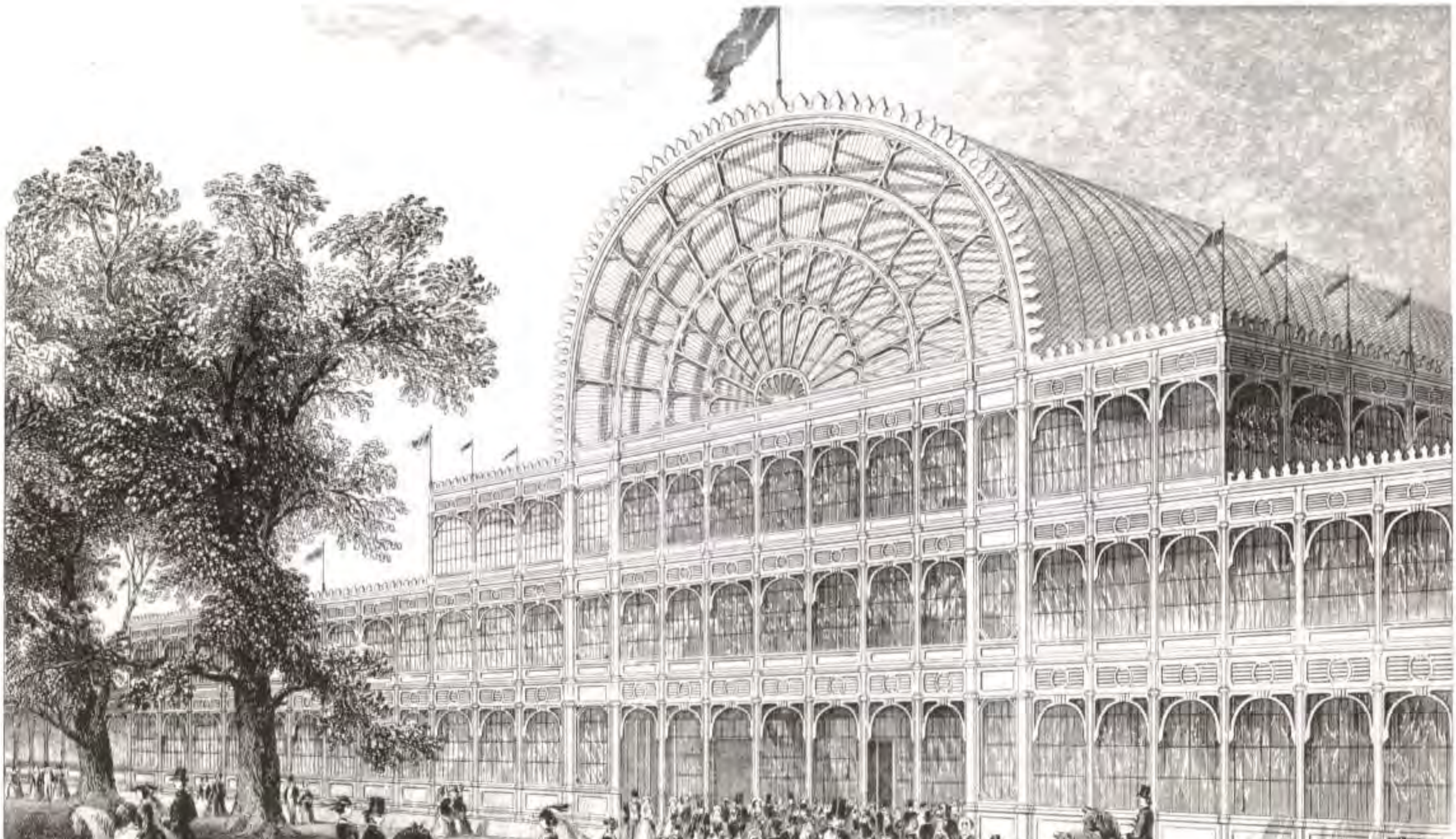

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Front Cover: Contemporary print of the North Transept of the Great Exhibition building: 1851, designed by Sir Joseph Paxton.

Back Cover: 1848 print of the Palm House at Kew, designed by Decimus Burton.
Cover illustrations reproduced by courtesy of Mary Evans Picture Library.

The Burntisland Project: Forth 150

Tom Ridley
Jim Shipway
Bryan Wright

Introduction

This project is the development of a design for an oil production platform to be built at Burntisland, Fife, by Caledonian Platform Structures Ltd. The design has been taken to sufficient detail to allow tenders to be prepared. The construction site (Fig. 1) and stages in construction have been granted full planning permission so that an immediate commencement of work can be offered to confirm the viability of future bids to oil companies.

All the major production platforms for the UK sector of the North Sea will be set in Scottish waters (Fig. 2) and most of the UK-built platforms will, in fact, be built in Scotland.

Scotland's coastline offers limited water depths for concrete platform construction sites, especially near existing industrial communities. Locating this new industry in isolated coastal sites on the West Coast caused acute planning and communications problems. Caledonian Platform Structures Ltd decided to choose an area where full use could be made of existing labour pools, housing and public services.

Fig. 1
Aerial view of site
(Photo: Fairey Surveys)

A promising site, which fulfilled many requirements, was found at Burntisland in the Firth of Forth on the east coast of Scotland. Owned by the Forth Ports Authority, who are responsible to the Department of the Environment for their land use policy, the Authority selected Caledonian Platform Structures Ltd to be given the site option for developing concrete platform construction at Burntisland, and the project has their enthusiastic support.

The main objective is to design a concrete platform structure, which in addition to meeting all other requirements for successful oil production, can be constructed with an allowable draft of 23/24 m.

As in so many North Sea ports, a site like Burntisland can only be found in tidal estuaries, which offer the limited water depths suitable for normal shipping activities. These depths cater for the largest sea-going vessels, with drafts of 19–24 m.

More usual designs of recent years have largely ignored the limits imposed by available UK sites. With floating drafts of 35–40 m they have created a demand for major industrial development in remote 'green field' areas, which have the necessary deep water to accommodate them. Not unnaturally, it has proved extremely difficult to reach agreement on using these remote sites and expensive to develop them for concrete platform construction purposes.

This article chiefly discusses the development of the design work for the proposed concrete platform, now known as 'Forth 150' (Fig. 3). The title designates its origin and offshore location water depth.

Resources required for platform design

The design skills needed to develop offshore structures are significantly different from those used on land-based structures. In the early days we had known little of the evaluation of the forces from waves, and a programme of research and development was undertaken which established a considerable degree of expertise in this field. Stability of the structure under different sea conditions and at different drafts demands a knowledge of naval architecture and sea states. Offshore, geotechnics is exceedingly important. Oil technology, dynamic response, thermal effects, towing and dredging studies all have to be explored and understood. Planning consents and programming of office and site work likewise have demanded attention.

Allied to these has been the associated laboratory work to prove our concepts. Wave forces have been extensively tested at the hydraulic laboratory at Trondheim, Norway. Model testing of sea behaviour under towing conditions for our designs has been investigated at the Universities of Glasgow and Newcastle-upon-Tyne. These tests provide preliminary proof that the designs are feasible under wave loading and afloat.

Oil companies require bidders to employ certification authorities to check platform bid designs both for adequacy in the offshore location and during construction. The best known in North Sea work are Det norske Veritas of Oslo, and Lloyds Register of Shipping. We have chosen to work with Det norske Veritas, who have issued the necessary 'quality assurance' covering our present designs.

Bidders are also required to obtain marine insurance, and this entails approval for the maritime aspects of their designs through every stage of construction from Noble, Denton & Associates, an established firm of marine assessors. Approval from Noble, Denton takes the form of a report similar to that issued by the certification authority.

All the foregoing activities generate costs which are not light. Staff costs run into tens of thousands of pounds, excluding the testing and certification expenses which are not insignificant.

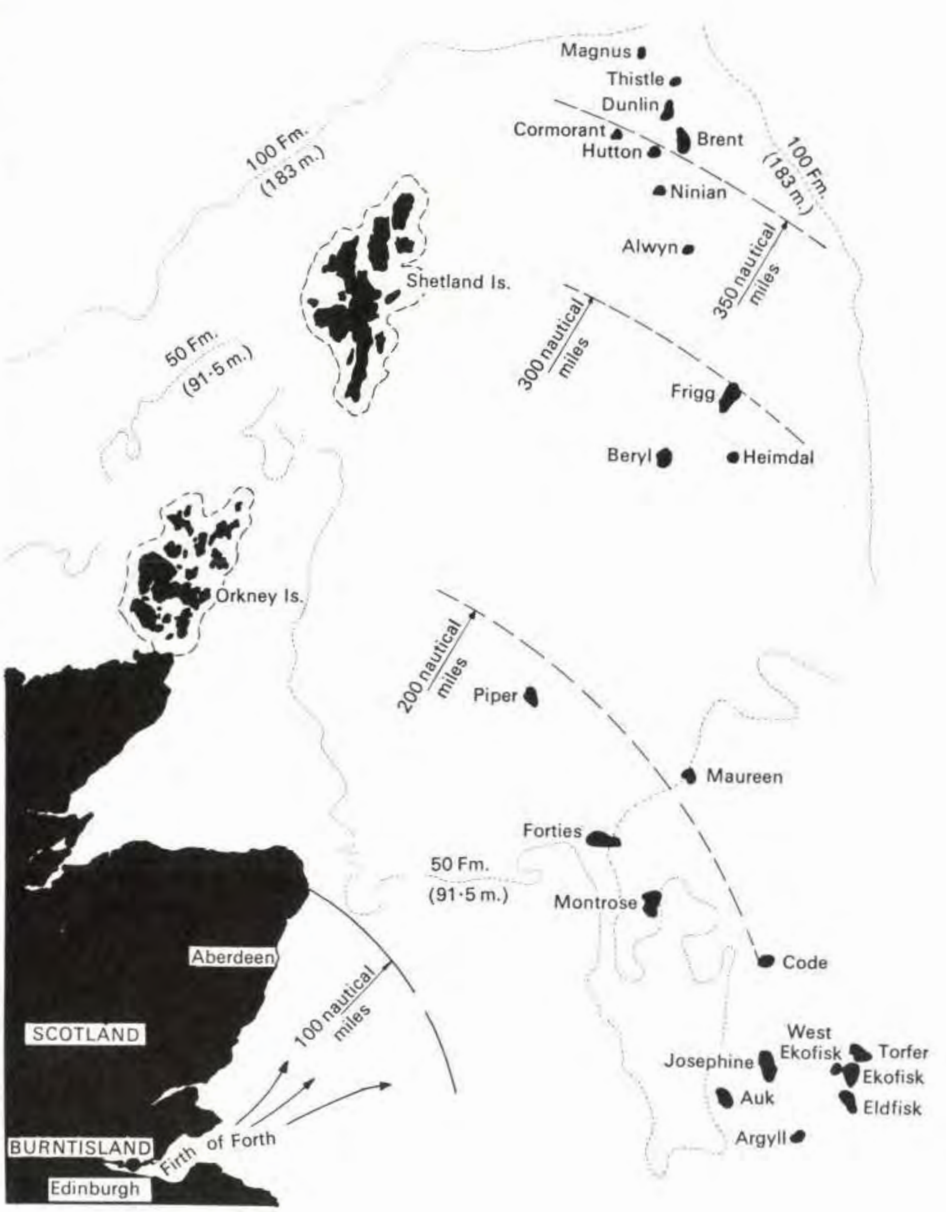

Fig. 2
Map of North Sea

The formation of Caledonian Platform Structures Ltd.

At Burntisland, Fife, 15 miles from Edinburgh and eight miles from the Arup office at South Queensferry, there is a derelict dockland site which is obviously suited to redevelopment. An examination of Burntisland shows it to have many advantages for platform building. There is access by sea, rail and road to the heart of the docks area, there is an abundant supply of skilled labour from Fife, Edinburgh and other towns, an international airport is within eight miles, and materials for making concrete are close at hand. There is no destruction of the environment involved, and the tow to the North Sea oilfields is short compared to West of Scotland platform sites.

There are, however, disadvantages. The construction site available is small, about 25 acres, and the tow-out channel in the Forth some miles from Burntisland has a bar which limits the draft of a platform to about 23/24 m if extensive dredging is to be avoided. Of these disadvantages, the first was not serious, since accommodation for workers and offices could be had in the town itself, and infilling part of the dock was a possible means of enlarging the site. But the channel restriction means that a platform design should have a draft about two-thirds that of current designs tendered from the West Coast.

In June 1974, a firm of merchant bankers in Edinburgh, Edwards Bates & Son, heard of the Burntisland site. They were interested in expanding their North Sea activities and after informal discussion with ourselves, began to think about setting up an all-Scottish group to develop the site. But there were no Scottish-based contractors of sufficient size uncommitted to platform work, and the company finally formed reflects the Auld Alliance between Scotland and France, namely, Whatlings, Fougerolle and Spie-Batignolles. An indication of the size of the two French companies is given by their turnovers for 1974 which total £500m, making the consortium the biggest construction group in Europe. Whatlings are a Scottish company with a valuable experience of work throughout Scotland, particularly in Fife. Caledonian Platform Structures Ltd was formally set-up in September 1974, with headquarters in Glasgow, and Ove Arup & Partners were appointed their consulting engineers.

Feasibility study

We were first appointed to carry out a detailed feasibility study of the site and the tow-out channel, including planning consents and a preliminary design for a platform. Outline planning permission was granted in November 1974.

Development of the construction site involves the making of a large excavation adjacent to the Forth to form a dry dock about 150 m square and 15 m deep (Fig. 16). Shortage of space and the nature of the ground requires different types of retaining wall for three of the four sides. After construction of the lower part of the caisson in the dry dock (Stage 1) it is moved and moored about 800 m offshore where it is serviced by a jetty (Stage 2).

A dredging study was necessary to determine the prevailing sediment transport characteristics of the Forth in the vicinity of Burntisland. The probable rate of siltation and frequency of maintenance of the exit channel from the dry dock and in the river were investigated. The effect of the proposed dredging on the regime of the river with particular reference to the stability of adjacent beaches had also to be probed. The outcome of these investigations was favourable to the project.

Allied to the dredging study was a marine geophysical and bathymetric survey, necessary to confirm the figures on the Admiralty charts and check the existence of any underwater obstructions. This survey showed that the sediment overlying rockhead is predominantly silty sand inshore, and that further out the material is a clay which is able to be dredged. The presence of rock is nowhere an obstruction to the preparation of a channel of the necessary depth.

The shallow draft platform design was achieved with the aid of a 'buoyancy jacket' (Fig. 3) enclosing the 115 m square base. Compressed air in compartments under the base is proposed. This proposal has approval from Noble, Denton & Associates and has a precedent in the successful float-out of the Andoc platform from its first stage in Rotterdam.

The conclusion reached at the end of the feasibility study was that the site could be developed successfully for a concrete platform design having a tow-out draft not greater than 23/24 m. This report was accepted by Caledonian Platform Structures Ltd and we were then instructed to develop the design to a stage such that detailed quality assurance could be obtained from Det norske Veritas.

Quality assurance

This has been obtained from Det norske Veritas in two stages; the first a preliminary investigation lasting three months until mid-February 1975, and the second a much more detailed investigation completed at the end of June 1975. The quality assurance covers all the aspects of the design – geotechnics, environmental loading, concrete structure, dynamic response, thermal effects due to oil processing, steelwork of deck and conductor bracing, oil storage, nautical stability and towing behaviour. It has included model tests at Newcastle and Glasgow Universities for behaviour and towing. Suitable concrete aggregates have also been located and their properties verified by laboratory testing in the University of Dundee.

Design process

Part of the challenge in producing a successful platform design lies in achieving a compromise between many conflicting demands. This conflict has been particularly evident in trying to optimize draft and stability values for Forth 150. For example, increasing base wall thickness by 10 per cent drastically cuts reinforcement percentages, but although stability is barely changed, the draft increases by almost a metre, thereby requiring further expensive dredging. On the other hand, the stability is sometimes very sensitive, as in the case of changes in payload weights, due to their high centre of gravity. For Forth 150, adding 1500 tonnes to comply with oil company requirements reduced the metacentric height by almost 40 per cent but added only 1 per cent to the draft. Any design therefore must be a compromise between many conflicting requirements.

Fig. 3
Model of the CPS platform
Photo: Scottish Studios and Engravers Ltd.

One of the most significant variables for stability studies is the payload of deck, equipment and pipework, etc., carried out to location on the structure. Many oil companies now ask for deck modules to be added at the construction site, and for Forth 150 this may involve the selection of a stage 3 site outside the Burntisland area with sufficient water depth to allow for the addition of solid ballast to compensate for the increased weight at deck level and the erection of production modules.

The design itself will now be described under the headings of environmental loading, naval architecture, geotechnics, structure and oil storage system.

Environmental loading

It is axiomatic that a good design depends as much on an accurate specification of the applied loadings as on an adequate analysis of their effects on the structure. The predominant environmental load on almost all deep-sea structures is that due to waves, whose characteristics depend on the speed, duration and fetch of the prevailing wind. The difficulties in obtaining long-term observations of the maximum storm wave conditions in the northern North Sea are formidable. Fortunately, the science of wave forecasting is sufficiently well advanced that with relatively short-term observed wave data at a particular location, and long-term meteorological information, it is possible to extrapolate with confidence to obtain a maximum design wave height or storm wave spectrum for a given recurrence interval.

The accepted design storm for an ultimate limit state analysis is that which has a maximum probability of occurrence of once in 100 years, and the associated extreme environmental criteria for a deterministic analysis are shown for typical North Sea locations in Fig. 4.

Latitude ° North	Water depth (m)	Wave height (m)	Extreme gust (m/sec)
56	75	26	47
58	105	29	52
60	135	30	58
62	165	31	61

Fig. 4
Typical environmental criteria in northern North Sea

The associated storm wave periods are more difficult to assess and consequently the effect on the design of a range of wave periods must be considered. Additionally, the frequency of occurrence of less severe sea states must be determined for evaluation of possible fatigue effects and the platform behaviour during tow-out and installation.

An alternative to the design wave method is a spectral approach. A wave record can be analyzed into a wave spectrum showing the distribution of energy density with wave frequency. Many records have been processed in this way and as a result theoretical spectra can now be derived. The advantage of the spectral probability approach over the design wave concept is that the calculated wave forces are far less dependent on the selection of design wave period. A disadvantage is the requirement of linear transfer functions which implies that the method is more applicable to large diameter members without appreciable non-linear drag forces. The method is outlined in the section on the structural analysis below.

The hydrodynamic loading on cylindrical members of the platform, i.e. legs, conductors and bracings, is determined by substituting the water particle velocities and accelerations from Stokes Fifth Order Gravity Wave Theory as developed by Sjkelbreia[2] in Morison's Equation.[1] This approach describes the applied force as the vector sum of an inertia force term, dependent on particle accelerations, and an out-of-phase drag force term, dependent on particle velocities. These values of unit loading on the leg and conductor profiles for a complete range of wave periods and phase angles are calculated by a program developed by ourselves for both Hewlett-Packard HP9830 desktop and large batch processing computer use. The steady-state current velocities are added to the wave particle velocities before substitution in the non-linear drag force term. The variation of hydrodynamic pressure and local shock pressures caused by breaking and broken-crested waves must also be considered.

The wave loading on the caisson has been determined by diffraction theory. When the ratio of diameter/wavelength exceeds 0.2 the disturbance of the flow field due to the structure is no longer trivial, and the wave scattering (diffraction) around the object becomes significant. In addition, the assumption that the wave kinematics do not change appreciably over the diameter of the structure no longer applies. With diffraction theories, the total velocity potential is obtained as a sum of the incident and reflected potentials which satisfy the appropriate boundary conditions such as zero velocity normal to the structure. The pressure distribution around the structural boundary can be obtained from this potential and integrated to compute the corresponding forces and moments.

The diffraction method for the caisson analysis is of the three-dimensional source-distribution kind as developed by Garrison and Chow[3] for calculating the hydrodynamic stress resultants on any fixed object of arbitrary shape placed in a regular progressive wave which is assumed linear and sinusoidal. The body is divided up into a number of small plane facets and a source is spread uniformly over each facet pulsating with the frequency of the incident wave. Imposing the condition of zero normal velocity across the body surface at the centre of each facet allows determination of the magnitude of each pulsating source. These sources set up a diffracted wave system which is superimposed linearly on the incident wave to obtain the total potential and consequently the applied hydrodynamic pressures, forces and moments. The version of the program we are using is one developed by Hogben and Standing at the National Physical Laboratory and available commercially as 'NPL WAVE'.

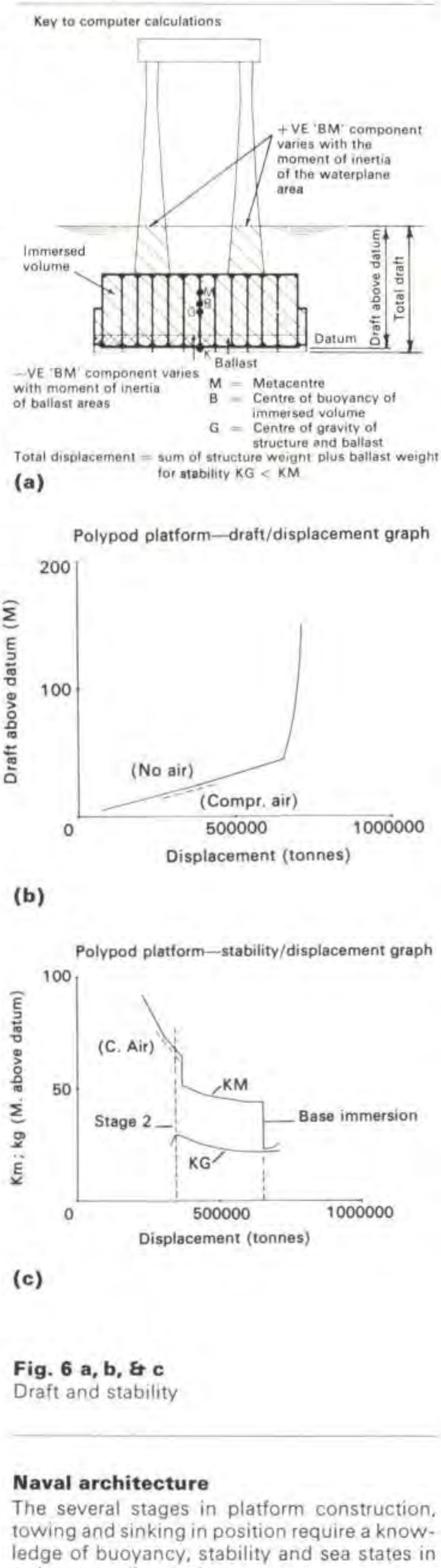

Fig. 6 a, b, & c
Draft and stability

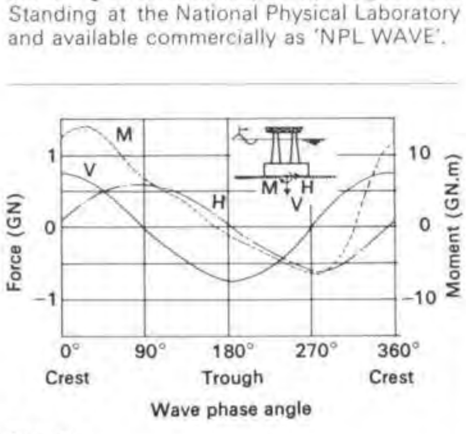

Fig. 5
Typical wave forces

A comprehensive analysis of all force resultants on the total structure, with caisson, legs and conductors, is then carried out to provide the force resultants applied to the soil at all wave periods, phase angles and directions, an example of which is shown in Fig. 5.

Naval architecture

The several stages in platform construction, towing and sinking in position require a knowledge of buoyancy, stability and sea states in order to predict motions.

The platform base raft is planned to be constructed in the dry dock (Stage 1) to an overall height of 19 m with a displacement of some 116,000 tons. To minimize the dry dock costs, only sufficient of the caisson will be built to give the required freeboard and the necessary strength to float on the crest of any possible swell while being completed at Stage 2.

At Stage 2 offshore site the structure is fully completed before towing to location in the North Sea. There are no ballasting operations at this stage, apart from trimming, so that the major marine aspect lies in the design of the moorings. Possibly three cables of 150 mm diameter chain and some 500 m long will be provided.

The assessment of the draft and under keel clearance required for Forth 150 during the passage down the Firth of Forth is a key factor in determining the feasibility of the Burntisland site, due to the Forth's limited water depths. The platform's draft will be reduced by pumping compressed air into the foundation cells below the base slab prior to its departure to sea under tow. Extensive soundings, a salinity survey, measurement of the current regime, concrete density tests, and tank model tests, have clearly established the towing channel characteristics.

Once at sea, to reduce windage and tug power requirements the platform will be ballasted first to 40 m and subsequently to 70 m draft. The latter draft with the caisson fully immersed gives less stability, but also smaller heave and pitch amplitudes and of longer periods. The stability of the platform has two distinct phases. First, a very stable phase before the top of the caisson is submerged and a second stable phase during immersion of the shafts. The minimum stability occurs as the caisson top is submerged and is an important point in the design of a platform.

On arrival over the oilfield the platform will be kept in position by tugs and sunk. Attitude control is achieved by cross ballasting. As ballasting proceeds, controlled penetration of the seabed will take place and the dissipator plates will engage. When fully ballasted the confined water is locked off and a slight deballasting is applied to stabilize the platform temporarily. The water in the confined void is then displaced by a pfa : cement grout.

Computer programs have been written for the HP9830A calculator to provide a rapid assessment for naval architecture of any particular design. Draft and stability at all stages of construction and installation, for any combination of structure, payload, and ballast weights, can be output either in tabular or graphical form, as illustrated in Fig. 6. The structure at any stage has a certain self-weight plus ballast which has a corresponding draft depending on the shape of the 'buoyancy envelope' and the sea water density. The large water plane area provides a large stabilizing component in the shallow tow mode. At location, ballast is added to sink the structure, the centre of gravity changes correspondingly, and as the roof is immersed, the large stabilizing component disappears leaving a small margin of stable equilibrium. This is illustrated in Fig. 6, graph of KM (distance to metacentre) against KG (distance to centre of gravity).

The maximum roll and heave motions of the platform had to be determined to confirm the behaviour under tow. A study was therefore undertaken with the aim of providing these amplitudes as simply as possible by developing appropriate theory and by tank model tests.

A 1/100th scale model of Forth 150 was tested at Newcastle and Glasgow Universities and covered five main areas:

(1) Wave induced vertical motion in the shallow draft conditions (Fig. 7)
(2) Towing resistance and course keeping in the above conditions
(3) North Sea tow performance, i.e. motions, resistance and course-keeping at 40 m and 70 m draft
(4) Motions during immersion when the caisson deck is awash
(5) Motions near touchdown.

Fortunately, the natural periods of heave and pitch range from 18 seconds and upwards, and are well beyond the periods of waves and swells. Regular waves were used in the tests so that the results are more readily interpreted, and spectral analysis has been used to predict extremes of motions in irregular seas. The testing has been one of the more comprehensive series carried out so far on concrete gravity type platforms.

It is not possible to model an air cushion, except in a vacuum tank, and an equal volume of rigid foam was substituted between the steel downstands and the effect of the air on initial stability was indirectly modelled by reducing the metacentric height by an appropriate amount.

A mathematical model has been developed that predicts the motion responses for concrete platforms, particularly in shallow water.

Geotechnics

The overall stability of the platform under wave loading is perhaps the most important consideration during operation conditions, hence stability analyses are made at an early stage, for in certain cases the soil conditions will control the width of the caisson. The modes of failure considered are overturning, sliding and soil hydraulic heave and two design waves are taken: the 100-year wave, and the smaller installation wave. To date stability analyses used are essentially static with extra load factors applied to wave forces to allow for dynamic effects. Overturning is analyzed by the Brinch Hansen bearing capacity method,[4] which considers an ideal plastic failure and includes for the action of vertical and lateral loads and moments. An essential element in the foundation is the provision of downstand walls, called skirts. Sliding is assumed to take place on a plane through the skirt tips and a simple shear stress check suffices at this stage. In both analyses the effect of clay softening under cyclic loading is included by applying an extra material factor. The hydraulic failure problem under platform rocking motion is analyzed by a simple heave theory, which considers hydrostatic conditions at failure; a generous safety factor is applied to allow for internal erosion.

The results of analyses show that in the case of Forth 150 and other shallow construction site platforms, caisson width is decided by tow-out draft, not soil conditions at location, assuming typical North Sea soils (boulder clay, dense sand). However, even in these soils the skirts must penetrate several metres below seabed to prevent sliding and hydraulic failure under wave loading.

Deformation of the structure under wave loading, and the dynamic amplification factors used in the above limit analyses, are derived from a simple spring-dashpot dynamic model. The soil is modelled as a boundary on the structure with two degrees of freedom represented by translational and rotational springs; both radiation and hysteretic soil damping are included. The model and its results are discussed in more detail later, but it is relevant here to mention that it clearly shows the influence of soil stiffness and damping on the dynamic behaviour of all parts of the structure. Behaviour within the soil can be studied more realistically by finite element representation, but this is not justified at the preliminary design stage.

Immediate and long-term settlements under self-weight are determined by finite element analysis[5] for this method can also provide the variations of displacements with depth and laterally for non-uniform soil. Such detail is needed to evaluate rationally soil downdrag loads on oil well casings and seabed pipeline distortion though simple hand calculations are adequate to predict settlement alone.

Forces throughout the structure depend indirectly on soil conditions but the skirts and base slab are subjected to direct soil loading. The skirts first undergo axial compression equal to the soil resistance, which is calculated by a conventional bearing capacity theory and a part empirical method based on cone resistances. Later during the installation storm lateral movement induces lateral soil pressures, with passive earth pressure as an upper-bound; residual axial force and bending from installation may still be present at this time. After grouting, the 100-year storm applies higher lateral loads but the skirts are more firmly confined laterally. Initial results suggest

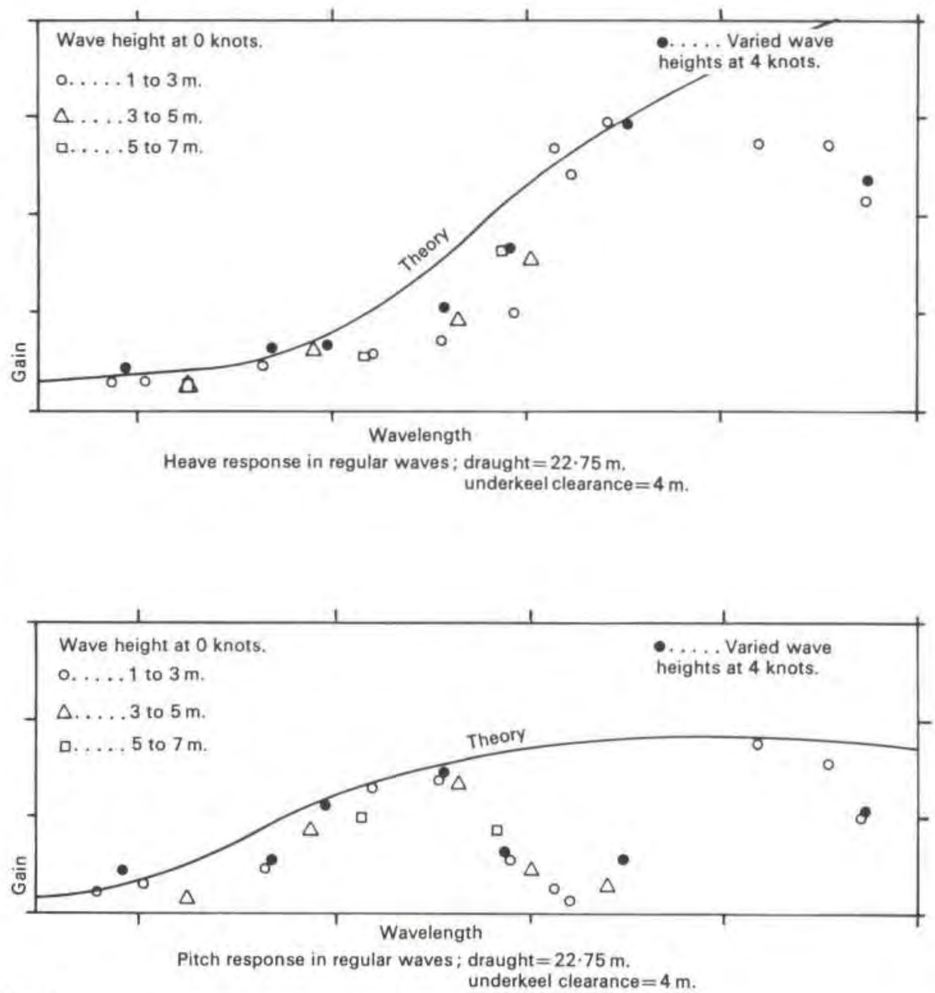

Fig. 7
Vertical motions in shallow draft conditions

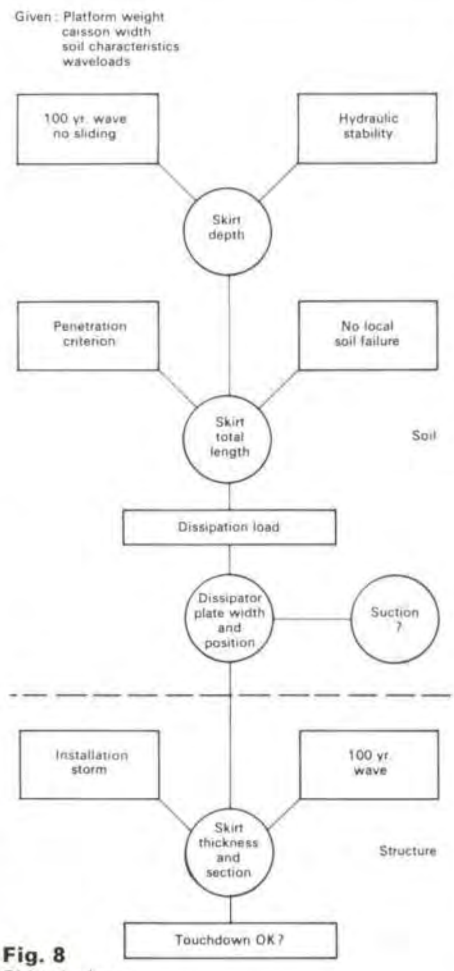

Fig. 8
Skirt design

that for most North Sea sites the skirts can be formed in corrugated steel plate, like oversize sheet piles, of 3 to 6 m depth. However, much work remains to be done on these loadcases to avoid gross over-design (Fig. 8). The prediction of soil pressure on the base if controls are not applied is also a difficult problem, though it can be controlled as described below.

During installation it is essential that the specified depth of skirt penetration is achieved, but once achieved, the platform should not over-run as this may cause irreversible tilt and may impose excessive base pressures. Accepting uncertainties of resistance calculation our solution is to assume a reasonably high soil pressure on the base for caisson analysis, and to attach 'dissipator' plates to the skirts, as shown in Fig. 9. These contact the seabed once the specified penetration is achieved; if not already completed, ballasting may then be continued with little further seabed penetration. Soil pressures on the base develop late in ballasting, by which time it can be arranged, if necessary, that the base slab no longer subject to hydrostatic pressure. This would be achieved by venting the caisson once it is full of water and thereby effectively doubling the allowable soil pressure on the base. In the unlikely event of the specified penetration not being achieved mild suction may be applied to the confined water below the base to increase the effective weight of the platform and effect further penetration.

The structural design

For the purpose of its analysis the structure (Fig. 14) can be considered as two separate but connected units; the frame comprising legs, deck and conductor bracings where structural behaviour is dictated by time-varying environmental loads, and the caisson where these effects have little influence. The two substructures thus pose very different problems and, for this reason, they were examined initially in isolation and then they were considered as one unit, the adequacy of the connection being confirmed by comprehensive stress analysis using an MSC/NASTRAN finite element idealization.

The frame itself forms a structure of considerable complexity. The three leg members are partially-prestressed tapering concrete tubes and the deck is fabricated from 1000 steel members connected at 400 joints. The scale of the structure can be illustrated by the seemingly insignificant conductor bracings which must carry loads of 3200 tonnes over a span of almost 60 m. Using a commercially available computer program this structure was idealized as a three-dimensional frame fixed to the top of the caisson and with every member modelled.

Once the structure has been idealized the loading must be defined so that the analysis can proceed. It is at this stage that a major problem may be encountered. Whilst for most structures it is evident that the maximum response in terms of stresses or deflections is given by the maximum applied load, for this structure the maximum response at a given section often occurs for applied loads less than the maximum because of dynamic effects (Fig. 10). The dynamic behaviour of the platform must be considered. The environmental loads, being caused by waves and wind, are random in nature varying in frequency, magnitude and direction of application. These wave loads are calculated using well-established wave theories and then, by utilizing dynamic analysis, the critical loads to which the structure exhibits maximum response may be given some definition.

If the foundation soil and the surrounding sea water are modelled as boundaries on the structure then the methods of analysis employed may be considered separately depending on the nature of the load input:

(*a*) ***The behaviour of the structure during the extreme design storm***

This may be approximated by studying structural response to applied loading defined over a short interval of time. The applied loads considered are those caused by single occurrences of wave characteristic of the 100-year storm or by simple combinations of waves selected to represent the likely natural environment of the storm wave. The results of this type of analysis, known as deterministic analysis, are load actions and deflections at various times during the passage of the wave or waves generating the load input. Two separate facilities were utilized for deterministic analysis. An in-house program, developed in conjunction with the R&D group, based on a simplified two-dimensional structure was used to calculate quasi-static and dynamic response simultaneously, thus defining dynamic amplification factors. A commercially available three-dimensional multi-degree of freedom dynamic analysis program, TITUS, marketed by Société de Traitement Automatique des Donness of Paris was also used. The Arup program is in two parts:

(i) a finite difference step-by-step solution of the equation of dynamic equilibrium with respect to time for the dynamic response, and
(ii) a static analysis of the structure subjected to instantaneous static loads at each time interval considered in the dynamic analysis for the quasi-static response.

The program was used to assess the variation of dynamic response of the structure placed on a range of soils and subjected to storm waves of different periods. It has been shown that the dynamic behaviour is influenced markedly by each of these variables (Fig. 11). The French program was used to provide more comprehensive information for the complete structure and for a single foundation soil. Agreement between results from the two programs was good.

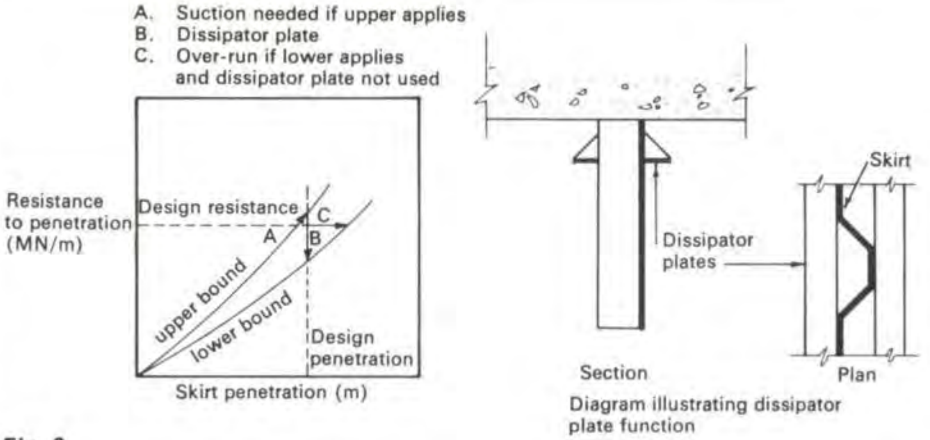

Fig. 9
Dissipator plates

(*b*) ***The long-term behaviour of the structure over a range of wave periods***

This may be quantified statistically by studying the relationship between the spectrum of wave energy characteristic of a range of wave periods in the North Sea and the response of the structure in terms of the spectrum of energy absorbed by it for the same range of wave periods. The Pierson–Moskowitz spectral energy density function with parameters specified by the International Ship Structure Congress defines the energy per unit surface area of the sea. By applying a series of transfer functions, first for wave force per unit wave height then for dynamic response per unit wave force, to the ordinates of the spectrum, the spectrum of structural response is derived (Fig. 12). This spectral analysis thus gives an expression of structural response over a wide range of wave periods and gives some indication of the relative level of response at different periods. In addition, statistical expressions for design values can be calculated depending on the particular probability levels of occurrences of these values. To enable this work to proceed, consideration was given to the modelling of wave loads on structures with large and small diameter loaded members providing an upper bound to the actual loads applied.

Thus, using the methods of (*a*) and (*b*), the critical loads on the structure can be defined and design values for load actions at different points in the structure can be identified.

In-house computer programs to assess the capacity of the leg sections to resist these load actions have been developed. Programs are available which calculate the moment of resistance at ultimate limit state and allowable moment for the serviceability limit states defined by limitation of tensile strain to less than 0.0002 and by limitation of maximum principal tensile and compressive stresses to allowable levels.

A further problem experienced by structures subjected to a large number of applications of fluctuating load is the cumulative damage caused by the range of stresses set up by these loads. The continuous stress range may be divided into discrete stress blocks with stress levels defined by dynamic analysis. The number of applications of these stress blocks can be surmised from extrapolated sea data and the fatigue behaviour may be assessed, acceptable performance being defined by Miner's Law. This fatigue behaviour, especially when reinforced concrete is involved, is largely conjectural, although some experimental results are available and thus must be considered when assessing the significance of fatigue calculations.

Consideration has also been given to a number of local effects on the frame structure, including impact loading from colliding vessels, conductor bracing fixing details and the fully-fixed deck/leg connection.

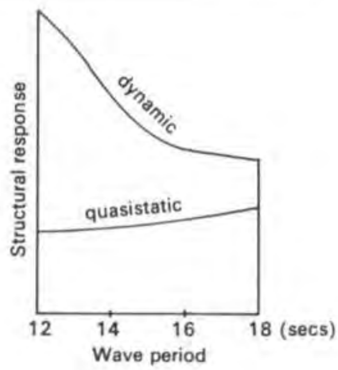

Fig. 10
Structural response

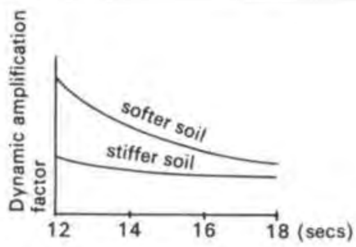

Fig. 11
Dynamic amplification factor – geotechnical variation

(*c*) ***The steel deck***

Design of the steel deck has been influenced by the presence at Leith of Motherwell Bridge (Offshore) Ltd, a firm of structural steel contractors who have experience in offshore work. A scheme has been prepared with their assistance which entails construction of the deck at Leith, floating it on pontoons across the Forth to Burntisland and jacking it upwards ahead of the concrete leg construction to its final position.

The deck is about 4500 m² in plan, comprising two 'floors' separated by the depth of the trusses. Steel of Grade 50D is proposed. The deck has to provide the necessary reactions to the tops of the concrete legs under wave loading as well as supply the working surface for oil production. It has to support about 25 000 tonnes and about 4000 tonnes of steel are used. Its rectangular plan shape is located on a three-point support provided by the legs and this geometry does not lead easily to an efficient and pleasing design. The present layout, however, is satisfactory and is in process of finalization by computer after hand calculation.

The deck is rigidly connected to the top of the concrete legs, and has been analyzed together with the legs as a three-dimensional frame. The cyclic forces induced in the deck members by wave action on the legs are less than were originally expected. This is due to the fact that

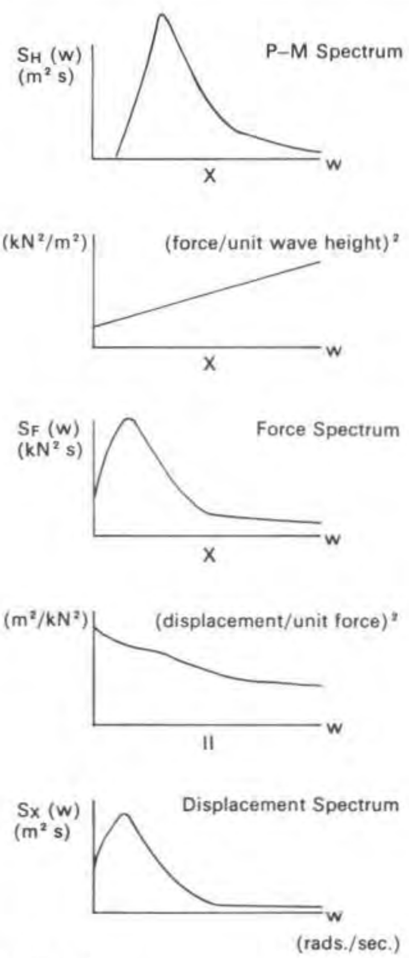

Fig. 12
Derivation of displacement spectrum

the shape of the legs makes them very stiff overall but relatively flexible towards the top. All members subjected to significant cyclic loading are checked for fatigue.

(*d*) *The caisson*

Analysis of the caisson has required the use of conventional methods in unconventional ways. Broadly speaking, design criteria for the caisson arise from load cases during installation. The hydrostatic pressures during installation induce large compressive loadings in the crosswalls which govern the thickness of these and very largely the weight, cost and draft of the structure. This fact facilitates rapid scheme designs, and calculation of the thickness of the walls is straightforward. Caisson flexure and shear at installation is more involved because these depend on the seabed topography and soil strength, which can be determined in advance only roughly. The impossibility of site preparation results is a large penalty in a gravity structure compared to a conventional land structure because the caisson may settle initially on a few hard but isolated humps on the seabed. In the absence of skirts, this would result in the structure eventually supporting about 350 000 tonnes of ballast whilst spanning perhaps 100 metres and would also mobilize local soil pressures which could not be sustained economically by the base slab.

On most soils, therefore, skirts are essential from structural considerations, as well as for overall stability. At present it is desirable to prove the structure on the most severe topography which may be encountered. We postulate a minimum of three support points on the outer edges of the base. At touchdown only skirts on the humps will be in contact but on further ballasting penetration occurs and the skirt contact length will grow as skirts nearer the middle of the base progressively begin to take load. It will be clear, therefore, that we have a situation in which the structure span is greatest at low loads; as loading is increased, the effective span diminishes. This is by no means unfavourable, of course, but it does make the critical sections for bending and shear rather hard to locate. It can further be argued that later actions at reduced spans will be superimposed on the deformed structure resulting from earlier, widespan bending.

The caisson has been modelled, therefore, as a grillage on springs along the skirt lines with stiffness derived from considerations of skirt penetration resistance. The analysis proceeds step-by-step with increments of ballast causing increments of penetration. This process we have termed 'analysis for finite settlement'. About four increments of load were generally necessary to achieve the specified penetration for the topography and soil strength assumed. The resulting structural actions were summed. Two skirt penetration stiffnesses representing extremes were chosen.

The procedure has to be repeated for the assumption of a single, nearly central support point causing hogging of the caisson. It is then possible to identify the most highly stressed walls for four cases: hogging and sagging on both 'good' and 'bad' soils. Finite element plane stress analyses, applying loads output from the grillage, were then performed. The result of this process was to demonstrate that principal tensile stresses associated with shearing action in the crosswalls would be low and the steel required for flexure would be less than that otherwise required in the roof and base slabs.

A large part of the effort so far has been devoted to the study of smaller but quite important problems. Whilst the structure is floating with the aid of the buoyancy jacket, the arch walls at each corner act without the benefit of springing thrust from the adjacent arches. This induces in the arch and the buoyancy jacket bending effects which required careful study. The junction of the perimeter arches with the base and roof induces bending which we examined using thick-shell elements. The base and roof slab cells have caused some controversy. A varying thickness roof slab has been analyzed and was found to offer the looked-for economies, at least by elastic theory, in a part of the structure governed by shear-stress limitations. The

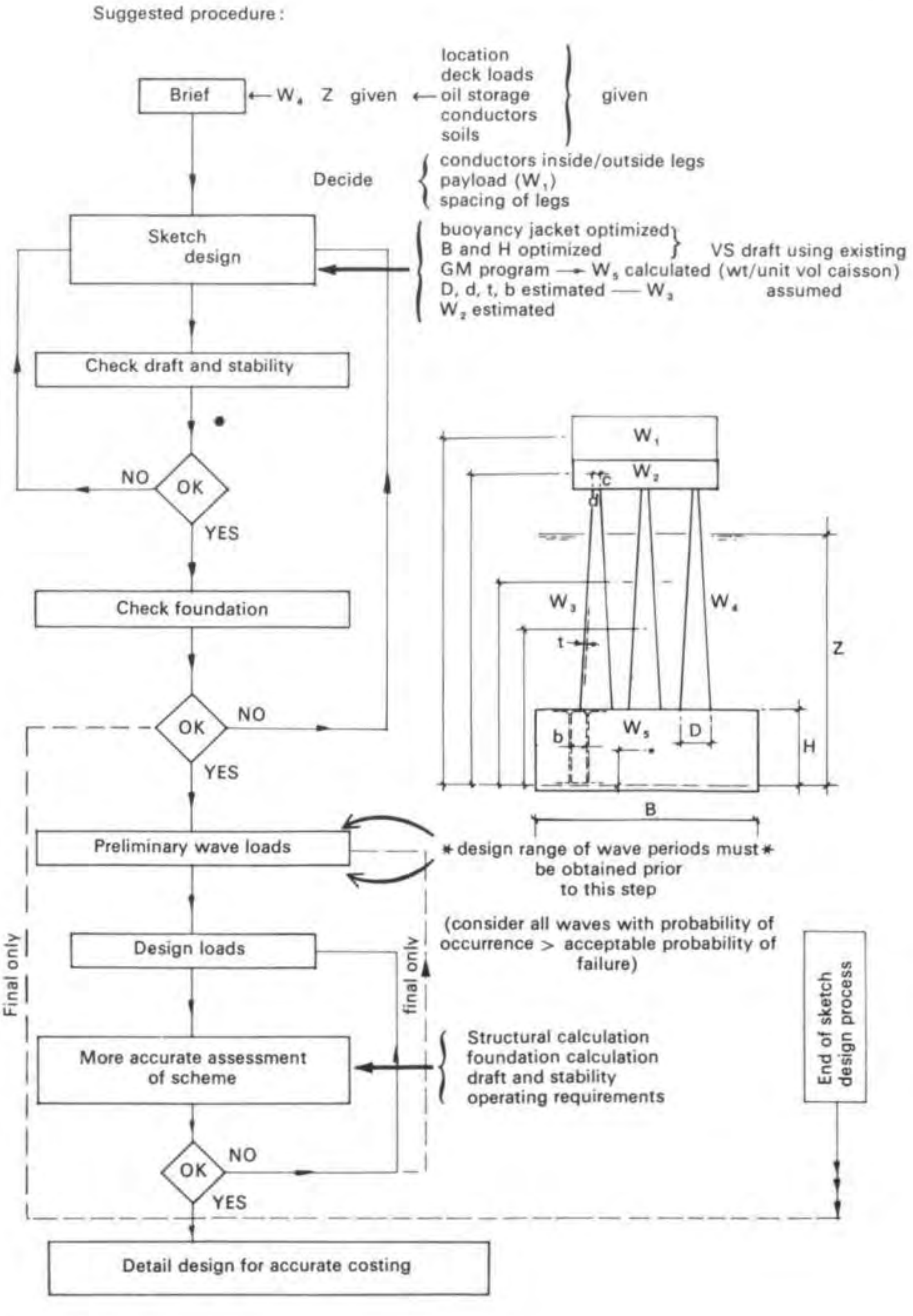

Fig. 13
Scheme design method

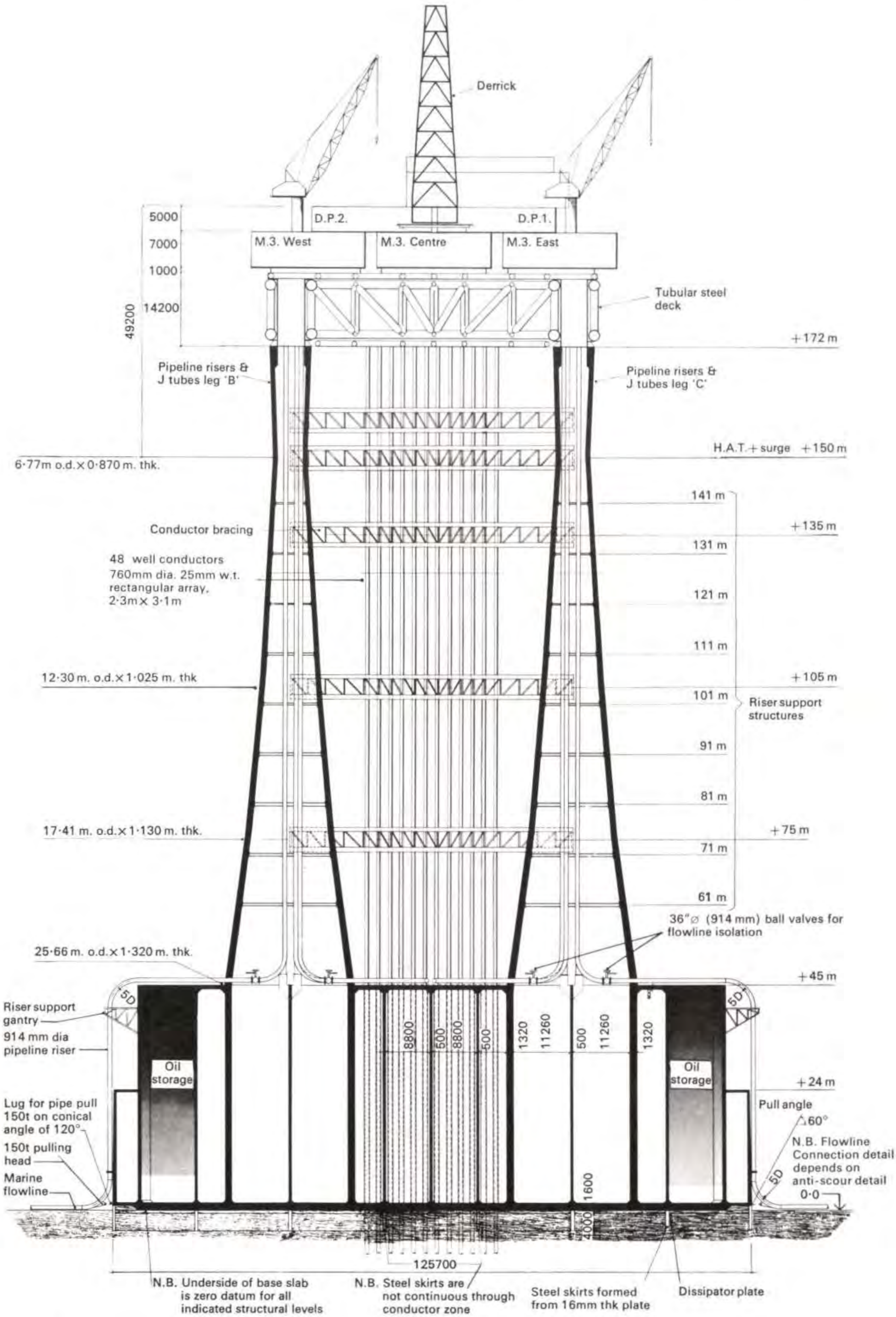

Fig. 14
Structural drawing (dimensions in mm except where otherwise stated).

shear capacity of reinforced concrete flat slabs, particularly under lateral compression, is an area in need of research.

Analysis of the stresses induced by the storage of hot oil has been of great interest. The problem divides into two main parts for economy of analysis – expansion and flexure of the cross-walls acting as a plane frame for most of their height and the more difficult problem of tensile stresses in the roof, some areas of which will be heated and others not. Several layouts of oil tanks were tried before finding one which appears suitable. Tensile membrane stresses are unavoidable in the roof except by prestressing which is awkward in the caisson. The problem reduces to one of keeping tension zones away from the oil tank areas at all times and maintaining stress levels elsewhere such that with a reasonably small steel percentage, yielding will not occur. As an additional precaution it is usual to reduce the internal pressure in oil-containing offshore structures to prevent leakage. In the event of this pressure differential being lost, cracks will open in the roof slab and it is important to ensure that there is sufficient steel to promote closure of the cracks when normal operation is restored.

In any case, vertical expansion of the tank walls will occur and cause the roof slab to be lifted in some areas. This induces bending stresses and wall shears which are studied using combined plate bending and membrane (thin-shell) elements.

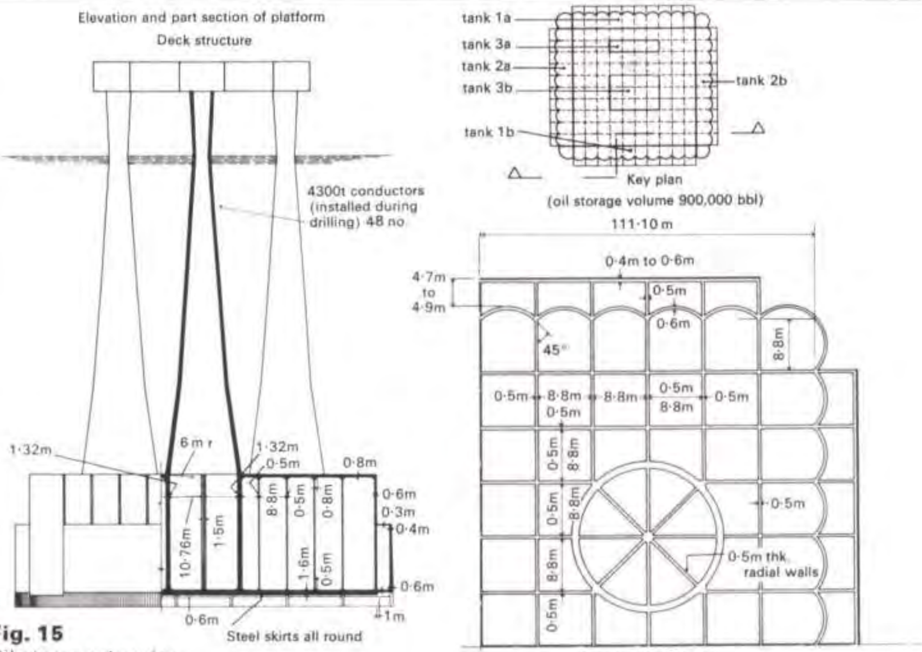

Fig. 15
Oil storage drawing

The oil storage system

The crude oil will be stored in three independent tanks comprising a total of 56 cells. Each group of cells will be connected at base level through ports to allow circulation of ballast water to the surrounding cells which in turn are connected to the bottom of the platform leg in which the control and pumping systems are installed (Fig. 15). This leg in effect acts as a sump into and out of which ballast water is pumped. This eliminates the need for installation of submerged valves or sophisticated control systems at an inaccessible level in the structure.

At the top of each of the cells in a tank group ports will be provided for distribution and circulation of the stored oil. At this level also, each tank will be connected to deck level by a separate pipeline which will serve as both inlet pipe and as a conductor for the evacuation pumps. Each tank is thus capable of being independently filled or emptied.

The crude oil from the process separators at deck level is delivered by gravity at a temperature of 40°C to the tanks via three 760 mm diameter pipelines which connect to the three storage tanks. Within the platform leg three 510 mm pump risers are connected to the 760 mm diameter pipelines. In these risers submersible pumps provide the means of lifting the oil as required to the main terminal pumps in the client's modules.

Ballast water displaced from a tank during oil filling into the surrounding cells is removed from the platform leg by three submersible pumps installed in three 510 mm diameter risers with the centre line of the pump suction 5 m below control level. These pumps are switched into and out of operation sequentially as required according to rate of oil filling monitored by water levels sensors. The ballast water removed by these pumps is passed through an oil/water separator plant situated within the deck space before being jettisoned overboard. Recovered oil is reinjected to storage via the final stage of the production separators.

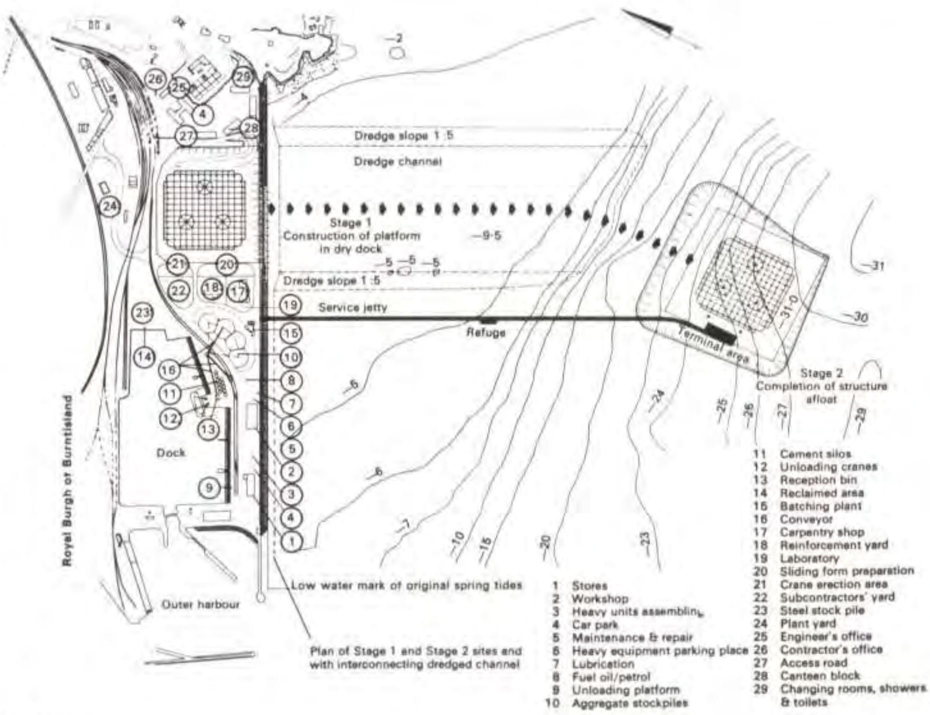

Fig. 16
Site layout

Tender design

For each invitation to bid, a design has to be produced in a very short time for perhaps quite different environmental and functional conditions from those assumed in our development work. Hence a rational procedure is required for arriving at a scheme from which detailed design can begin to evolve. The procedure must be rapid but, above all, it must produce a solution close to the right answer. Synthesis of the separate technical aspects described here has led to a quick design procedure which is illustrated in Fig. 13. Apart from giving a first scheme design it also allows us to determine quickly whether or not the Forth 150 concept can be adapted to the requirements of the brief.

Acknowledgements

The Burntisland project has employed a team of over 20 people from the Scottish office as well as staff from Newcastle office. We have had assistance from many sources in the firm, including Planning, Geotechnics, Offshore Projects Group, and R & D. No names have been mentioned in the article, mainly because it is difficult to say how much is owed to so many who have contributed in such a variety of ways. We have welcomed your support and hope that we will have it again as soon as Forth 150 wins an order!

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Bundesgartenschau, Mannheim

Edmund Happold
Ian Liddell

It lies there like a fat well-fed snake which has swallowed its prey and waits idly for digestion, while the sun reflects dully or the fresh rain sparkles on its grey skin, a hall, a multi-hall, consisting of a single roof, the most complex simple roof in the world.

Manfred Sacks – famous German architect/critic in *Die Zeit*.

Introduction

Since 1946 Professor Frei Otto of Stuttgart University has been using hanging chain nets to define possible structures in which, when inverted, the self-weight produces direct force only.[1] Fine chains cannot transmit moment and a suspended chain net can easily be used, provided it is a shear-free mechanism, to determine the statically most favourable dome shape under gravity loading for any continuous boundary condition. Such a direct force structure can theoretically be extremely thin, but its thickness will be determined by the stiffness required to withstand buckling and asymmetrical loading.

With this technique, Professor Otto was using the same method employed by the Spanish architect, Antoni Gaudi, at the end of the nineteenth century.[2] Methods of graphical analysis were popular with the engineers at that time (Fig. 1) and Gaudi developed their two-dimensional modelling into three dimensions. For the Guell Colony chapel he made wire models hung with appropriate weights to achieve in reverse a logical structure which he could amend until it satisfied him architecturally (Fig. 2a and b). This method of visualizing more complex forms gave him a buildable sculptural freedom which conventional simulation methods, such as drawing, could never have allowed.

Professor Otto went on to develop an erection method from the fact that the shape of a hung, quadrangular, chain net can be recreated in the initial shape by a flexurally semi-rigid lattice of steel or wooden rods in a uniform mesh, provided that the lattice is rotatable at the intersection points. The lattice can be prefabricated as an equal grid, pulled or pushed up and then fixed against collapse by the edge forces. Such a lattice has no in-plane shear stiffness and there are differing angular displacements at various points, the largest at the diagonal edges and the smallest near the principal axes. Shear stiffness can, to a limited extent, be provided by fixing the joints, though if the deformations are excessive, diagonal ties must be provided.

Professor Otto's first structure of this type was erected for the German Building Exhibition at Essen[3] in 1962 (Fig. 3). It was a lattice dome, on a super elliptical base 15 × 15 m, with a height of 5 m at the centre and a mesh size of 0.48 m. It was made with Oregon pine laths of 40 × 60 mm cross section and a length of 19 m achieved with finger joints. The laths were connected by bolts. The shape of the dome and lengths of the members were determined by a suspended chain net though, naturally, it was found that the flexural deflection curves of the laths were not exactly those of catenaries.

The edge beam was spiked into the ground and erection carried out with a crane with spreader beams, the base being pulled in with diagonal ties.

Later in 1962, at a seminar at the University of California, Berkeley, USA, Professor Otto conducted a series of study projects on suspended catenaries, the deflection of a dome by use of a net and, finally, a full-scale erection of a lattice dome made of round steel bars (Fig. 4).

In 1965, in collaboration with Professor Rolf Gutbrod of Stuttgart, he won the competition for the design of the German Federal Pavilion for Expo '67 at Montreal.[4] The main structure was a large continuous cable net roof but, within the roof, there was an auditorium with its vestibule covered by a timber lattice dome (Fig. 5). The plan shape was very irregular, with a re-entrant angle and spans of 17 × 13 m

Fig. 1
Model of arch formed of radiused vousoirs which allow the line of thrust to move in response to point loads. This model was made by Frei Otto to demonstrate the principle of inverting a hanging chain

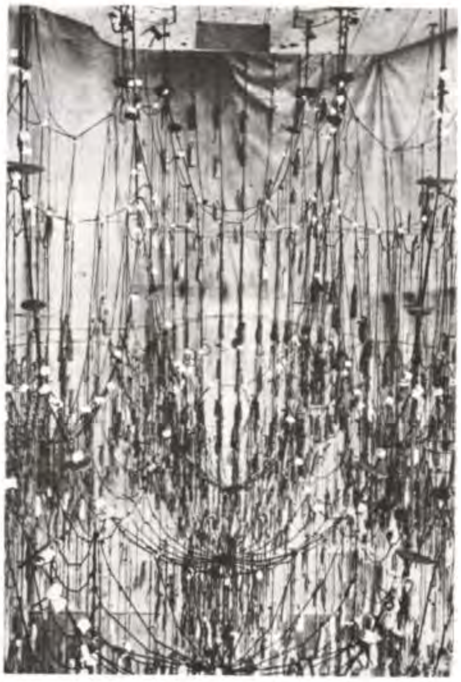


Fig. 2a
Hanging model by A. Gaudi

Fig. 2b
Detail from structure

Fig. 3
Lattice dome at Essen 1962

Fig. 4
Lattice dome at Berkeley

Fig. 5
Lattice dome at Montreal

This is an abridged version of the article which appeared in the March 1975 issue of The Structural Engineer. *It is reproduced here with the permission of The Council of The Institution of Structural Engineers.*

and 20 × 4.5 m. For this project a further refinement in the erection method was used when the lattices were prefabricated in Germany, collapsed diagonally into narrow bundles of strips side by side, transported collapsed to the site in Canada and expanded and erected there.

These three lattice domes are the only ones previously built, though several studies have been carried out, including one for a banqueting hall for the Conference Centre in Mecca, Saudi Arabia, where Professor Otto is again in partnership with Professor Gutbrod and Ove Arup & Partners. Within the last two years, however, a rigorous series of shape studies had been carried out with chain nets by a group at the Institute für Leichte Flächentragwerke at the University of Stuttgart.[5]

Nature of the structure

We use the term lattice shell to describe a double curved surface formed from a lattice of timber laths bolted together at uniform spacing in two directions. When flat, the lattice is a mechanism with one degree of freedom. If it were formed of rigid members with frictionless joints movement of one lath parallel to another would evoke a sympathetic movement of the whole frame causing all the squares to become similar parallelograms. This movement causes changes in length of diagonal lines through the nodes (Fig. 6). It is this property which allowed the lattice to be formed into the doubly curved shape of the shell.

The shape for the shell is established by photogrammatic measurement of a hanging chain model and in funicular. If the shell is loaded with its own weight only, no bending forces result. This is an ideal condition, as in practice the imposed loads on the shell are greater than the self-weight and are not uniformly distributed at the nodes. A funicular shape is an advantage but is not essential.

When the lattice has been curved to the shape of the shell, it is fixed only by its connections to the boundaries. The funicular shape is modified by the effect of bending of the laths and, with no loads applied, it would be such that the strain energy is minimized.

In this condition the lattice shell resists point loads by bending of the laths. This is accompanied by large movements of the shell and changes in the angles between the laths. These movements indicate that the overall shape of the shell can be easily altered, and to resist these movements diagonal stiffness has to be introduced.

A continuous shell made from an isotropic sheet material has equal properties in all directions. An elemental square on the surface can take direct forces and out-of-plane bending on orthogonal directions (Fig. 7). The force/displacement properties are not affected by the orientation of the element. However, an element of a lattice shell consists of a parallelogram of four laths. This element can only resist direct forces in the directions of the laths. It can also resist out-of-plane bending. In its initial pinned condition, it cannot resist diagonal forces. It cannot therefore transmit forces directly from one lath to the next.

Diagonal stiffness can be introduced in various ways:

(a) by making the joints rigid so that shear forces are carried by bending moments around the element ring

(b) by adding cross ties of cross-sectional area considerably less than the laths

(c) by adding rigid cross bracing of equal area to the laths.

Bracing type (c) would produce a shell which was directly comparable with a continuous shell. Obviously the amount of diagonal stiffness introduced by bracing type (b) can be varied by altering the thickness or material of the ties.

Load carrying behaviour

The loads on a shell can be divided into funicular loads which produce only direct forces in the laths, and disturbing loads which produce bending moments and large deflections. The deflections produced by the disturbing forces change the shape of the shell from its original funicular shape. The direct forces from the funicular loads then produce bending moments which increase the bending moments produced by the disturbing loads. As the funicular loads are increased, the stiffness and resistance to disturbing loads is decreased. At a critical funicular load there is no resistance to disturbing loads; a small deflection from the funicular shape causes collapse. This decrease in stiffness to disturbing loads characterizes compression structures. With a tension net the opposite is true; the deflections under load increase the stiffness and resistance to disturbing loads up to the point at which the members break.

With a continuous shell there is an infinite number of funicular loads. Any distribution of load without discontinuities can be carried by direct forces within the shell without the necessity for primary bending moments. This means that deflections will be much smaller than for a lattice shell where disturbing loads are carried by bending of the laths. So the effect of funicular loads in reducing the resistance to disturbing loads is less and the collapse load tends to increase.

Typical load deflection curves of a tension net, a lattice shell and a continuous shell for disturbing loads are shown in Fig. 8.

Lattice shells with diagonal stiffness

A pinned lattice shell can only carry disturbing loads by bending of the laths. Under small deflections the structure behaves as a series of inter-connected flexible arches. The bending effects are spread along the whole length of the arch. With large distortions a dimple is formed, and certain of the laths outside the dimple are stretched into tension, preventing further deflection in that zone and causing some increase in stiffness. This behaviour is only possible with low levels of funicular load. With a large funicular load the decrease in stiffness is so great that collapse will occur before the deflections become large.

These large deflections are accompanied by changes in length of diagonal lines through the nodes. In the real structure these changes in length would rupture the covering membrane unless it were made strong enough to resist them. If it is made strong enough then in effect diagonal ties are introduced, the large deflections are controlled and the collapse load is increased. Clearly it is necessary to introduce diagonal stiffness by using the membrane or ties.

Double layer grids

While ties can be used to increase the in-plane stiffness and prevent excessive shear distor-

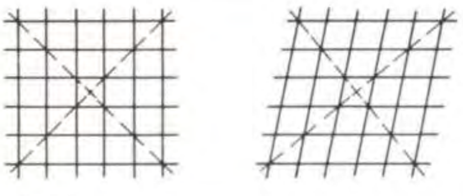


Fig. 6
Lattice distortions

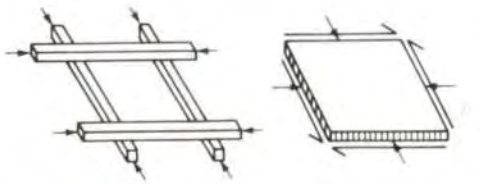


Fig. 7
Continuous shell and lattice shell elements

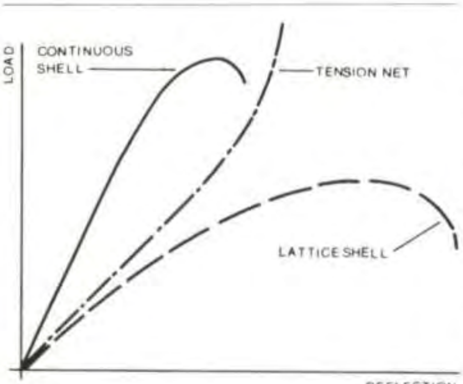


Fig. 8
Load deflection curves for different types of structure

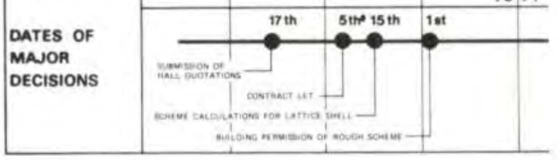


Fig. 11a
Initial programme

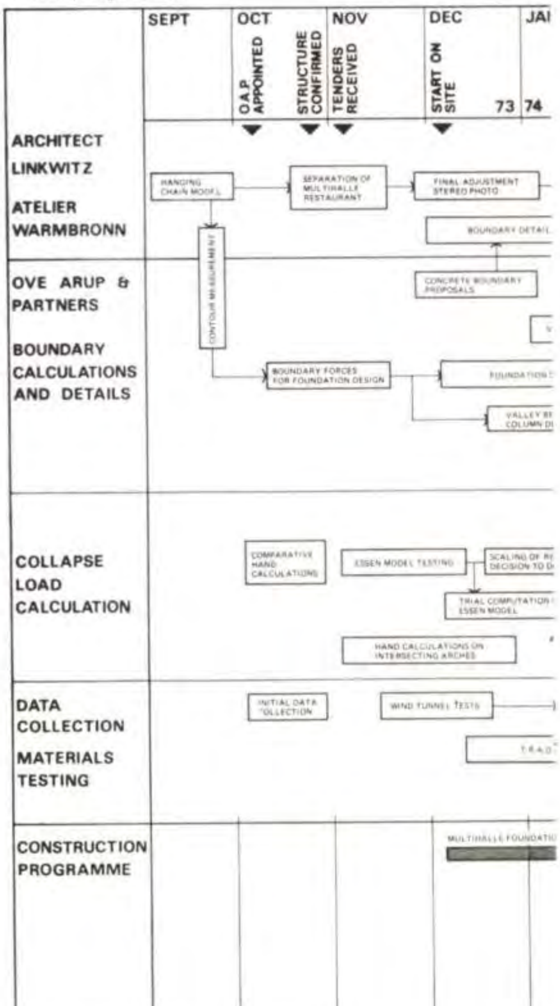


Fig. 11b
Flow chart of design process as it happened

tion, the only way to increase the out-of-plane bending stiffness is to increase the moment of inertia of the individual members. A double layer grid does this most effectively but introduces other problems. During the bending into shape the two parallel laths along each grid must slide relative to each other. To allow this to happen, one of the layers must have slotted holes. When the final shape is achieved the laths must be prevented from slipping and even when slipping is prevented the shear stiffness of the composite member is far from ideal.

ARCHITECTURAL DESIGN

A federal garden show is held every two years in one of the principal cities in West Germany. The exhibition is open for six months and about four million people visit it. It usually consists of a large park which is landscaped with flowers and shrubs, where growers and nurseries exhibit new and special strains of plant. Playgrounds are included in the landscaping, both for children and adults. Special events– concerts, theatre, games and sports – are held on most days.

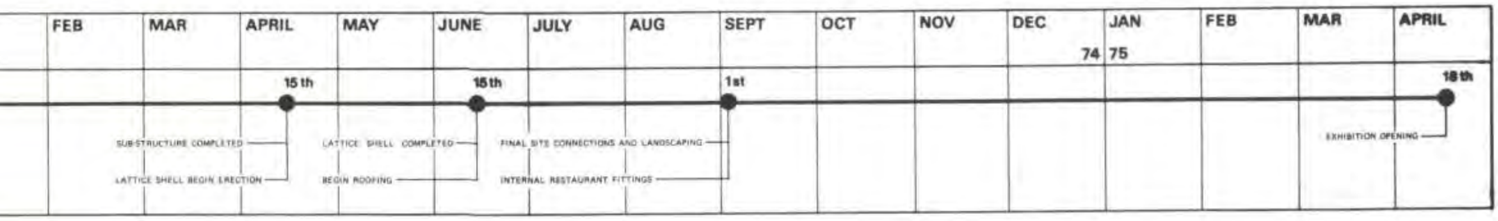
The garden show is popular among cities because a depressed open area may be re-landscaped with gardens of every kind, paths, lighting, lakes, kiosks and restaurants. The exhibition provides the stimulus to local pride to do this and brings considerable financial help.

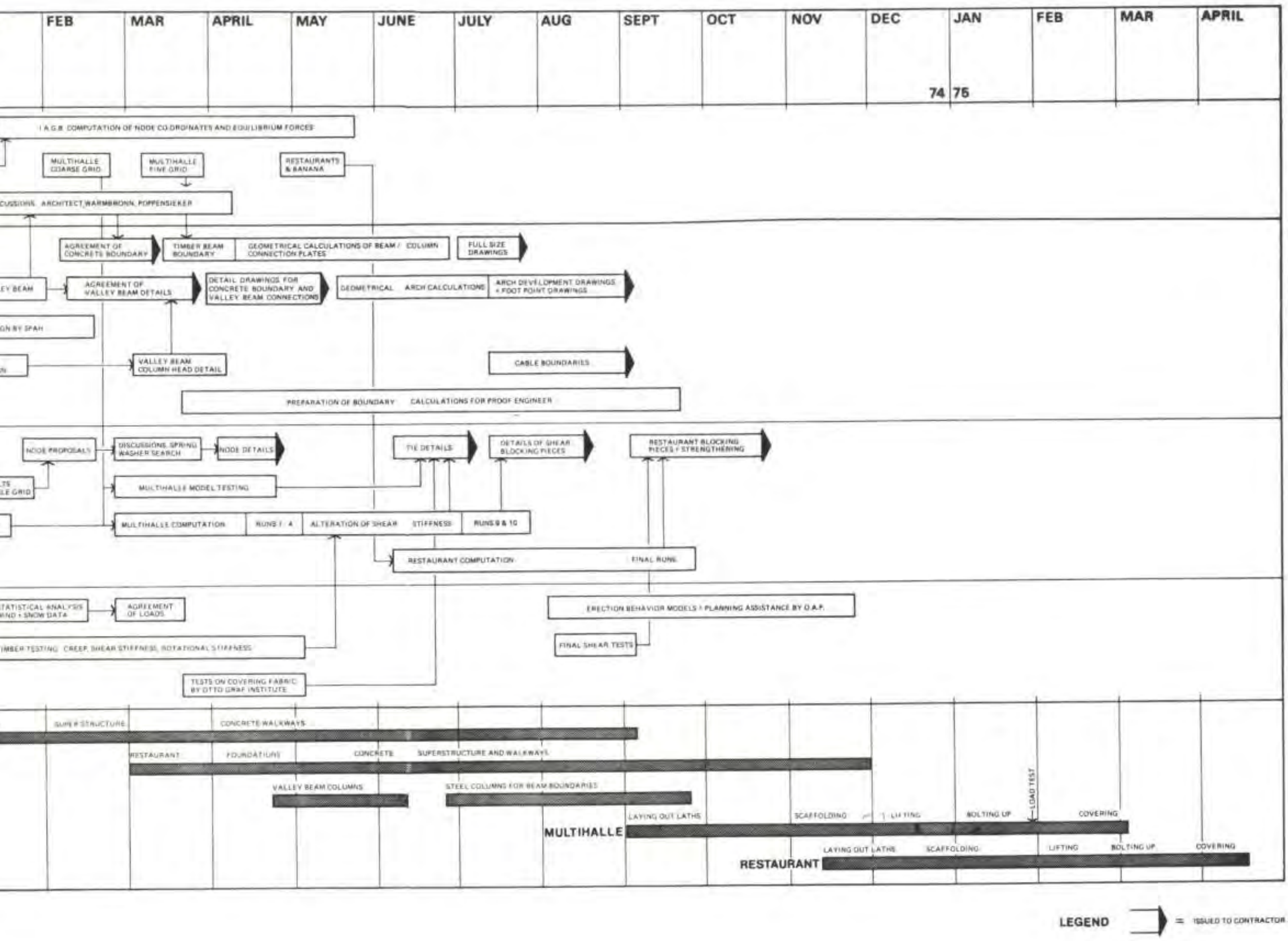
The cities of Mannheim and Ludwigshafen (joint population 600,000) were selected in 1970 as the home of the 1975 exhibition. Two sites, one on either side of the River Neckar, were planned and a cable car transport system has been erected to connect the two entry points.

A Mannheim architectural practice, Mutschler & Partners, was one of the joint winners of a competition held in 1971 to design the parks and was appointed to design the section which included a multi-purpose hall and restaurant. In their competition design the main path of

Fig. 9
Frei Otto's wire mesh model for Mannheim

Fig. 10
Hanging chain model





one of the parks led through this complex. For the Multihalle they proposed to use a membrane roof supported by balloons. This scheme was unacceptable to the building authorities, and after some more early thought they asked Professor Otto, who had designed several tent structures for previous garden exhibitions, to help them with a solution.

Together, they investigated various tent and pneumatic solutions. They felt that a free form, rounded structure would be most appropriate, as the hall was to be adjacent to the town's newly-created and only hill. Lattice shells were then considered and they evolved a scheme with two principal tunnels over the walkways. This complex was adjacent to the hill and was surrounded by an artificial canal. Fig. 9 shows the final wire mesh model that was made at Professor Otto's office, Atelier Warmbronn. The lattice was to be covered with a translucent PVC membrane reinforced with open-meshed polyester fabric.

After this model was agreed and accepted, Professor Otto prepared a hanging chain model which was to define the geometry of the roof. This model was made to a scale of 1:98.9, the node points being small rings connected by rigid links each 15mm long to represent every third grid of a 500mm mesh. The whole mesh was put together by hand and lowered onto a support system prepared on a flat marble slab marked with grid lines. The boundaries were then transferred to the support system and adjusted to find the optimum shape (Fig. 10).

A Mannheim engineering practice, Brauer Spaeh, was appointed to act as structural engineers. Professor Linkwitz and his Institut für Anwendungen der Geodäsie im Bauwesen, of Stuttgart University, were appointed to express the geometry of the chain net model by the photographic and mathematical methods he had developed during the design of the German Federal Pavilion at Montreal and the Munich Olympic stadia.

The responsibility for approving the building for public safety lies with the Board of Surveyors, but for special structures they can appoint a Proof Engineer who approves the calculations and the standard of the finished buildings. Professor Wenzel of Karlsruhe University was selected for this task.

It was decided to divide the contract into three parts: concrete work, timber and covering. Tender documents were sent out separately during September/October 1973 to three firms for each part. These firms were then put together with their agreement into three groups.

In October 1973 the consulting structural engineer resigned from his appointment and Ove Arup & Partners were invited to undertake the work. The practice was already aware of the project and knew that there was no previous engineering experience in this field. The roofs were already out to tender and the tenders were due back in three weeks. This period represented the time available to make the preliminary analysis necessary to underwrite the basic sizing. Construction on site had to commence in December 1973 and erection of the lattice shells was to take place in April/June 1974. The building was to be substantially complete in November 1974 and the exhibition was to open on 18 April 1975. This programme is shown in Fig. 11.

STRUCTURAL DESIGN

Design process

When we accepted the commission we realized that in order to meet the critical dates for supplying information shown on the initial programme, Fig. 11, design decisions would have to be made at the appropriate times, based on the information then available. In view of the lack of previous experience we anticipated a design programme which started out on a number of paths simultaneously. The flow chart of the design process as it happened is also shown on Fig. 11.

Initial studies

Tender documents had been sent out previously showing the shells fabricated as a single lattice from 50×50mm timber sizes. As only three weeks were available before the tenders were due in and they had to be accepted only a short time after that, an immediate start was made on:

(a) investigations to establish the design loads, and

(b) hand calculations on shell buckling.

As only three lattice shells had been built before, and these had been very much smaller, there was not much previous experience available. Rough calculations on the buckling of the shell based on a paper written by Wright[6] indicated that the shell was too thin. 100×100mm laths were required, but these could not be bent to shape. It was thought that the strain energy imparted to the laths in the initial bending might improve the buckling performance, although it was realized that this would only be a temporary improvement as the initial stress would creep away. This concept was later demonstrated to be irrelevant.

The building geometry was defined at this time by the hanging chain model at the Atelier Warmbronn. The final geometry was to be defined by computed co-ordinates after Büro Linkwitz had taken stereo photographs of the model and had corrected them to achieve equilibrium of forces in the net, but this was not programmed to take place for two or three months.

To get a graphical representation of the geometry, a contour drawing of the hanging chain model was prepared by Atelier Warmbronn. This drawing also showed the positions of every 10th model grid and the radii at the intersection of these grids, measured by using railway curves.

While waiting for this geometry it was felt essential to gain understanding of the structural behaviour as quickly as possible. Work was started on investigating the properties of timber and, in order to have something real to feel and test, it was decided to make a working model at one-sixteenth scale of the Essen shell in perspex strip which had a Youngs modulus a quarter of that of timber (Fig. 12). This was chosen as the geometrical details were immediately available from Professor Otto.

Four sets of tests were carried out with different conditions of diagonal restraint. These were:

(a) pinned joints

(b) glued joints, i.e. no rotation at the joints

(c) pinned with loose ties

(d) glued with nylon ties at all nodes.

Dimensional analysis was used to derive appropriate scale factors and applying these to the results demonstrated quite clearly the low buckling capacity of a single layer grid for the Mannheim shells. They also indicated the advantages of having a double layer grid with ties.

While the Essen model was being made, the tenders were received in Germany and we were formally asked to confirm that the structure would work. The rough calculations had already indicated that the shells were too thin and this was stated. However, the amount and extent of thickening required was uncertain.

It was agreed that some provision should be made for extra material and so prices were obtained for areas of doubled laths.

As work on the physical model proceeded, thought was being given to setting up a mathematical model. The ECI603[8] non-linear space frame program was thought to be the most suitable. Trial computations on the Essen model were carried out to test the modelling, which predicted a collapse load within 10 per cent of that established by load testing and so established confidence in the computer results.

As a result of these tests, it was realized that the collapse load of the shells had to be improved even though the contract had already been let and the 50×50mm lath size was agreed. In any case, increasing the lath size would mean that the initial bending stresses would be too great. Doubling the laths one above the other was thought to be the only reasonable way to get adequate bending stiffness, so the decision was taken to do this.

During this stage, work was starting on site and there was pressure to get loads for the foundation design. Using the contour drawing which gave an idea of the curvatures, calculations were made on a coarse grid to establish the direct forces in the laths. These calculations were based on:

$$\text{load} = \frac{\text{force}}{\text{radius}}$$

and on the assumption that the rate of change of force along a lath would be small because of low in-plane shear stiffness. The distribution of forces was estimated by eye and turned out to be close to the Büro Linkwitz distribution. In defining the boundary forces an allowance was made for variations in distribution.

Design loadings

From data obtained from the Weather Office the snowfall in the area is much less than the average for Germany and less than the 75 kgf/m² required as a minimum by the DIN

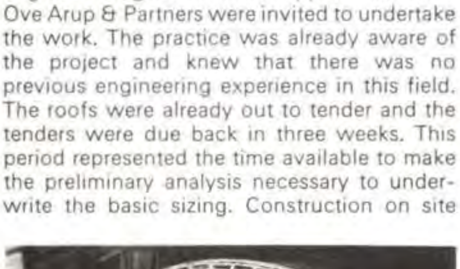


Fig. 12
Perspex model of Essen used to establish behaviour parameters

Fig. 13
Stereo photography of hanging chain model by IAGB

code. Snow was obviously the critical form of loading for a lightweight compression structure. The effects of wind were felt to be less important as the resultant force would be mainly uplift. Nevertheless, there was a possibility of gusting and dynamic effects which had to be investigated.

A 1:200 scale wind tunnel model was made and tests were carried out by the BHRA in the wind tunnel of the Cranfield Institute. Full wind and snowfall records were obtained from the weather service in Karlsruhe and a statistical analysis was performed on them to obtain a design load with a suitable level of probability.

Definition of geometry

The geometry of the shells was to be defined for construction by Büro Linkwitz and the Institut für Anwendungen der Geodäsie im Bauwesen at Stuttgart University. This started from the hanging chain model which was photographed using stereo cameras (Fig. 13). These photographs were processed to yield an initial set of co-ordinates for the nodes of the hanging chain model. The hanging chain model contained a number of inaccuracies; for example, the links were not always the proper length of 15 mm, and in places they went into compression. To imrpove this situation the initial co-ordinates were corrected so that the final set of co-ordinates represented an ideal hanging chain with the distances between the nodes exact and all nodes in equilibrium under self-weight. This processing was done on a CDC 6600 computer using a program prepared by the IAGB.[9] As well as a full set of node co-ordinates, the result of this work included a plan plot of the net and the member forces from the equilibrium calculation (Fig. 14).

Modelling

The results were made available to the architect, Atelier Warmbronn, and to us. This marked the start of detailed analysis work on the shells. The co-ordinate system was used to set up the non-linear analysis programs. The computer plot was used to provide the geometrical information to make a perspex structural model of the Multihalle similar to the Essen one and to a scale of 1:60 (Fig. 15). As with the Essen model tests, the results were scaled and the collapse load of the Multihalle shell was predicted as 63 kgf/m² with no ties and 280 kgf/m² with ties, and consequently ties were adopted for the real structure.

When the computer analysis of the shells was first considered, two alternatives were available: either to use an 'off the shelf' program or to write one specially. The second alternative held and still holds many attractions, since there were certain phenomena – for example, the shear stiffness of the double layer grid – which required special study. It was felt that it should be possible to write a program which was economical in computer time and storage if the regular nature of the grid were utilized. This last point is important since the principle of superposition cannot be used in a situation where buckling is being investigated, and thus many more computer runs are necessary than is usually the case. However, it was decided that there was insufficient time to develop a program, and also the development costs could be prohibitive. The program adopted used the method described in Tezcan and Ovunc, was written by Electronic Calculus Inc. and was used on a Univac 1108 computer with augmented core storage.

The method solves the set of non-linear simultaneous equations:

$$P_j = \varnothing_j (d_1 d_2 \ldots dj \ldots d_N) \qquad (1)$$

using a technique based on the Newton–Raphson iteration scheme. The method involves the repeated solving of sets of linear equations to obtain successive approximations to the solution. The calculation is terminated when a sufficient degree of accuracy has been obtained. The solution obtained applies to only one load distribution and load factor. The whole analysis has to be repeated for each separate loading condition.

This procedure can be expressed graphically for a structure with only one degree of freedom (Fig. 16).

If the load imposed on the structure is greater than the buckling load, then no solution can be found. Thus bounds can be obtained for the buckling load – the lower bound being the highest load at which the program converges and the upper bound the lowest load for no solution.

In setting up the input for the Multihalle program, it was necessary to use one member to represent 12 parallel laths. To ensure that such a coarse grid adequately modelled the behaviour of the real structure (made of a material with differing non-linear stress strain properties in three directions) it was necessary to make assumptions about the stiffnesses and to convert these into equivalent member properties for the computer model. The relationships between the behaviour of the computer members and that of the real structure were tested mathematically. The behaviour of the lattice joints was proved by tests.

Western Hemlock had been chosen by Professor Otto because it was available in long lengths which are normally straight-grained. The tree, *tsuga heterophylla*, is native to the western coast of America from southern Alaska to north California. It reaches a height of 60 m with a bole diameter of 2–2.5 m. The timber is non-resinous, pale brown in colour and of even texture. The darker, summer wood bands produce a well-marked growth ring figure.

The structural properties of timber depend on the direction of the stress in relation to the fibres. They also vary with moisture content and with the duration of load. The strength and stiffness properties of various species of timber have been established by a large number of short-term tests on small clear specimens carried out by several testing authorities. These results show a normal Gaussian distribution of values with a high standard deviation, but have a strong correlation with specific gravity.

For a simple element in bending, these variations in properties do not present a great problem. A design-breaking strength can be established by finding the stress below which only 1 per cent of the results will fall and then applying modification factors to allow for these effects and for additional security. Long-term reductions in E will increase deflections, but these can be taken care of by selecting a suitable beam depth.

For constructing lattice shells many of the properties of timber can be advantageous; a low Young's Modulus (E) enables it to be bent and the creep effects cause the initial bending stresses to reduce. For the subsequent behaviour most of these variable properties are disadvantageous and produce structural modelling problems. It was therefore necessary to investigate them in depth to be sure of the validity of the structural modelling.

Detailing

The details of the grid were developed in parallel with the computing work, so that the properties of the structure agreed with those of the mathematical model which was finally accepted as having an adequate factor of safety. The boundary details were developed to meet the requirements of the geometry and the erection process, and to have adequate strength to supply the reactions to the lath forces.

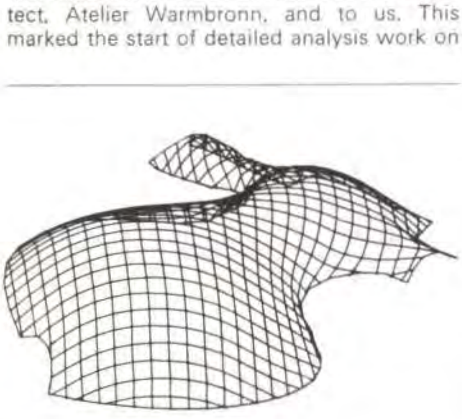


Fig. 14
Computer plot of 3 m net of Multihalle

Fig. 15
Perspex model of Multihalle under load

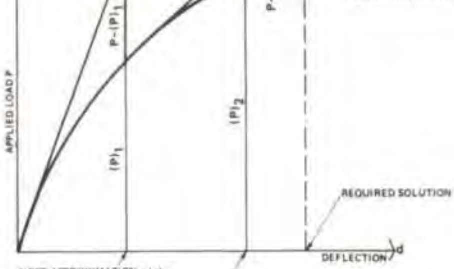


Fig. 16
Graphical representation of computer solution technique

The typical node joint

In order to take advantage of the increased stiffness of the double layer lattice it is necessary to transmit shear at each node joint, but during erection one layer has to slip over the other. This implies that mechanical connectors cannot be utilized to generate shear in the finished joint, and so friction between the timber surfaces must be used. To achieve adequate friction there has to be a clamping force which must be maintained when the timber shrinks. This can be effected with a bolt and spring if the range and stiffness of the spring is adequate.

Report on tests carried out previously by TRADA suggested a coefficient of friction of 0.47 with the timber stressed to its allowable bearing stress. Initial calculations indicated that a shear force of 150 kg would be required, which meant that a clamping force of 400 kg would be suitable. This force gave a bearing pressure at about the same level as the allowable stress quoted in *CP 112*. At this level, creep was not thought to be a problem. It was anticipated that the movement of the timber between 21 per cent and 13 per cent moisture content would be around 2 per cent. It was therefore necessary to find a spring system which could maintain a force of 400 kg after 5 mm shrinkage. The final joint, Fig. 17, has four 35 mm diameter disc springs which, in series, gave a suitable load deflection curve.

This typical node was tested at TRADA to establish its shear stiffness.

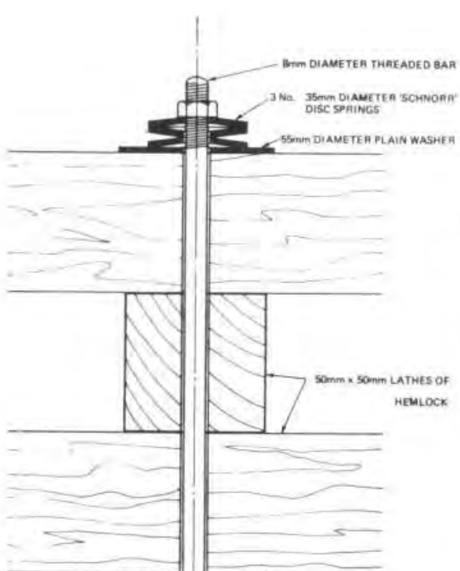


Fig. 17
Typical node joint

Ties

The non-linear programme for the Multihalle showed that a pair of 6 mm diameter 19 wire strand ties at 4.5 m/cc, through every sixth node in each direction, would provide suitable stiffness for the diagonal ties.

For the fixings at each node some small aluminium cable clamps were found which are used in electrical engineering and which use two 8 mm tightening bolts. One half of the clamp is threaded and can be used to replace the top nut on the standard node. The loose part of the clamp could be placed later and secured with another nut and an additional loose bolt. To connect the clamp to the timber a 50 mm diameter bulldog connector with an 8 mm hole was used (see Fig. 18). The same aluminium clamp was used in groups of three for the end connection of the ties connected to strips of steel, which were bolted to the edge members in various ways according to type.

The non-linear programmes also indicated that it was necessary to increase the out-of-plane shear stiffness in members which had a high axial load. This was done by installing blocking pieces between the laths at a suitable spacing which were clamped up with bolts and spring washers.

ERECTION

The lattice shell system can be thought of as a construction technique which is related to the form finding process of using hanging chain nets. During erection the laths which have been laid out flat are lifted into shape, then fixed at the boundaries and the node bolts tightened.

The contractors had stuck to their original decision to lift the grid with a crane. We were therefore asked to find suitable lifting points which would fit the cranes and spreaders and which would produce a good shape for the shell. It appeared to be very difficult to calculate the effect of the lifting points on the shape, and it was thought that a model which scaled the weight and stiffness of the two unbolted single layer grids would provide the most information and would allow for experimenting with different spreader arrangements.

A suitable woven wire mesh was found in a builders' merchants, the weight and stiffness of which was found by measurement and proved by measuring the period of a vibrating wire. Applying scale factors indicated that if the weight of the mesh was increased to five times the self-weight, the properties would be correctly scaled for a 1 : 60 model. 10 mm lead fishing weights were used to achieve the necessary increase in weight and a model was made from the Linkwitz cutting pattern (Fig. 19). This model was taken to Mannheim and was used in discussions with the contractor to find the best lifting arrangement and to define with him the step-by-step sequence of operations.

It was then decided to push up the lattice using scaffolding towers and fork lift trucks which could lift and move laterally, the lattice being anchored with cables at certain key points to prevent collapse (Fig. 20).

Finally an area of the roof was test loaded to 1.7 times the design load, using dust bins filled with water which had been borrowed from the City of Mannheim.

The deflection of 7.9 cm maximum corresponded almost exactly with the computer calculations for this load case. This was taken as justification of one calculated factor of safety against buckling collapse.

Fig. 18
Detail of the connection

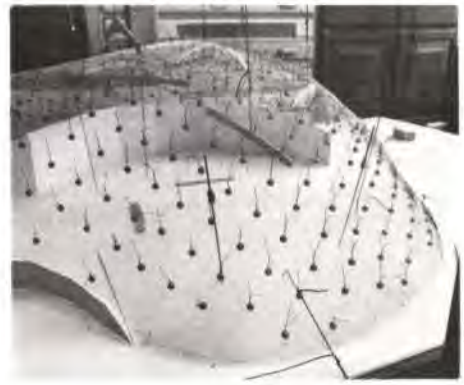


Fig. 19
Wire mesh model to simulate

Fig. 20
Fork lift truck being used to raise a tower

CONCLUSIONS

The experience was almost wholly delightful, especially since we were never suspected of having local interest and our advice was probably taken as being entirely unbiased because it was thought we stood to lose the most and gain the least according to the success of the building. There was an almost entire lack of nationalistic prejudice and we wonder if a German firm working in Britain would have been welcomed and helped quite so openly. There is a harder interface in Germany between architects and their structural engineers and the latter has more status in the public's eye, more power than the clients and more integration in the design development. This we found slightly disconcerting, though it turned out to be more apparent than real.

Like all major jobs, done to a tight time scale, it had its moments. Our favourite was when the contractor, having been on site for six weeks, told the client it would never stand up and he would never enter it. We told the client, in a meeting in Mannheim, that we should bring our resident engineers and take control on the site but told them they had to pay. To the architect's amazement the client agreed and then said when could the resident engineers be there. We stood up, opened the door, and Terry Ealey and Mike Dickson were brought in and it is largely due to their work that the site planning and site control was so successful. Many others helped: John Howells worked on the aerodynamics and timber testing, Joachim Schock on the analysis for the restaurant, Rod Macdonald on the geometry of the boundaries, Frank Pyle on the detailing and production of drawings, Robert Pearson on advice for analysis and the checking of calculations, and as big a contribution as the others, Chris Williams on the mathematical interpretations and justifications and the analysis of the Multihalle. Obviously that is not the whole list. Heribert Spaeh was our collaborating engineer, Professor Linkwitz and H. Preuss carried out the photogrammetry, and Professor Wenzel and his associates were the proof engineers. The engineer in charge of the work on site was W. Toll.

Fig. 21
Aerial view

Fig. 22
Restaurant area with cable boundaries

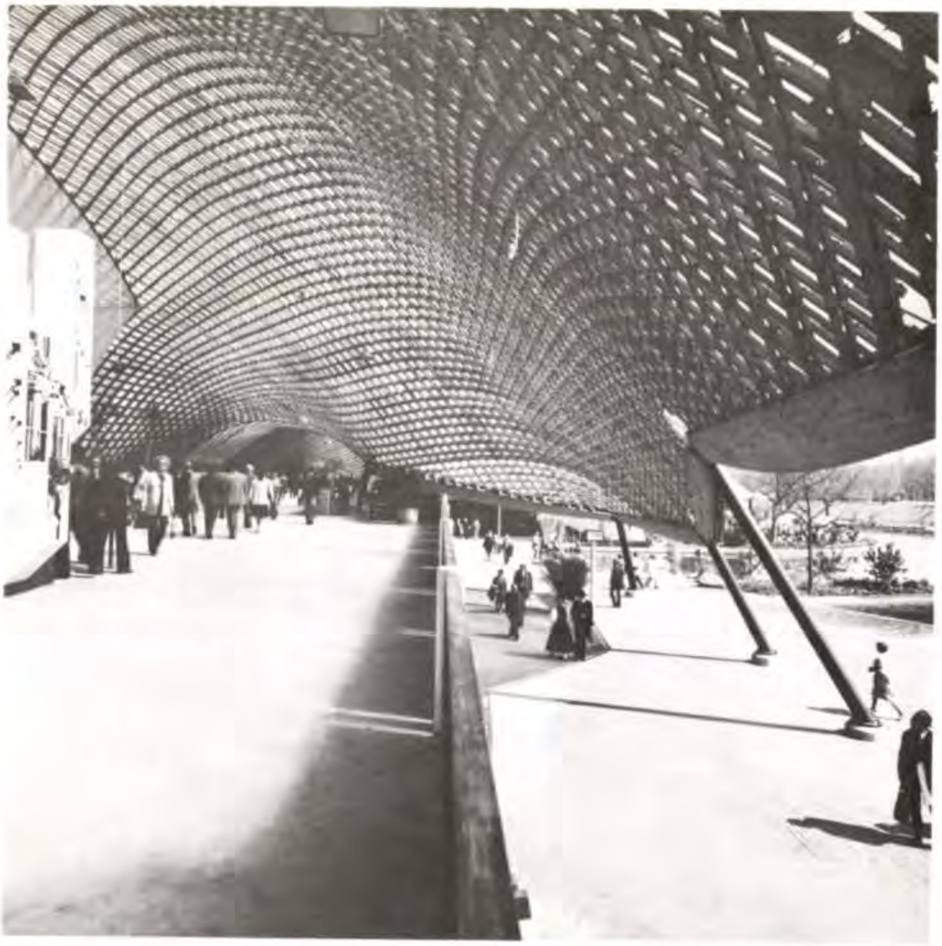


Fig. 23
Entrance to building showing beam boundaries

Fig. 24
Internal view of hall showing entrance arch

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Credits

Client:
City of Mannheim

Architects:
Mutschler & Partners

Roof consultant:
Prof. Frei Otto, Atelier Warmbronn

Main contractor:
M. Gartner

Timber contractor:
Poppensieker GMBH

Photos:
Figs. 1, 3, 4 & 9:
Institut fur Leichte Flachentragwerke
Figs. 22 & 24: Robert Hausser GDL
Fig. 23: Albrecht Brugger

The National Exhibition Centre

Richard Haryott

Introduction

This article on the National Exhibition Centre project sets out to describe in general terms the development and solutions adopted. It would be rather like trying to put a quart into a pint pot to describe all the design parameters and engineering solutions in one *Arup Journal*. Possibly there ought to be subsequent articles which concentrate upon technical details in each of the several disciplines involved.

When we set out to design we all felt conscious of the need to design within a very tight budget, and to design something that was very buildable so that it stood a chance of completion on site in the time available.

In the end, despite three-day weeks, shortages of materials, fires and other factors, it was completed on time, and although costs nationally have shot up at an enormous rate during the building it still represents remarkably good value. The whole project has cost about £25 m. of which about £5 m. was spent on civil engineering works. The halls themselves cost about £10 m. which represents a cost of about £110/m².

The completion of the development represents a lot of work by many people, both in house and outside, and each one of us who has worked on it is grateful for this effort and for the friendships made with architects, surveyors, consultants and contractors alike.

In the beginning

For many years there has been a need in the UK for a modern, large exhibition centre to display British goods and to accommodate international trade fairs. The location of such a centre has been the subject of much controversy, and several different sites and schemes have been proposed. In particular, there has been considerable debate as to whether or not such a centre would flourish outside London.

In 1970 the Birmingham Chamber of Commerce and Industry and the City Corporation jointly formed a company known as the National Exhibition Centre Ltd, and put forward a proposal for a new national centre to be built in the Midlands.

Their scheme was for a centre of some 90 000 m² (1 m.ft²) of exhibition space, together with all the necessary support, office and catering facilities which would accommodate large international trade fairs. They managed to obtain a remarkable site which could be provided with excellent communication facilities, nationally and internationally. In all, some 400 acres were purchased adjoining Birmingham's Elmdon airport and straddling the main electrified Euston to Birmingham railway line. The site also adjoins the M6 and M42 motorways, and the main A45 Birmingham to Coventry road.

The company appointed Edward D. Mills and Partners architects for the exhibition buildings and R. Seifert and Partners as architect for the external works. Ove Arup and Partners Building Engineering Division was appointed in 1971 as consulting engineers for the design of much of the civil and all of the structural and building services works, and also to carry out basic traffic studies for traffic arriving at and leaving the site as a whole.

Public inquiry

A scheme was prepared initially by the Birmingham City Architect, and an application made for outline planning consent. The application was called in by the Secretary of State for his consideration because of opposition from local residents and our first task was to prepare a case to demonstrate the engineering feasibility at the local public inquiry held in the summer of 1971. Preliminary designs were prepared to show that the site could be adequately served by main services, and that the traffic generated by such a large development could be accommodated reasonably on the major road system in the area.

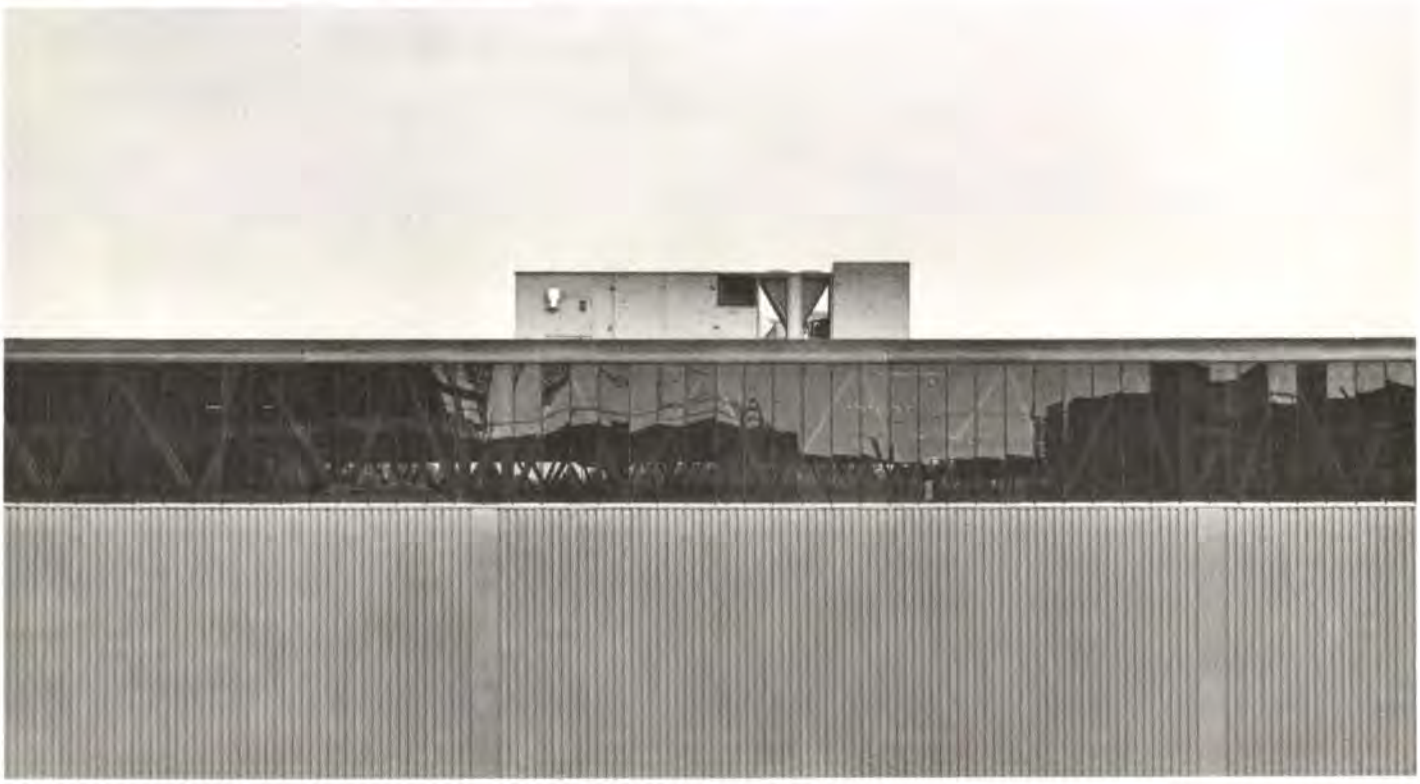
Fortunately a high pressure natural gas main already crossed the site, as did overhead lines from the national grid, and so our main effort concentrated on preliminary studies to show that foul and stormwater drainage could be provided without undue financial penalty, that water supplies for domestic and fire fighting purposes could be obtained from local mains which were already overloaded at peak hours, and, of course, on the major question of vehicular traffic.

Predictions of visitor attendance were made by the NEC based on information available from London halls, and from the major continental centres. Using these attendance predictions which showed huge variations between peak, average and minimum use, the engineering schemes were developed. Considerable effort had to be made to develop economical solutions to counter a persistent argument by opponents of the scheme, that the cost would be so high that the centre would be a white elephant. On the transportation side a fair amount of research into visitor and exhibitor behaviour patterns had to be carried out, and surveys were carried out to establish hourly flow rates for people entering and leaving exhibitions halls at Earls Court, Olympia and Bingley Hall, Birmingham. Using this data, studies were completed to predict modal splits between air, rail and road, and to show the traffic and car parking requirements for a wide range of exhibition types.

The traffic studies were carried out by our Civil Engineering Division in close collaboration with the Birmingham City Architects Department who were carrying out studies for improving the Birmingham Airport, and with British Rail, who were planning to build a new station to serve both the new Exhibition Centre and the airport.

The results of all our studies were published as written evidence before the public inquiry, and, although challenged in the usual way, did not in fact prove unduly contentious. Generally speaking the main opposition was concentrated on economic viability, green belt and amenity considerations.

Fig. 1
Exhibition Hall cladding and glazing with air handling unit. (Photo: Harry Sowden)



Design

The results of the local public inquiry were published late in 1971 and outline planning approval obtained, and the design of the Centre (which had not been taken past the preliminary studies needed for the public inquiry) started in earnest in January 1972. The NEC Ltd. wanted completion, if possible, during 1975, and it was decided to plan for a start on site on the main civil engineering works in January 1973. Because of the need for fairly substantial road and earthmoving works before any start could be made on the buildings themselves, it was decided to let a separate contract under the ICE conditions for the civil engineering works, to be followed by an RIBA contract for the buildings themselves. It was proposed to let both contracts to the same contractor if at all possible.

Tenders were sought for both contracts in October 1973, with the civil engineering contract designs substantially complete, and with the building designs in a preliminary state. This was in no small way made necessary because the NEC company itself, starting from scratch with no staff, and no in-built exhibition centre expertise and experience, was unable to complete the detailed briefing for the technical requirements in time for the buildings to be developed earlier.

Civil engineering works

The scope of the Arup commission on the civil engineering works was limited by the NEC to the design of the main roadways, the main drainage and water supply; the design of the car parks, street lighting and the co-ordination of the utilities was undertaken by Richard Seifert and Partners.

Roads

There are some 4 km (2½ miles) of main road within the site. These were designed for speeds of 50 kph generally. They were designed with economy firmly in mind, using as much material as possible from within the site area itself, and following the existing contours as far as possible to minimize cut and fill. They are all of flexible construction, with those round the site generally constructed using a dense bituminous macadam base and hot-rolled asphalt wearing courses and those round the halls using dense tar surfacing to withstand possible oil drips and general wear from heavy goods vehicles. These were constructed as early as possible within the contract, and trafficked without the wearing course.

Fig. 2
View of the roof from a corner of an Exhibition Hall. (Photo: Harry Sowden)

Surface water drainage

The site covers most of the upper catchment area of a small stream known as Hollywell Brook. This was subject to flooding previously, and a requirement of the planning permission was that the flow should be regulated in the new development. The roofs of the exhibition buildings themselves cover over 9 hectares, and together with the hard standings, roads and surfaced car parks, a very rapid run-off had to be catered for. A balancing reservoir in the form of an artificial lake of about 2 mm depth was created, covering some 6.5 hectares of low-lying land. No linings were required because of the impervious type of ground, and the flow of water is regulated by a weir which is designed to allow a 600 mm rise in the level to hold some 40 000 m^3 of stormwater which will control the flow in the brook downstream for 50 year storm conditions. The weir needs no mechanical equipment to control the flow, and thus requires very little attention. This balancing lake also serves as a pollution control and of course has considerable amenity value.

All the surface water drainage from the site gravitates into this lake through silt ponds and oil booms.

Foul drainage

The nearest public foul sewer capable of taking the NEC requirement was about 3 km from the site, and it was not possible to reach this sewer without pumping. A pumping station was built at the lowest lying part of the site to which all the foul sewers from the buildings can gravitate, and from there foul water is pumped up a rise of some 10 m over a distance of about 1.3 km and from there it gravitates through a new sewer the remaining 2.6 km.

The design of the pumping station itself, and the pumping equipment, was complicated by the need to cater for the large variation in demand. A large wet sump was constructed, served by two small pumps of 30 litres/second capacity each, and three large pumps of 105 litres/second capacity each. The order of operation of these pumps can be varied to cater for flows of 19–385 litres/second. There are two rising mains, one 250 mm diameter, the other 450 mm diameter, to cater for the varying flows.

Domestic water reservoir

Although there is sufficient water to be had from the local Birmingham Water Department mains nearby, it was a condition of planning that the NEC could extract water only at off-peak times, and mainly at night. A 3.4 m litre fresh water reservoir has therefore been designed and constructed to serve the whole site. From the reservoir two ring mains encompass the exhibition buildings, one domestic water and one fire main. Spurs from these rings serve various other buildings, including the new hotel constructed on the site. The mains are pressurized by a series of normal duty and standby pumps and the system has virtually eliminated the need for separate storage (including for sprinklers) tanks around the buildings.

The exhibition buildings

A basic NEC requirement was for all the exhibition area to be at ground level, with no basements or galleries for that purpose. It was also a requirement that large loads could be imposed at any point within the halls and that as far as was feasible it should be possible to dig up the floor to allow special foundations for abnormal loads or exhibits.

Another fundamental requirement was that the halls could be sub-divided to permit several different exhibitions to be staged at one time, and for exhibitions to be in process of assembly in one part of the complex, being dismantled in another, and in full swing in yet another.

The arrangement of five major halls of 14 010, 11 520, 18 505, 16 700 and 24 900 m^2, grouped round a central administrative and facilities area, achieves this. A sixth, smaller hall 3785 m^2 is positioned close to the main buildings, and one of the five main halls is

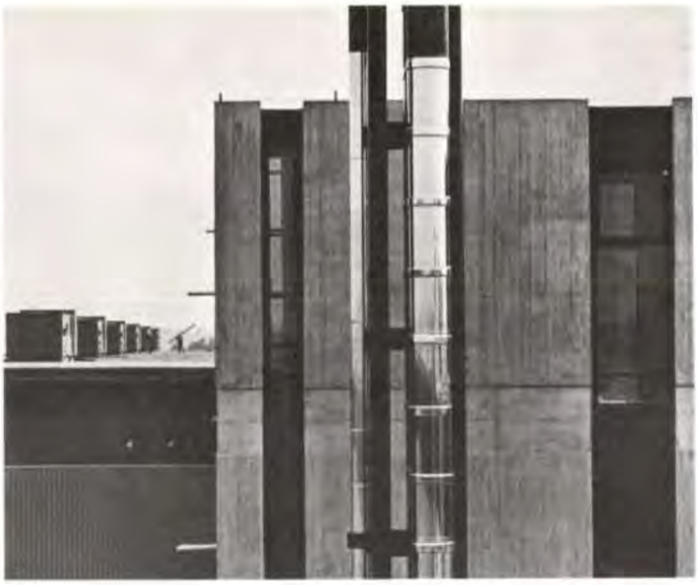
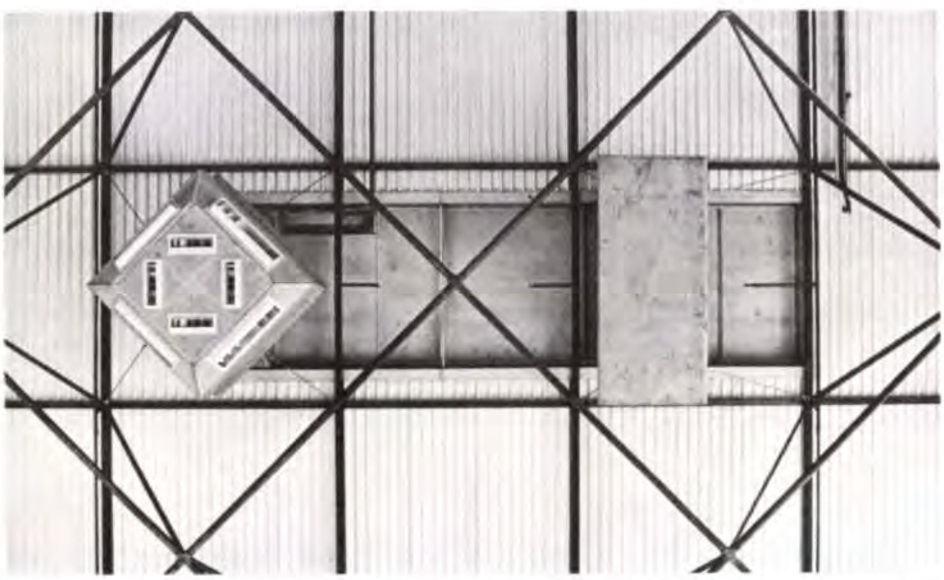


Fig. 3
Centre core north stair tower and boiler flues with Hall 4 in background. (Photo: Harry Sowden)

Fig. 4
Helicopter lifting small air handling units on to the centre core over roofs. (Photo: Harry Sowden)

Fig. 5
Erecting services in Exhibition Hall roof. Note high-pressure sodium lamps and air handling unit with supply diffuser in foreground. (Photo: Harry Sowden)

Fig. 6
Air handling unit from below showing supply diffusers (left) and extract ducting (right). (Photo: Harry Sowden)



itself sub-divisible to make seven separate areas in all.

The layout of the buildings separates goods access from public access allowing the separate operation of the halls, and a high-level bridge link from the central area to the new BR station allows pedestrian access straight from the station to the exhibition centre.

Several different schemes were investigated for servicing the stands and exhibits, including the possibility of much of the servicing being done from high level to reduce foundation costs, but the final solution adopted was to provide all stand services from a system of walk through subways, and small floor trenches with removable covers. The subways are provided with a permanent installation of gas, water, electricity, compressed air, and drainage facilities, and the trenches which cross the floors at 6 m centres allow temporary connections to be made from stands to subways. The subways are connected to main service basements containing the main electrical HV and MV transformers and switchgear, gas governors, etc.

The client wanted his exhibition space to be relatively column free and to have relatively large headrooms ranging from 12 m to 23 m. Studies showed a square column grid of about 30 m centres to be the most appropriate, and we wanted to use a system which as far as possible would eliminate the need for a mass of secondary steelwork to support services.

Several different forms of construction were investigated, but it became clear that the only practical solution was for a flat roof construction. This was partly because it allowed easy-to-build and maintain layouts of lighting, sprinklers and air handling equipment, and partly because the height specified by building safety regulations for the nearby airport ruled out very tall domed or cable stayed structures.

It was felt very strongly that it was important to choose a form of construction which would allow the structure to be erected without sequencing problems, and to allow maximum flexibility for the installation of plant and equipment during the contract, and changes during the building's life. It was also felt essential to adopt a system which would allow both structure and systems to be installed without a need for major scaffolding, as this would not only add to the cost, but would occupy valuable floor space for a significant part of the construction programme.

The light weight of the roof and the nature of the ground ruled out the possibility of fixity at the foot of the columns. It was also clear that the stability of the structure would need to be assured without any internal cross bracing, and, if possible, without external cross bracing, this last facility allowing the client to remove sections of the outer walling to link the buildings with outside exhibitions.

The structural solution adopted for the halls uses a steel frame system of multi-bay, pinned, foot portal frames. The columns (30 m centres internally and 15 m centres on the perimeter) are all four-poster, fully welded rolled hollow sections. The trusses are all welded rolled hollow sections, of Warren braced box section, providing easy access walkways and main routes for services. The box trusses provide edge support for 93 identical Nodus two-layer space frames, the upper grid being on a 3.1 m square module to suit the roof decking. The system avoided the need for extra secondary steelwork to support items of plant. To reduce the costs, shotblasting and application of the two-coat Metalife zinc rich epoxy paint system was carried out in the works, with touching-up of site welds and bolts being the only on-site painting.

A two-stage tendering process was used for the main steelwork sub-contract, which ultimately used some 7000 tonnes of structural steel, mainly tube. Preliminary tenders were sought in August 1972 to test the design and obtain realistic budget prices, and the second tender was sought in January 1973. The main contract had already been let by that stage and to ensure delivery of steel in time for the works, firm orders were placed for the supply of the bulk of the steel in December 1972, prior to letting either main contract or sub-contract. After the second tender there was a period of detailed negotiations aimed at reducing the costs by modifying the details to suit the chosen contractor's fabrication methods and facilities and by simplifying erection techniques to save construction time. This dialogue was very successful, and in many ways is an essential ingredient of good design. To match a design in detail to the skills and capacities of a fabricator and erector is common sense, and can save a good deal of money.

One area of the largest hall has a raised roof to give a clear headroom of 23 m to allow very tall exhibits, and this area is column-free over a 60 m × 60 m area. It was originally intended that some form of space frame would be used, but the proposals proved uneconomic, and simple trusses and purlins were used instead, the choice of an alternative system was very limited at that stage by the availability of suitable material.

To a very large extent expansion and movement joints were eliminated from the structure to help with roofing and glazing details, and to help with the standardization of components. This has resulted in buildings up to 210 m long without joints and without undue problems.

In the civil engineering contract the bulk of the earthworks for the halls buildings was completed. Cut and fill operations using very large earthmoving equipment produced a level site, with the fill material being a fine sand found on part of the site near the halls. Dewatering by a cut-off system of land drains around the perimeter of the buildings lowered the water table which was initially very high, and subsequently all but the deepest excavations were carried out in relatively dry conditions. Piled foundations were used throughout for the large concentrated loads of the main hall columns, and raft or pad foundations were used for all basements and the lighter weight central facilities area.

The floors of the exhibitions areas were designed as a form of road construction using a flexible pavement of leanmix concrete on a granular sub-base. It is designed to withstand a substantial load of $2^T/ft^2$ as a uniform load, and up to $4^T/ft^2$ as isolated loads. The floor surface chosen was 'latexfalt' in which cracks 'heal-up' with use, and which is a non-dusting finish. It can easily be dug up and replaced without needing skilled labour.

The 13 km of floor trenches to supply the stand services are constructed without joints in dense unreinforced concrete, and were designed to allow them to be slip-formed by modified kerb laying machines. In fact this technique was not used by the main contractor, who used a sophisticated form of conventional movable steel formwork. Floor trench covers were designed for a 13^T wheel load.

The building systems

The environment of the exhibition halls themselves is controlled by the landlord by systems mounted at high level in the roofs of the halls. This separates the landlord's systems from those in the subways beneath the floors which are used by exhibition tenants.

Lighting is by high pressure mercury discharge luminaires which are capable of being operated to give a floor level intensity of light of 200 lux or 400 lux. When operated at the lower level half the fittings are in use, and a timing device automatically ensures that the fittings are used alternatively to even lamp wear.

High level roof lights occupy about 4 per cent of the roof area, and, together with high level perimeter patent glazing (single glazed, Pilkingtons Antisun Grey) which occupies between 15 per cent and 20 per cent of the perimeter area, provide a very good level of natural light, sufficient to allow the building up and taking down of exhibitions on most days.

Externally mounted air handlings units positioned centrally on each space-frame provide heating and cooling for the space. The machines were made by the Trane Co. in America and were standard production units modified to suit the NEC needs, and to conform to British Standard requirements. Each units weighs some $5\frac{1}{2}$ tonnes and provides 40 TR cooling capacity and between 400 000 and 750 000 Btu/hr heating capacity, depending on location. The cooling capacity is sufficient to maintain reasonable standards of comfort under all but the most exceptional load conditions. Tenders were obtained from American, continental, and one British supplier

Fig. 7
Bridge link through hall to the new British Rail station

Fig. 8
Lake outfall, the hydraulic control structure and oil booms

Fig. 9
Aerial view of the National Exhibition Centre. (Figs. 7 to 9. Photos: Courtesy of R. M. Douglas Construction Ltd.)

for these units, but it was clear that the Americans, with their large domestic market for this type of equipment, were the most competitive, not only on cost, but also on capacity to manufacture the number required in the time available. The standard of design and quality of the casings to this standard product is, however, disappointingly low, even after much effort by consultant and client had succeeded in obtaining improvements.

The installation of these units could have posed problems, particularly if it had been necessary to carry this out sequentially with the erection of the structure to keep craneage distances down to manageable distances, but in the end, a special multi-tyred trolley was designed in our office to enable the units to be pushed across the completed roof without difficulty. The same technique can obviously be used at a future date to remove or replace plant.

Using the same trolley, but with modifications to change its width, heavy, externally-mounted roof-top sub-stations were also placed on the roof to supply power to the air handling units and for lighting.

The administrative and catering facilities for the halls are positioned within the central area around which the halls are grouped, or in a series of 'pods' set into the sides of the halls. The central area serves the NEC landlord's offices and suites of offices and catering facilities for exhibitors and exhibition organizers. The pods are for visitors to exhibitions.

The pods are heated and ventilated only, but the other areas are fully air-conditioned. In the central area the air-conditioning and heating is supplied from central boiler and chiller plants, with air handling plant contained in separate plant rooms or mounted externally to serve the various zones. There is also a separate plant room for each pod.

The halls are sprinkler protected to Ordinary Hazard Group I standard, with the rest of the centre to Ordinary Hazard Group III Special, the only exceptions being pod basements, subways and some other areas where sprinkler protection is undesirable. Although sprinkler protection was thought to be necessary for insurance purposes at scheme stage, they were not included in the contract at tender stage because they were not deemed necessary or economic by the insurance companies. This decision was subsequently reversed after about a year on site, and it was found necessary to proceed with what amounted to be a fairly substantial variation. The facilities for the installation were mainly there, however, and the extra work could be accommodated. Here again this was accomplished without the need for major scaffolding within the halls.

Conclusions

The debate about the suitability of the location of this centre will presumably be concluded fairly soon, but a personal view is that it must be a success. The building should undoubtedly provide an extremely effective shop window for exhibits and exhibitors. A great deal of effort and skills were combined to create it, from designers to builders, who numbered over 2000 on site at peak times.

Credits

Client:
National Exhibition Centre Ltd

Architects:
Richard Seifert and Partners
Edward D. Mills and Partners

Main Contractor:
R. M. Douglas Construction Ltd

The Metal Box Company Head Office

Nigel Baldock

Introduction

Early in 1972, after considering designs from four well-known architectural firms, the Metal Box Company selected a scheme prepared by Llewelyn-Davies, Weeks, Forestier-Walker & Bor for the design of their new head office in Reading.

The client's brief called for flexible office accommodation, conference, restaurant and recreational facilities, car parking for up to 400 cars and landscaping. The buildings were to be of 'high quality but medium price'.

The site was an area of disused railway land a few hundred yards to the east of Reading Station, lying between, and about 5 m below, the main line and Forbury Gardens. Planning requirements demanded a low structure to form the fourth side of Forbury Gardens, the colour of which would harmonize with the surrounding historic buildings of flint and Bath stone.

The office building has an open octagonal plan form which makes the most of what will be, after extensive road works, a confined site. There are four office floors over three open car parking levels: a relatively wide cross section (17.2 m) satisfies the requirements of open planned offices and double-sided parking.

A 21 m swimming pool is the main feature of the sports facilities building. Two squash courts, a bar and club room, together with a sun lounging lawn and patio, are also provided.

Fig. 1
Axonometric showing structure, duct provision for vertical services, rainwater and soilpipes

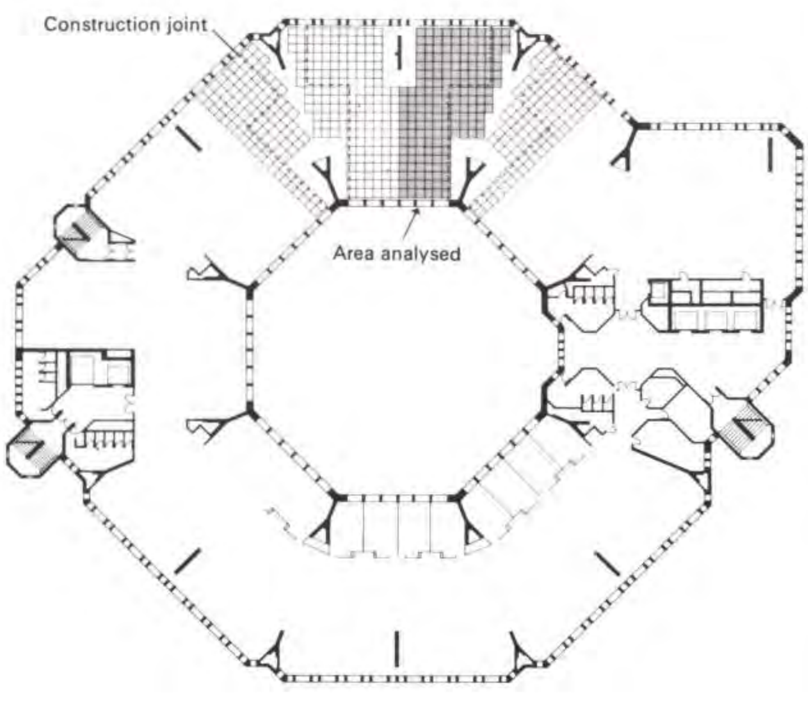


Fig. 2
Typical office floor plan (1st, 2nd and 3rd floors) showing coffer layout, construction of joints and possible sub-division of space

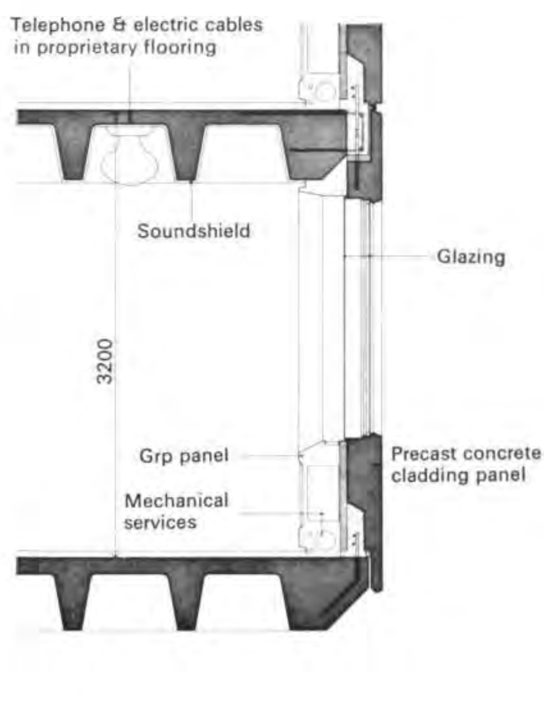


Fig. 3
Section through external wall showing relationship of precast concrete panels, services and inner GRP lining panels, also coffered ceiling, lighting and acoustic treatment

In the design of the Metal Box buildings the architect felt that he should make a very real attempt to express the inherent qualities of his building materials. Throughout the early design stages properties such as the mouldability of in situ concrete, the uniformity and crispness of precast concrete were uppermost in the team's minds.

Properties of materials are fundamental to the manufacturing and building process. Thus, bearing in mind the emphasis placed upon these qualities during the design, it seems important that the body of the text should clearly demonstrate the role of materials used in the project.

Design and construction of the office building

In situ concrete

362 cast in situ 'Franki' piles, each of 110 tonnes capacity, were installed during August and September 1972 and George Wimpey and Co., the main contractor, started work in October 1972.

Over the years of railway use a clinker/rubble crust had built up, stabilizing naturally very poor soil conditions. Our pile caps were to be constructed some 2 m into soft, silty sand through which water flowed towards the Thames (a quarter of a mile away). Despite almost instantaneous collapse of the 'upstream' side of the digs, the contractor persisted in his attempt to construct the pile caps by open cut techniques. Weeks later, having established that all the site had the same unfortunate quality, a revised approach was proposed. Steel frames were made to the size of the pile caps, sheet piles were then driven enclosing the excavation, and the digs made. This technique proved to be very successful and some of the time lost was recovered. The extremely tight programme, however, was still significantly affected.

To make up for this lost time the contractor decided to delay the construction of the ramped parking floors, which have little servicing, and go straight to the office levels, where early completion of the structure enabled a start to be made on the complex air conditioning/heating system and the roof-level plant room. As work on the office floors progressed so the contractor infilled the car parking slabs below.

A 500 mm deep coffered floor slab was chosen to accommodate a span of 13.4 m and at the same time provide a double surface area for the acoustic treatment of the soffit. (By spraying with a thick plaster the desired acoustic performance has been achieved without the need for a false ceiling. Hence the height of the building has been kept to a minimum.)

The slabs were analyzed using the ECI program 'Analysis of larger general space structures'. By prescribing suitable boundary conditions it was possible to analyze one segment only; here some 280 joints and 400 beam elements were considered, whereas had a quarter of the slabs been checked, an extremely costly 1000 joints and 1500 beams would have been included.

The office floors were power-floated to receive a modular *Buroplan* floor system which was in turn carpeted. (This form of finish achieves flexibility for the local Post Office and power services and is an essential part of the acoustic specification.) The floors were poured in 200 m² panels, with a 30 N/mm² pumped mix distributed by the permanent *Schwing* concrete pump. Although we were originally sceptical about the practicability of power floating, no problems were encountered during construction. In summer, floating was started three to five hours after pouring. In cold weather, however, the concrete was dewatered using a CCL/Tremco vacuum system prior to floating.

The soffit of the first floor, which is over the car park and entrance and exhibition area, has no applied finish. During the warm summer months achieving a good fair face finish proved to be a problem. The polypropylene GKN 'M' moulds are temperature sensitive; coffers packed tight on a warm afternoon would exhibit gaps of up to 5 mm on the following cool morning.

Each of the 21 1000 mm diameter octagonal columns, which rise from the pile caps to first-floor level, are reinforced with 25 Y50 bars. To ease congestion we specified CCL *Omega* compression splices. Despite early consultation with CCL, when it came to an order being placed, it transpired that the 50 mm splices were not readily available. Eventually, after numerous heated discussions, the splices were specially manufactured in Holland and supplied in time to have little effect upon the programme.

The use of *Omega* splices and the stiffness of the 50 mm diameter bars meant that exceptional measures had to be taken to ensure the accuracy of the placing of the column reinforcement. Permanent steel patterns were used within the pile caps and 20 mm plywood jigs used to restrain the bars during each subsequent lift.

Precast concrete

The office floors are enclosed by storey height, precast cladding units, manufactured by Minsterstone Ltd, using calcined flint aggregates. The units were cast face down and, after

Fig. 4 Elevation of the office block (nearing completion) from the Forbury road

curing, the face ground off with a hand-held disc to give a smooth, white, nearly impervious surface. The precast units were lifted into place at night, which avoided daytime monopolization of the single, centrally located, tower crane and the interruption of the other site activities. The units were hand-manoeuvered into their final position next day.

The outer pane of the double glazing system is fixed directly into the precast panel. The inner skin of the double glazing is mounted within the GRP panels that line the inside of the building. These full-height panels conceal the air conditioning/heating system which is fitted to the back of the precast units.

Fig. 5 above
Rear elevation of the office block, showing the sports centre nearing completion

Fig. 6
View of the office accommodation from the internal courtyard

Fig. 7
View of the completed job, showing the sports centre in the foreground, from the Forbury road

Internal glass fibre reinforced plastic (GRP) liners

The architect required the inside lining of the building to reflect the outside properties of the precast units. He wanted the same depth for the full height with no bulge at the air conditioning equipment. Repetition demanded the use of a material that could be readily moulded and accept the curvatures demanded by the architect. At this time GRP was the only material that satisfied these requirements and was strong and light enough to be used for internal lining units.

The GRP units were manufactured by Bourne Plastics Ltd.

During the design process great attention was paid to the stiffness of the panels. They had not only to perform satisfactorily under their own weight but also demonstrate an adequate response when sat on, leant against, etc. The desired properties were achieved by the combined effect of ribs laminated into the reverse side of the panel and the adoption of 'sandwich' construction.

In sandwich construction two outer skins of glass laminate are bonded to a low density core; the material used for the core varied between panel shape and size and the fire rating required. Intricate panels demanding a 'Class 1' spread of flame rating were constructed with a polyurethane foam core. Large flat panels could have been similarly constructed; however, stiffening ribs would have been required and these inevitably would have caused shadowing, due to local shrinkage, on the exposed face of the unit. In these situations end grain balsa was used and ribs avoided. Where 'Class 0' combustibility was required (e.g. escape routes) then *Vicuclad*, a silicate-bonded, exfoliated, vermiculate was used as the core material.

During the design process, and even the early manufacturing stages, the problems of adequate fire performance appeared almost insurmountable. Laminates that had previously achieved the desired fire rating now seemed unable to meet the standards. The resin system from which these laminates were manufactured was modified to its limit by the ever-optimistic supplier and still failed to achieve the required performance. (These failures are probably accounted for by a disparity between the original basic materials and those used during our tests.)

With the production programme daily becoming more critical, and despite the supplier's reticence, Bourne Plastics proposed a complete change of concept: rather than use a heavily filled, conventional gel coat, use an intumescent gel coat. When the gel coat is subjected to flame a coating of carbonaceous foam is formed together with inert gases that insulate the main structure of the laminate from the flame.

Indicative fire tests demonstrated that the intumescent gel coat would present a viable system. With the understanding that the specified full series of tests would be performed to confirm the indications, the contractor was permitted to proceed. The tests were carried out on samples of production panels of all types throughout the manufacturing period and have proved the system to be 100 per cent successful.

As mentioned earlier, great emphasis was placed upon the stiffness of the panels, to the extent that some had full height 50 mm diameter tubes laminated within the stiffening ribs. The first two or three panels thus constructed exhibited severe bowing parallel to the line of the stiffening tubes. An explanation of this phenomenon seemed to be found in considering the shrinkage of laminate that occurs during curing. Those edges of the panel distant from the reinforcing tube could shrink freely whereas in the areas immediately adjacent to the tube the laminates were rigidly bonded to the tube and shrinkage locally restrained. Ring shrinkage distant from the reinforcement could cause a bowing of the unit away from the free edge. Production panels were constructed without the tubes and the bowing has been almost entirely eliminated. Whilst the panels are slightly more flexible than the originals their stiffness is more than adequate.

Fig. 8
Typical office interior showing the GRP liners and the acoustic treatment to the coffered soffit

Design and construction of the sports facilities

The design concept of this building was similar to that of the office block. Two suspended coffered slabs span between shear walls and/or octagonal columns. The pool roof is a Space Deck system finished with chlorinated rubber. Terracing to the south of the Sports Building has achieved continuity of level between the roof-level gardens and the landscaped areas.

Construction commenced in June 1973. With all piles placed, ground slabs and pool base cast, the Chairman of Metal Box Company visited the site. 'I've got a bigger one in my garden', he said and disappeared. Just a passing comment? Not when spoken by the Chairman. Within a week we were desperately trying to re-design a pool twice as big using that structure already built and popping in piles just where we could. (The depth of the excavation meant that a rig could operate only in a very limited number of locations.)

Job completion

The office building was occupied in January and the pool filled in mid-July this year. The Sports Complex is due to be handed to the client in August.

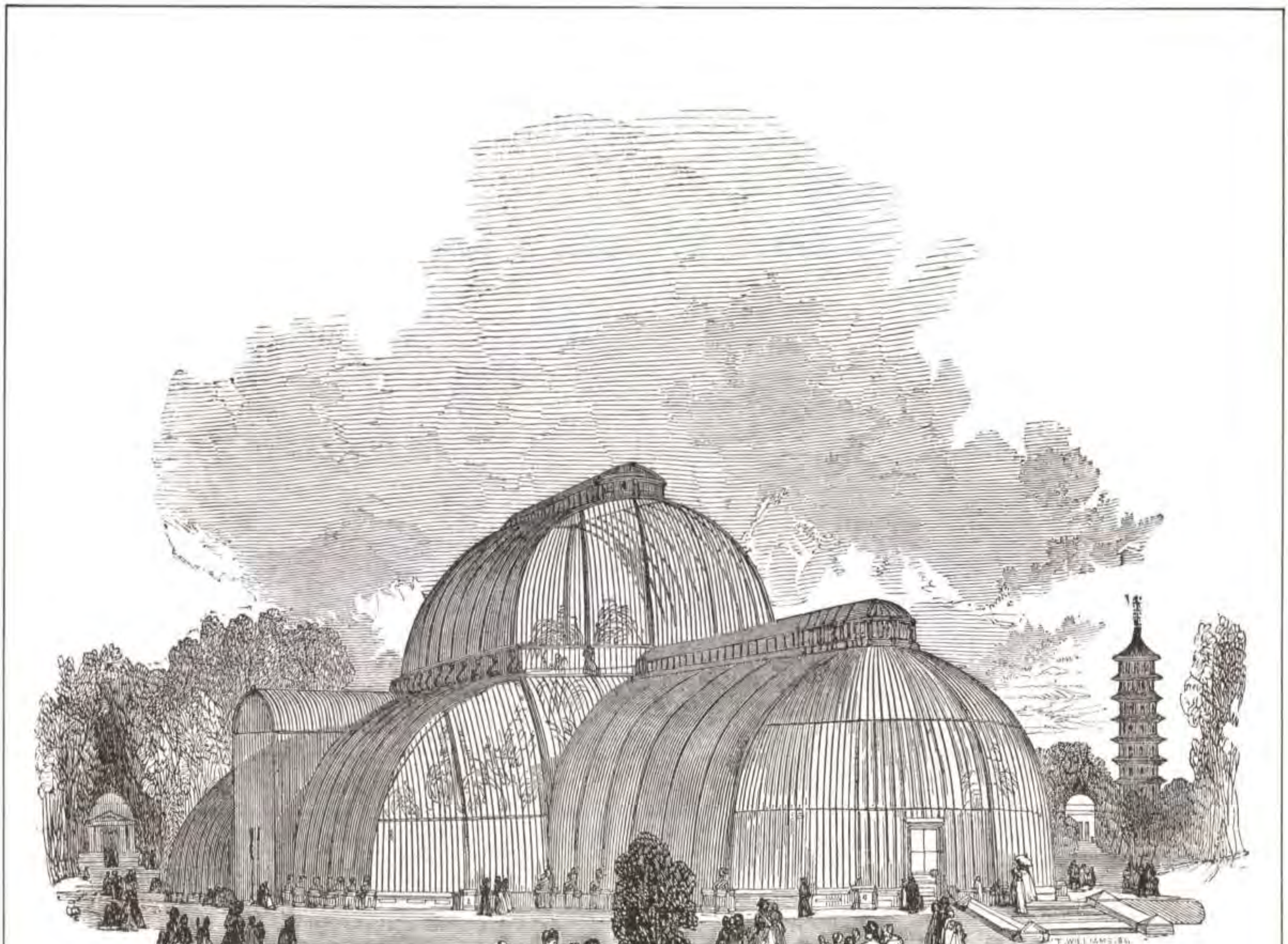
Credits

Client:
The Metal Box Company Ltd.

Architect:
Llewelyn-Davies, Weeks, Forestier-Walker & Bor

Main contractor:
George Wimpey and Company Ltd.

Photos:
Figs. 4 & 8: Henk Snoek
Figs. 5, 6 & 7: Courtesy of George Wimpey & Co. Ltd.



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