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Front cover: Montage of the Hongkong and Shanghai Bank in its location.
Back cover: Model of structure of the Hongkong and Shanghai Bank. (Courtesy of Foster Associates).

The Hongkong and Shanghai Bank

Architects: Foster Associates

Jack Zunz
Mike Glover

Preface

We are living at a time when many questions have been asked about the state of contemporary architecture and its attendant technologies. The questions and much of the consequent criticisms, however justified, have cast doubt on the validity of much of what we have done. We have become just that little less sure of ourselves.

The design and construction of the new headquarters for The Hongkong and Shanghai Banking Corporation was commissioned against this background. But the client wanted the best building he could possibly have and he chose an uncompromising architect who is committed to the technology of the day, who has an unquenchable thirst for perfection and an unerring way of achieving it.

As civil and structural engineers for this project we face an exciting challenge. The unusual geometric configuration and detail of the steel frame, the comprehensive wind tunnel studies, the construction of a deep and complex basement in difficult soil conditions, and some new developments in protective coatings to steelwork are just some of the problems which are testing the depth of our expertise. What a marvellous stimulus it is for those of us who find ourselves once again working at the frontiers of our technology.

This paper is a broad description and interim progress report of the structure for The Hongkong and Shanghai Bank. The specialist studies and investigations that we have initiated and developed will be written up separately and reported on later.

Fig. 1
Colony of Hongkong bank note

Introduction

The Hongkong and Shanghai Banking Corporation is not only Hong Kong's principal banking institution, but has developed into one of the world's major banks. The Bank's headquarters, completed in 1935, was the tallest building between Cairo and San Francisco. The first building in Hong Kong to be air-conditioned, its roof was designed to take autogyros and wiring was installed in advance of the arrival of telecasters. The building was completed in two years. At the time it was viewed as a masterpiece and an astonishing achievement since there was no comparable building in size or technical sophistication outside of Europe or the United States. The image of the building decorates the Colony's currency as if to underline its symbolic importance (Fig. 1).

By the middle of the 1970s the building had been out-grown by the Bank's requirements and its redevelopment was considered.

In June 1979 seven prominent architectural practices from the USA, Australia, Hong Kong and the United Kingdom, were invited to take part in a limited competition for a design to redevelop the site occupied by the Bank's headquarters, known as No. 1 Queens Road Central.

In October 1979 Foster Associates were announced as the winning architects. Ove Arup & Partners, who had assisted Fosters during the competition, were appointed as civil, structural and geotechnical engineers. This appointment was later extended to include project planning advice, fire and transportation engineering and acoustics.

The client's instructions to the architect, not surprisingly, were to build the best building in the world.

Redevelopment

The 5000m² site is arguably the most important site in Hong Kong, being at the head of Statue Square, the only substantial public open space in the central business district

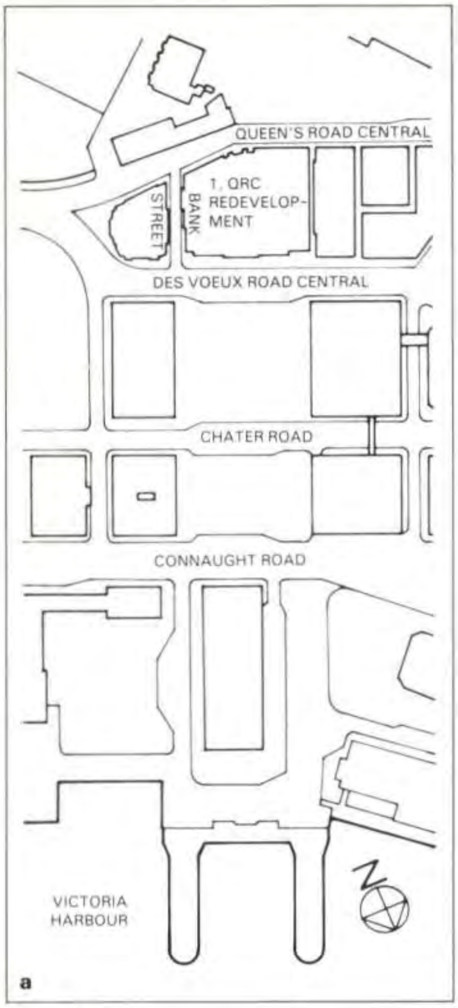
(Fig. 2). The square is a major pedestrian route to the harbour waterfront and ferry services, a distance of about 400m from the site. Recent development proposals for the square have reinforced the importance of this pedestrian route and the amenity that it provides.

The site is roughly square on plan, bounded by roads to the north (Des Voeux Road), south (Queens Road Central) and east (Bank Street) and to the west shares a common boundary with Chartered Bank.

Redevelopment of the site is constrained by a height restriction of 178m and a plot ratio of approximately 15:1. Statutory lighting angles limit the bulk of building that may be erected on any road frontage.

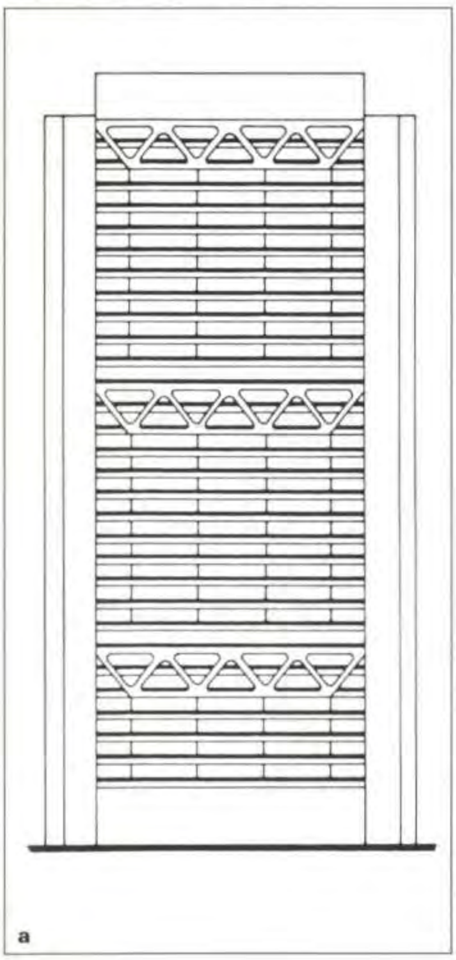
The facilities to be provided in the building are extensive and include the following:

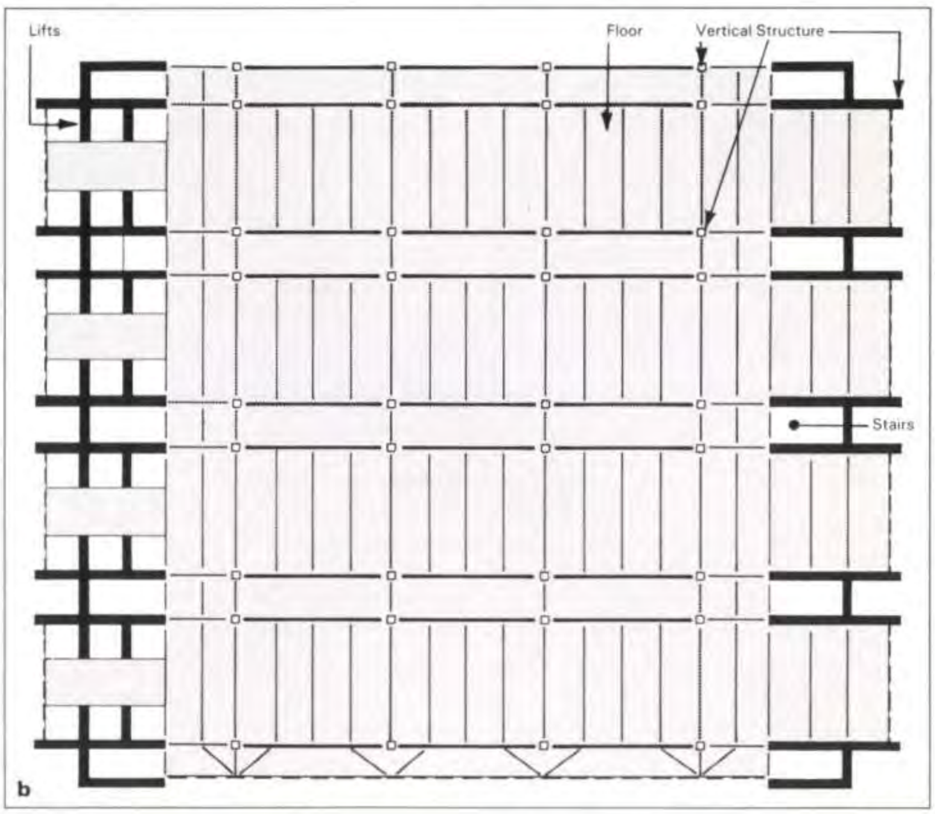
(1) Headquarters office accommodation including central computer installation
(2) Multi-level banking hall around a central atrium space
(3) Security and safe deposit vaults with secure access and unloading facilities
(4) Recreation areas, restaurant and kitchen facilities
(5) Open gardens on terraces at high levels
(6) Apartments and executive suites
(7) Potential for large swimming pool
(8) Heliport and multi-level viewing gallery at the top of the building.



Figs. 2a-b
a) Site plan
b) Aerial view

Figs. 3a-b
Competition concept
a) Elevation b) Typical plan





Design development

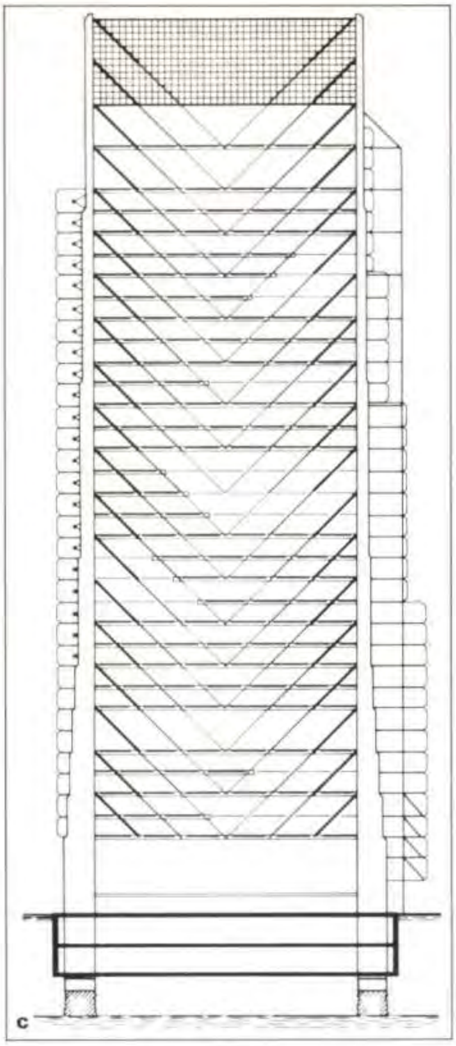
The development of the building design took place between October 1979 and January 1981 when the final concept was presented to and accepted by the Board of the Bank.

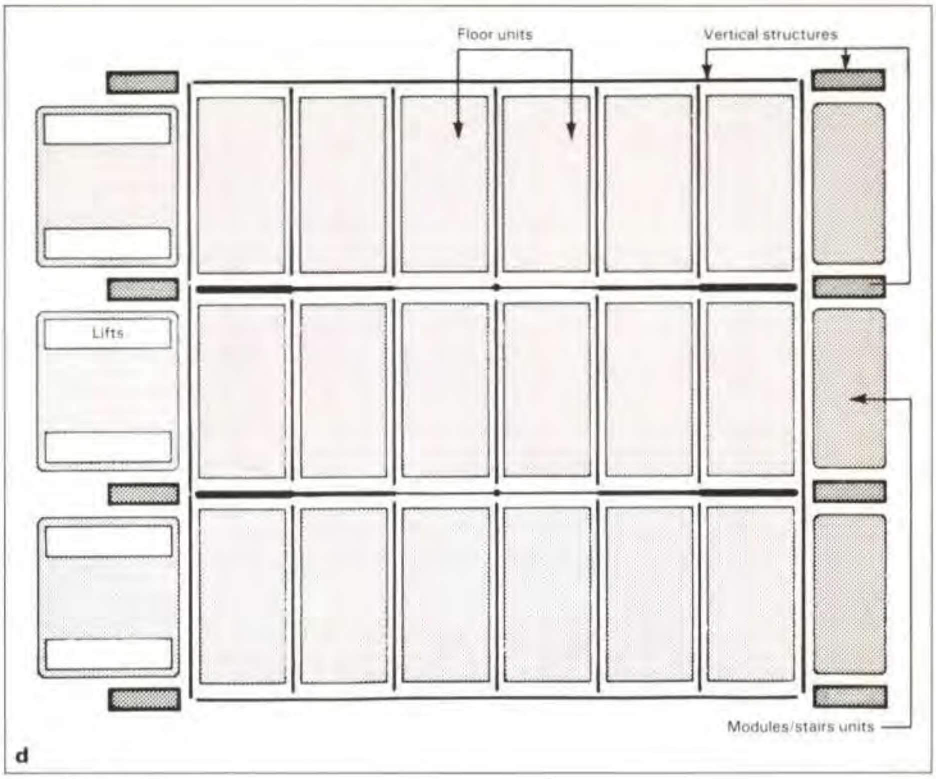
Like all design development it passed through perceptive and imperceptive shapes. Fig. 3 indicates some of the significant milestones.

The key features of the architect's concept that have shaped the development of the structural frame can be summarized as:

(1) *Flexibility of internal planning*

This was achieved by reducing the number and size of vertical structural and servicing elements within the centre of the building and providing special multi-level spaces.





Figs. 3c-d
Chevron concept
c) Elevation d) Typical plan

The size and proportion of the individual elements have been selected to reduce visual and planning obstruction: wall elements have been discriminated against.

The design floor live loadings are greater than the statutory recommendations, principally to provide flexibility in the future arrangement of storage areas, computing facilities and safes on the general floors. In addition, special high loading allowances have been made within the building to accommodate multi-use spaces including terraces, gardens and a swimming pool. The vertical loading for this building is estimated to be 20% higher than for a commercial building of similar proportions.

(2) *Large open space at ground level*

This space forms a physical continuation of Statue Square and has been dedicated to public use. In recognition of this dedication the maximum plot ratio has been granted for this building.

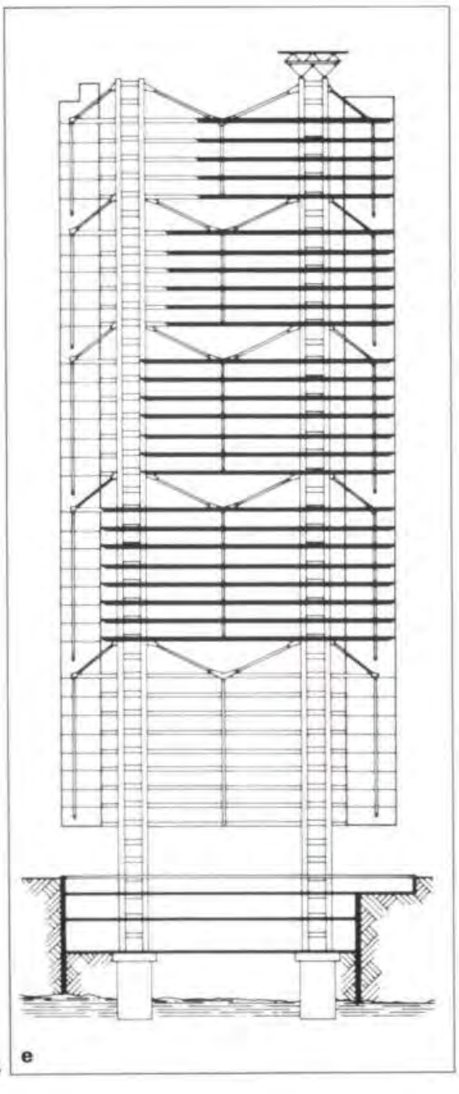
The requirement to provide an 'open' space has limited the 'footprint' of the structural frame at ground level to localized tower structures to the east and west of the site.

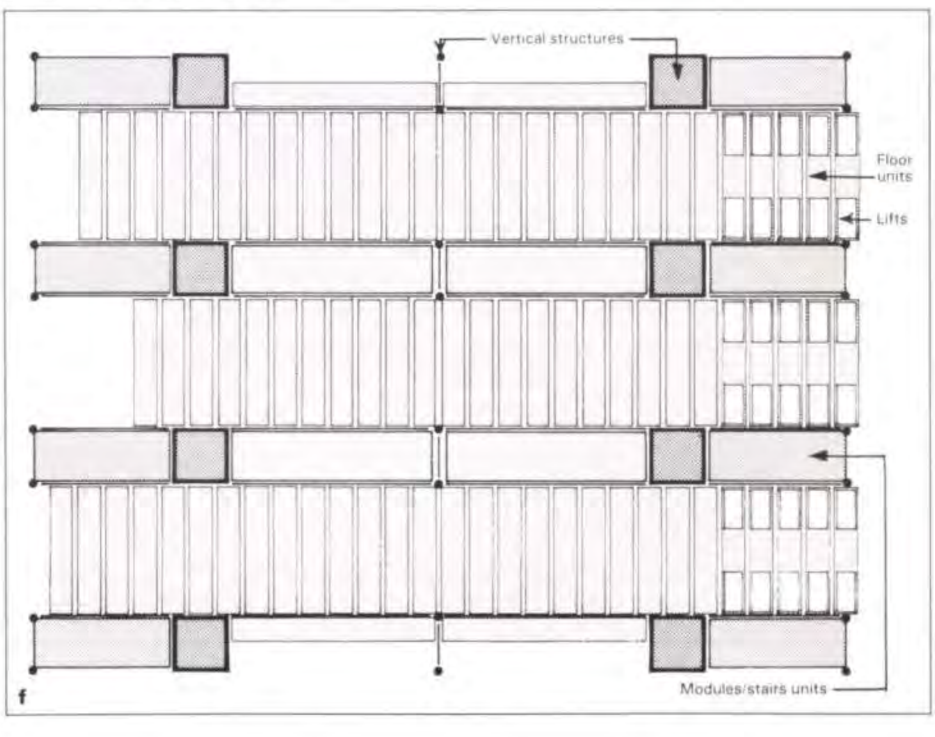
(3) *Expressed structural form*

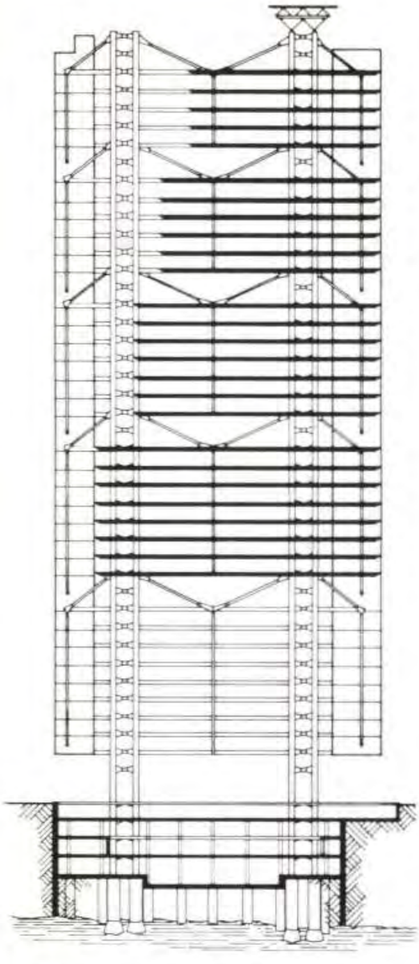
A further effect of the open space has been to lead the design development towards hanging or tensile forms which, by nature of their discrete suspension structures and

Continued page 6

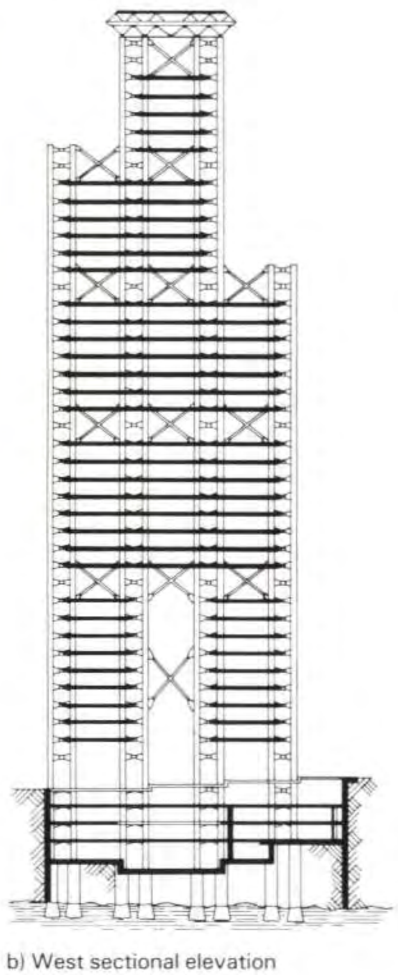
Figs. 3e-f
Final concept
e) Elevation f) Typical plan



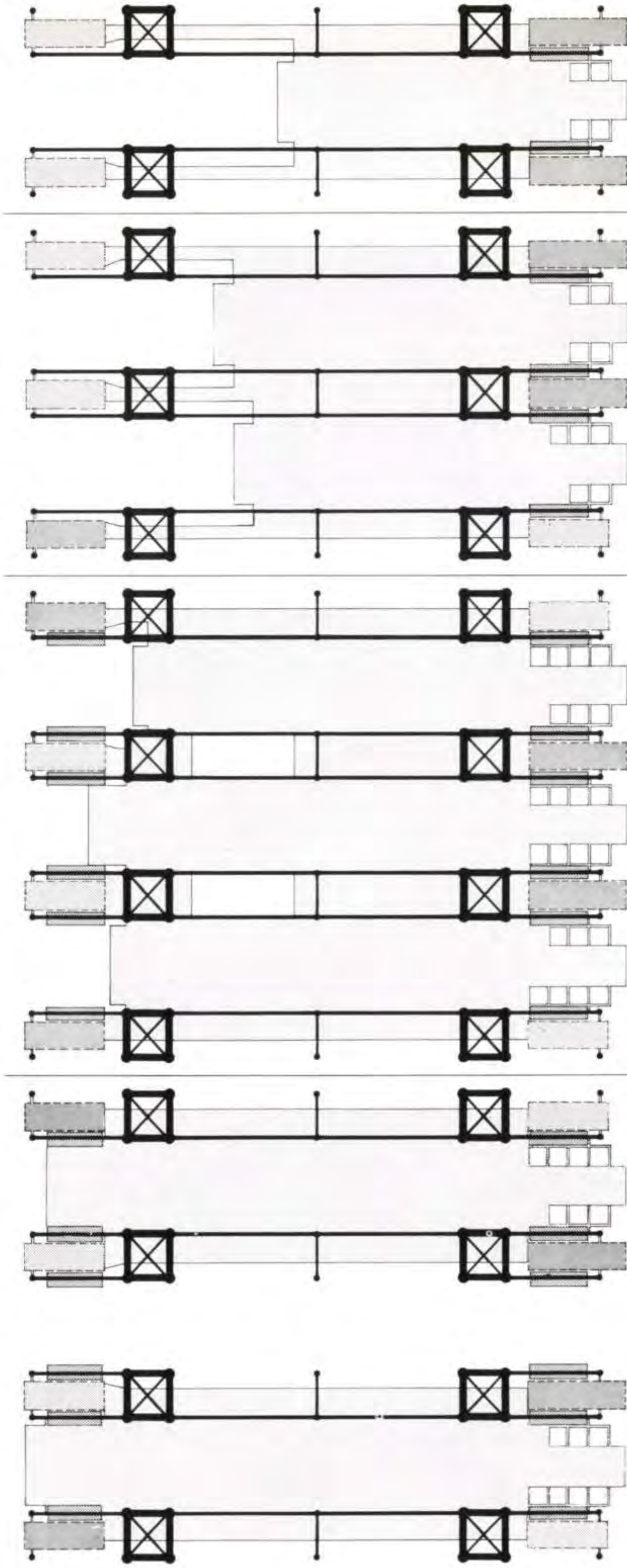




a) North sectional elevation



b) West sectional elevation



c) Typical one-bay office (Levels 37-41)

d) Typical two-bay offices (Levels 30-35)

e) Typical three-bay offices (Levels 13-28)

f) Typical banking hall level (Levels 3-11)

Key: Risers Modules Stairs Floors

Figs. 4a-f
Final design

support towers, project a strong image of the structural form. Initial schemes considered the possibility of expressing the unclad steel framework but the combination of fire protection requirements and specific visual considerations around a high quality, long-life finish ruled against their incorporation. High performance metal cladding solutions were identified as being a universal solution at an early stage of the design development.

(4) *Integration of structure, services and architecture*

The servicing concept of the building has departed from the traditional in a number of ways. The solutions adopted, although not unique or novel in concept, should be considered so by the very large scale and diversity of application.

The concepts which have had a significant bearing on the evolution of the structural framework are:

(a) Vertical movement system provided by a combination of lifts and escalators

(b) Decentralized modular services and plantrooms which consist of prefabricated, fitted out and commissioned units completely preassembled off-site

(c) Full access raised floor containing all servicing for light, air and communications.

(5) *Prefabricated building*

In order to achieve a high quality building with a fast construction programme, as many components as possible have been designed to enable prefabrication of finished products in off-site factory conditions.

This concept led the design development of the structural framework towards a totally steel construction.

(6) *Extendability*

Statutory regulations restrict the notional shadow projection of the building development to a specified proportion of the width of road projected upon. At the initial stages of the project it was hoped that a waiver could be gained and early design development schemes did not reflect the full implications of this restriction. Its introduction in later schemes produced the 'stepped' north-south cross section of the building and the 'setting-back' of the elevation along the narrow road boundary of Bank Street.

Despite this restriction, the building has been designed to provide the full area allowed by the plot ratio.

Although the statutory regulations limiting plot ratio and 'shadow' encroachment confine the building to its proposed floor area and external geometry, all the schemes developed included the provision of infilling the multi-storey voids within the building envelope and also the Bank Street 'set-backs' for later schemes. It is estimated that, if realised, these provisions would produce an additional 30% of floor area.

Final concept

The scheme comprises a rectangular plan approximately 55m x 72m overall, rising 175m above ground level over a 20m deep basement extending over the full site area.

There are four levels of basements and 43 floors in the superstructure topped by a multi-level helipad structure.

The structural scheme comprises:

(1) Reinforced concrete large diameter caisson foundations

(2) Multi-level reinforced concrete basement structure within a diaphragm wall enclosure

(3) Two parallel rows of four steel masts rising from the lowest basement level to the top of the building and dividing the building horizontally into three bays of 30, 37 and 43 storeys

(4) Five discrete levels of double-storey height steel stability structures comprising east-west suspension trusses and north-south cross-bracings at levels 11, 20, 28, 35 and 41 dividing the building vertically into five structurally independent sub-buildings

(5) Groups of steel-framed suspended floors which set back successively up the building on the east side in response to 'shadow' restrictions

(6) Modules and stairs

(7) Cladding and curtain walling.

Structural behaviour

Gravity loads

The gravity load support system is divided into five distinct zones vertically up the building by the suspension trusses. At the top of each zone the series of suspension structures span between and cantilever beyond the masts in the east-west direction, two per mast. Three vertical hangers are suspended from each truss which support primary floor elements spanning between the hangers and the masts. The masts transmit the loading direct to the foundations. The system is illustrated in Fig. 4.

The frequency of these structures is dictated by planning and not structural considerations.

In general the suspension trusses include a top boom between the masts. However, these are omitted for architectural reasons on the building façade frames and at the topmost zone of each mast. This significantly changes the force actions in these areas to a direct corbel action which because of the relative flexibility of the masts under this form of loading causes an increase in truss deflection.

Lateral loads

The static deflected shapes of the building under wind in the two principal directions are shown in Figs. 6 and 7. It can be seen that the behaviour in each direction is that of a five-storey portal frame, the masts acting as the columns and the east-west suspension trusses and north-south cross-bracing as the beams. The framed structure of the masts gives a predominance of 'shear' rather than 'bending' deformation.

The masts are restrained laterally at the highest basement level and the foundations.

Structural components

Considerable design development has been undertaken with the relevant sub-contractors in evolving the present design. This collaboration has enabled details to be developed, combining erection and fabrication advantages with improved performance in terms of overall buildability.

The total quantity of structural steelwork is

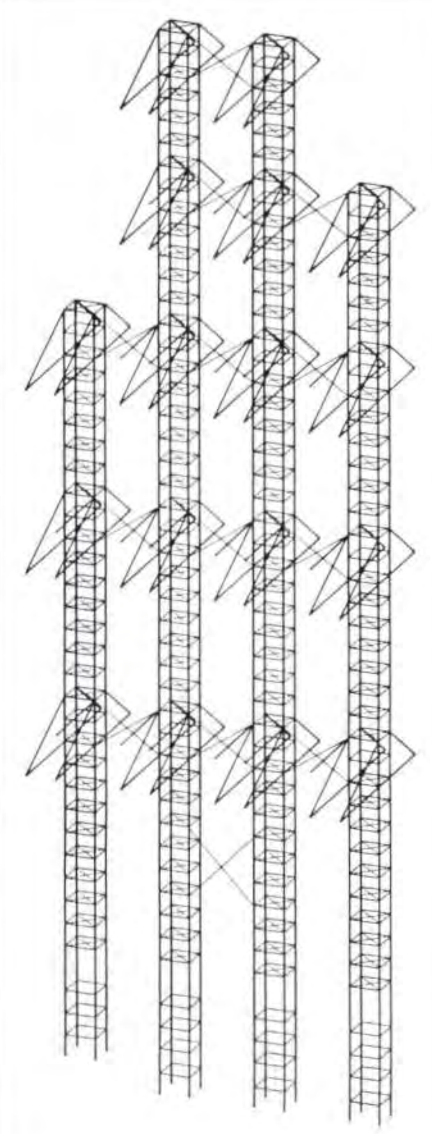


Fig. 5
Analysis model with floor plates omitted for clarity

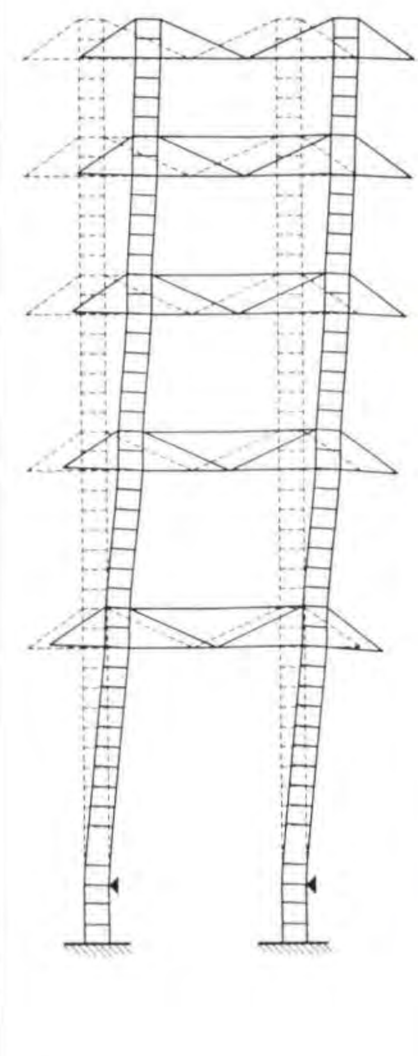


Fig. 6
East-west wind loading deflected profile

approximately 25,000 tonnes, most in grades of steel equivalent to *BS 4360:50D* and much of this with through thickness properties.

Masts

The masts are formed by four circular tubular elements interconnected by rectangular 'bowtie' beam elements at storey height intervals of 3.9m to form a vierendeel beam structure. The masts are 4.8m x 5.1m centre to centre in plan.

The elevational geometry of the vierendeel beams is constant throughout the height of the building; the plan geometry and diameter of the tubulars decrease up the building height to reflect the decrease in axial and lateral loads.

The tubulars at the base of the building are typically 1400mm in diameter with wall thicknesses of 90 and 100mm. At the top of the building they reduce to 800mm in diameter and 40mm thickness. All tubulars are formed from plate by hot or cold rolling techniques. Centrifugal cast and forged elements had been considered during the early development of the project, but these have not been progressed to the final design.

The thickness of vierendeel beam flanges is generally less than 50mm but towards the base of the building and within the suspension structure zones these increase to 100mm. The components forming the mast elements are shown in Fig. 8.

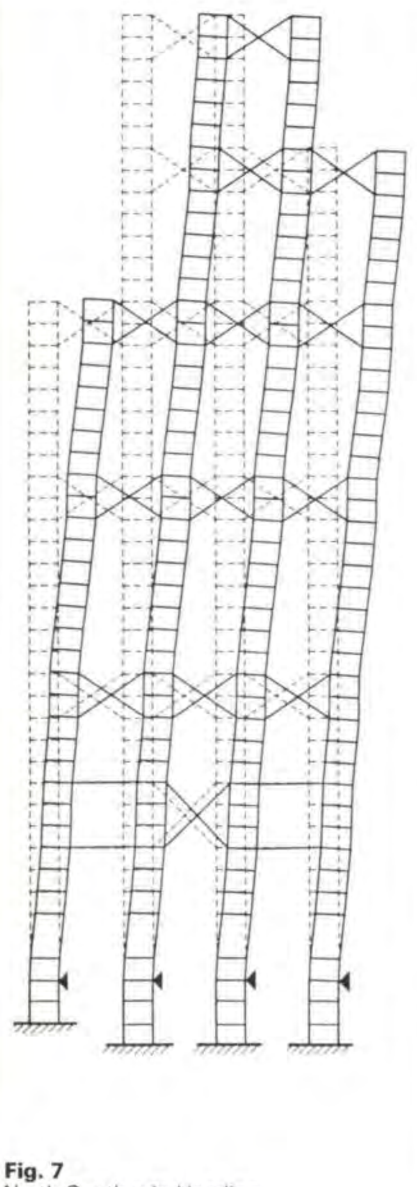


Fig. 7
North-South wind loading deflected profile

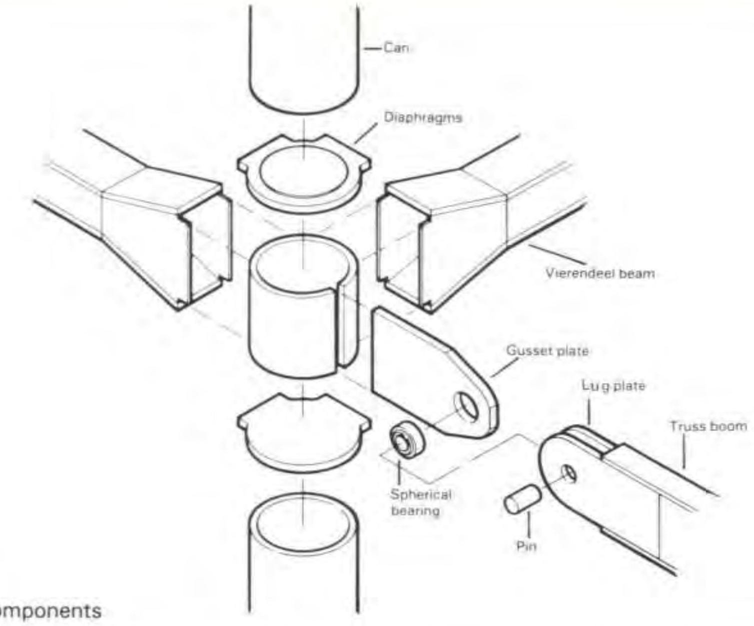


Fig. 8
Mast components

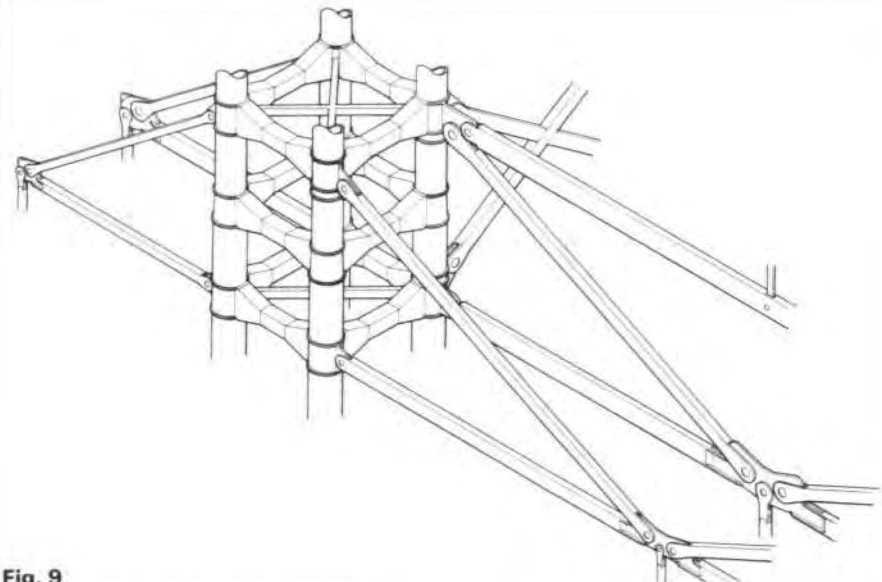


Fig. 9
Suspension structure components

Stability structures

Suspension trusses

The suspension trusses span 33.6m between, and cantilever 10.8m beyond the masts.

The truss structures comprise rectangular elements interconnected and connected to the mast by pins passing through end lug plates into large spherical bearings located within thick gusset plates (Fig. 9).

The horizontal truss elements also provide the primary floor beams at these levels. Generally the depth of inclined elements is 500mm and the horizontal beams 900mm.

The thicknesses of the lug and gusset plates are in excess of 100mm and reach a maximum of 175mm in the stability structures at the lower levels of the building. The bearings vary in size from 150mm to 600mm diameter; the pins are slightly smaller. The individual truss elements are formed from two thick plates spaced apart by thinner web or spacer plates, the thicker plates top and bottom in the horizontal elements for bending efficiency, and vertically in the inclined elements to ease connection detailing.

Schemes utilizing multiplated elements with friction grip bolt joints were researched and developed. These were not favoured by the contractors because of the large number of plates and bolts involved.

Cross-bracing

The cross-bracing linking the mast in the north-south direction at suspension structure levels is located on the inner line of mast tubulars only. Their construction and size are very similar to the inclined elements of the suspension trusses, incorporating large diameter spherical bearings within their connections to the masts.

Floor structure

The detailing of the floor has been dominated by the servicing concept of a raised, full access floor. The levels of the floor beams and concrete are governed by ceiling profiles, not by floor levels. The general arrangement of the floors at various levels of the building is shown in Fig. 4. The floor structure may be considered to be in three zones: 11.1m wide general use zone, 5.1m wide circulation and escalator zone and an edge zone.

The general floor structure comprises a system of 400mm deep beams at 2.4m spacing, acting compositely with a 100mm thick concrete slab on permanent profiled metal decking. These secondary beams span 11.1m between 900mm deep plate girder primary beams which span 16.8m between the mast and the central hanger. Within the circulation and edge zones is a similar arrangement of secondary beams and decking.

The primary beams in the east and west zones beyond the masts support the independent service modules and staircases.

Early designs for the floor construction were developed around the principle of assembling the floor structure, services and raised floor into completed 2.4m wide units off-site to speed erection. But upon close analysis these solutions showed only a marginal advantage over more limited prefabrication approaches and were not adopted.

Modules and stairs

These structurally independent elements are to be located between the primary beam structures at the east and west edges of the building for ease of erection access.

A total of 160 service modules are provided in the building. At the lower levels there are four modules servicing each floor, reducing to two at the upper levels where the building plan reduces. The maximum module size is 12m x 3.6m × 3.9m height and the installation weight is typically 30 tonnes with a maximum of 55 tonnes.

The vertical services are located in risers grouped in prefabricated frames two and three storeys high. A total of approximately 8 km of riser frames will be provided for the building.

The structural form of the modules has been developed with the sub-contractor. Monocoque stressed skin design solutions were investigated but were not developed due to the difficulty of providing structural fire protection requirements of one or two hours.

The module structure which has been adopted is a simple trussed box with lightweight steel deck floors.

There are 160 stair assemblies alternating in position with the modules. They are preassembled off-site into complete landing and stair flights 12m long and attach to the floor structure in a similar manner to the modules.

Cladding and curtain walling

The design of the various curtain wall and cladding systems is considered to be possibly more complex than any previous building. In particular the set-back building geometry and external steelwork produce a great variety of joint details and structural penetrations.

The connection details between the steelwork frame and floors and the cladding and curtain walling have been detailed to enable the latter to be erected with minimum tolerances. Movement joints are provided at each floor level to allow for structural deflections.

Three different curtain wall systems have been developed for different locations on the building elevation:

(1) Typical curtain wall for office floors comprising aluminium mullion and sill extrusions with 12mm toughened glass. At double-height spaces the typical mullions are stiffened by adding a vierendeel truss.

(2) Glass grid wall to the stairs, which comprises 12mm toughened glass fixed to an aluminium extrusion framework using structural silicone sealant.

(3) Panel wall for the service modules and risers comprising 25mm thick aluminuim honeycomb panel fixed to an aluminuim extrusion framework.

The cladding to the structural elements comprises 5mm thick aluminium sheet fixed to extrusions at each edge which form pressure equalized joints. The finish to all exposed aluminium surfaces of the cladding and curtain wall extrusions is backed fluoropolymer coating. The cladding profiles are fixed to the structural frame over the corrosion and fire protection coatings.

The cladding shape generally conforms with the shape of the structural element enclosed but does not reflect the connection details. The cladding surfaces are continuous and precision manufactured with a close tolerance on flatness.

Prototype tests

Main frame elements

As part of the overall quality assurance approach on this project a number of full-scale prototypes of key structural elements are being fabricated, instrumented and then loaded to destruction. The elements are:

(i) A column/vierendeel assembly
(ii) A connection (screwed coupling) in the vertical hangers
(iii) A typical truss connection with spherical bearing
(iv) A N-S cross-bracing connection with spherical bearing
(v) A number of secondary beam connections.

The tests are currently in progress using the British Steel Corporation's 1250 ton testing machine at the Britannia Works, Middlesbrough. In addition to confirming the structural design of these elements, the prototype programme is providing valuable information on potential problems in fabrication and erection of the production elements.

Module

A series of strength tests to destruction are being carried out on floor units and individual joints within the framework, as well as a deflection test on a complete module.

Cladding

Prior to production a full-scale prototype test of each type of cladding arrangement is carried out to 1.5 times the design wind pressure. Testing for water tightness and air leakage are also carried out.

Structural analysis

Analysis of the primary frame

It was considered that the only practicable way to obtain reliable design forces for the framework was to analyze a detailed finite element model with each member included explicitly. To simplify the model and to allow accurately for the effect of the non-prismatic beam sections and the relatively large joint zones within the masts, a detailed investigation was made into the bending and shear stiffness characteristics of the column/vierendeel assembly to enable these elements to be substructured. An isometric view of the primary elements of half of the model developed is shown in Fig. 5. Elements modelling the floor at each level are omitted for clarity.

This model enabled the overall force and deflection patterns of the building to be determined and detailed studies to be made of effects such as:

(1) The absence of top booms on the external and uppermost frames of the suspension trusses

(2) The twisting of the mast and high local loading caused at the east-west truss and north-south cross-bracing connection to the mast

(3) The interaction between the main frames and the floors

(4) The individual vierendeel beam and column forces.

Stability

Although the individual column elements between vierendeel beams are very stocky ($l/r > 16$), an investigation of the overall stability and force magnification of P-Δ effects of the frame was made in view of the relatively flexible vierendeel mast construction. The magnifications of around 10% were considered acceptable from a stability viewpoint.

Dynamic analysis

The overall dynamic properties of the structure were estimated for use in conjunction with the wind engineering studies described subsequently to establish the effects of dynamic response on:

(1) The peak lateral force due to wind

(2) The perception of movement and accelerations within the building

(3) Fatigue.

The effect of earthquakes was considered, but the seismicity of Hong Kong is low and the height of the building combined with the relatively severe wind climate dictate that wind is the more onerous lateral loading criterion.

The first mode (E-W direction) predicted by the dynamic analysis has a period of 4.5 seconds. The centre of mass of the proposed building becomes progressively more offset from the shear centre towards the top, because of the Bank Street 'set-backs' and coupled lateral torsional modes were found as expected, particularly in the N-S direction.

Wind engineering

Professor Davenport of the Boundary Layer Wind Tunnel Laboratory, University of Western Ontario, was commissioned in early 1981 to carry out an extensive study of the wind effects on and around the building. The study and associated wind tunnel investigations that were undertaken are considered to represent the most comprehensive study ever made of Hong Kong.

The work included a fundamental reassessment of the climatology of the area involving a new and critical analysis of all available records and the calibration of wind records from specific locations in Hong Kong by using small-scale wind tunnel models. The wind regime at the site was established for all wind directions by using the relevant slices of a 1:2500 topographical model of Hong Kong in the wind tunnel (Fig. 10b). When fully assembled the model was as large as a tennis court. The results from these tests were applied to a detailed instrumented model of the building in its surroundings for overall force, surface pressure and environmental studies (Fig. 10c).

The results of the study combined with the computer dynamic analyses have enabled us to confirm the adequacy of the framework for strength, fatigue, deflection and personal comfort. Further, the favourable results of the dynamic response analyses have enabled the stiffness and hence weight of certain elements to be reduced.

The study shows that the application of the Hong Kong statutory wind loads and cladding pressures is conservative, particularly for this location, principally because of the sheltering afforded by the topology around the general area of the site. It is hoped that the results of the climatology research will be used to influence the loadings selected for future work in this area.

Corrosion protection

We have made a number of studies of available corrosion protection systems for this building, particularly for the framework elements which are to be outside the building envelope and permanently enclosed within the aluminium cladding.

The traditional protection scheme for external elements is to encase them within a minimum thickness of 50mm of dense concrete. However, constructional considerations made such a solution inappropriate and consequently it was decided early in the project to research and develop a system with similar properties.

The system that has been developed and tested is based upon a polymer-modified gunite, reinforced with stainless steel fibres. The proving trials have demonstrated that a thickness of 12mm will give the required properties, particularly low permeability. Apart from site joints the coating will be applied off-site prior to erection, and in recognition the trials have included tests of impact resistance and differential heating due to welding. Fire tests have been conducted to demonstrate the stability of the material under fire load conditions. The selected sub-contractor is about to be appointed and final user trials undertaken.

The gunite coating will be applied to all external and main framework elements including masts and suspension structures while the other elements will be galvanized.

Fire engineering

We have been closely involved in advising on the fire engineering concepts and details for the building. The superstructure is divided into five principal compartments which are located between the truss suspension structures. Refuge terraces are provided at the lower levels of each truss which act as places of safety for the building occupants in the case of fire. Escape stairs are located in the outer zones of the building on the east and west elevations which provide access from the office floors to these refuge terraces.

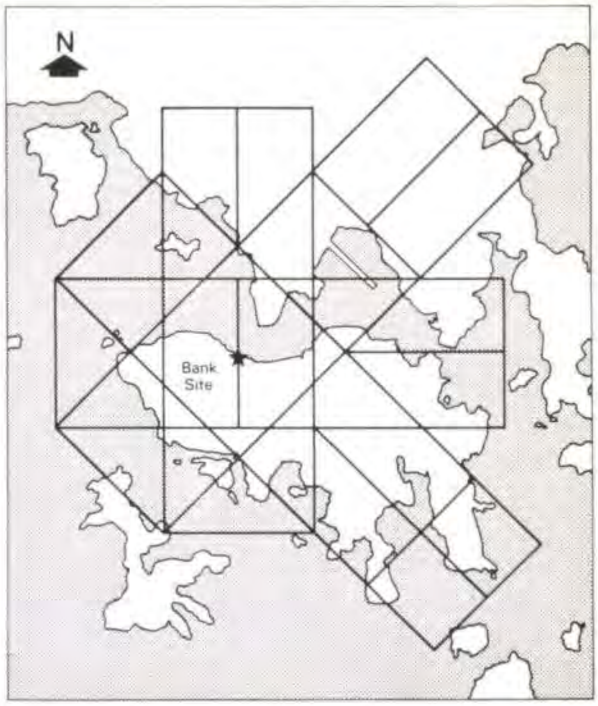
The compartment sizes vary between four to eight floors. All floor areas are sprinklered in accordance with regulations to reduce risk of fire spread within a compartment. Particular consideration has been given to smoke extract within the banking hall which has a full-height central atrium.

The main superstructure steel frame elements are provided with two hour fire protection. During earlier scheme developments the possibility of fire protecting the main steel elements by water cooling was investigated. However, since these elements will now be clad, conventional fire protection materials will generally be adopted.

It is proposed that the main frame steel elements will be fire protected using a ceramic fibre blanket. Fire tests have been conducted to determine the relationship between blanket thickness and steel section size. The floor steelwork beams will be fire protected using a board system.

It is proposed that the steelwork for the stairs and floor link bridges, where the steelwork is exposed, will be protected using intumescent coatings.

Figs. 10a-c
Wind tunnel study



a) Layout of topographical model for wind tunnel tests
b) Section of topographical model in wind tunnel
c) 1:500 scale proximity model including proposed building

b

c

Substructure

The basements to the redevelopment in many ways present an interesting contrast to the superstructure. The constraints on design are more dominantly external: physical factors rather than internal planning constraints. It is the architect's intention, however, to maintain the high quality finish, special environment and expression of structural elements throughout the substructure.

The site, which is at the base of the northern slopes of Victoria Peak, has a 2m incline across it. 100 years ago the shore line cut across this site but now after successive reclamations the shore is some 400m further north. The geology of the site is typical of a Hong Kong harbourside location, comprising 5m depth of fill and marine deposits, generally over the northern part of the site, overlying granite in various stages of decomposition, to rockhead approximately 25m below ground level. The stratigraphy is described in Fig. 11. Ground water level is within 2m of ground level on all sides of the site. A series of three site investigations has been undertaken to confirm the site geology, establish the soil properties and install a comprehensive array of surface and underground monitoring stations.

The basements, which are up to 20m deep, are on four levels and house the vaults, lorry loading facilities, and main plant hall, containing all major items of plant. The basement planning reflects the superstructure planning with vertical service distribution stairs and lifts, concentrated on the eastern and

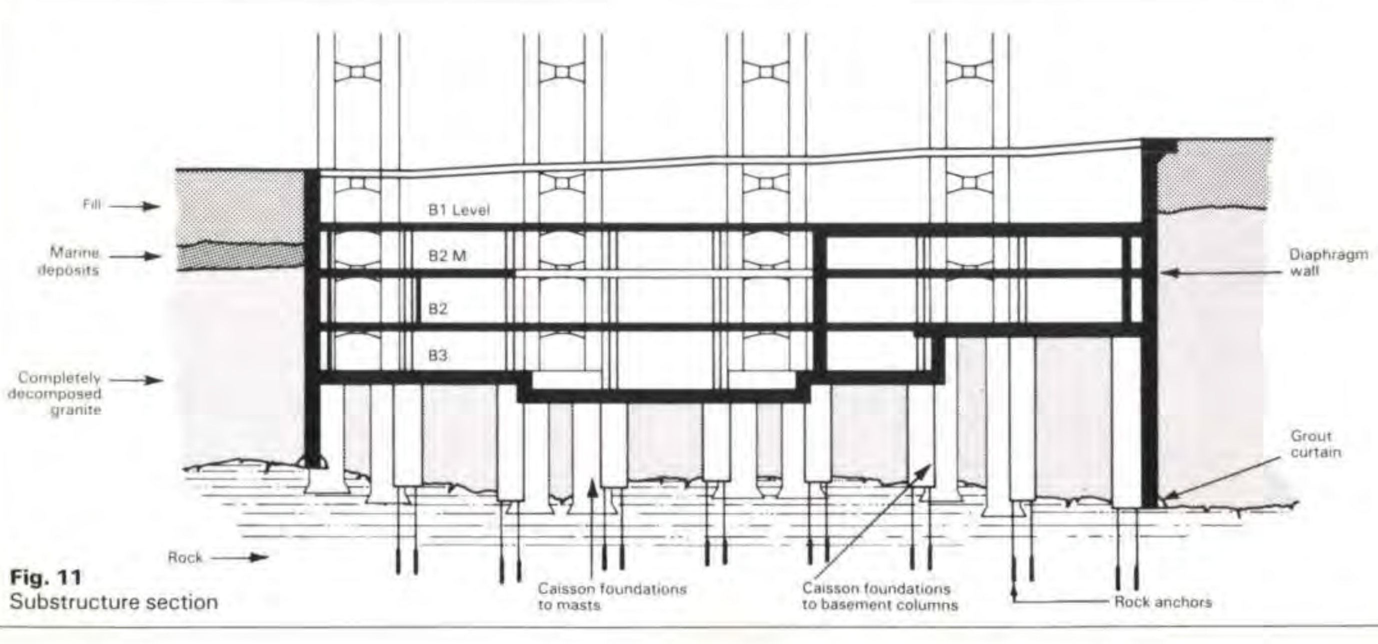


Fig. 11
Substructure section

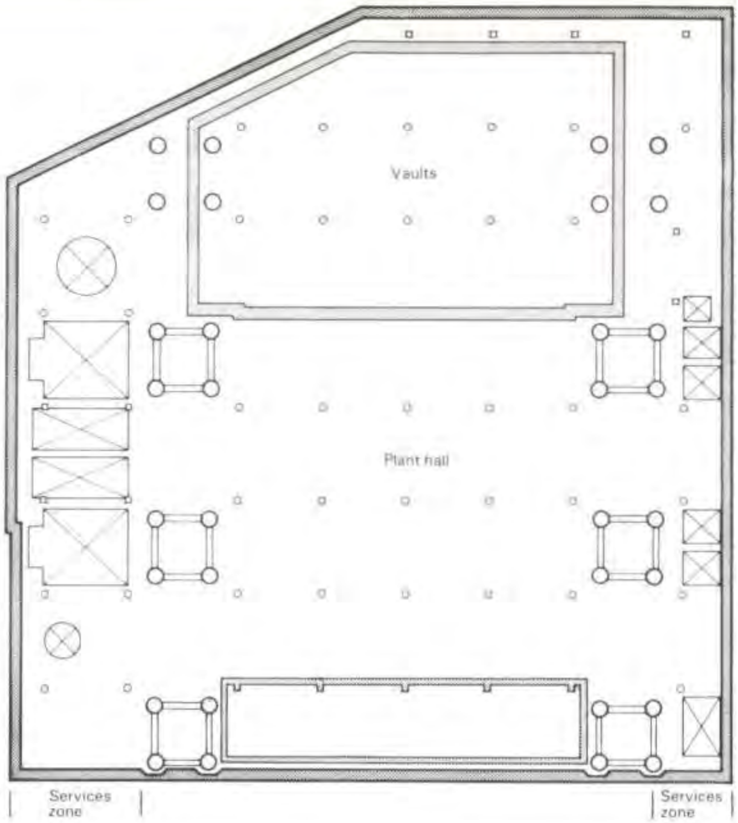


Fig. 12
Substructure plan

western perimeters of the building. This planning is shown in Fig. 12.

The basement structure comprises a perimeter diaphragm wall extending to rock, propped by solid reinforced concrete flat slabs spanning onto columns located on a 7.2m by 8.1m grid. The design floor loadings are generally $15kN/m^2$ with $25kN/m^2$ and $50kN/m^2$ in the safe depository and vault areas respectively.

The columns are cased stanchions supported on 2m diameter caissons founded onto the rock. The weight of the completed substructure construction is less than the potential hydrostatic uplift and consequently a permanent rock anchor system has been adopted and incorporated into the caisson construction to maintain stability. The foundations to the superstructure masts are groups of four 3m diameter caissons, one for each column of the mast.

Internal walls are essentially non-structural, very high quality fair-faced blockwork. However, security requirements in vault areas dictate that many should be reinforced concrete. All exposed surfaces are to be fair-faced and the concrete surfaces of the walls and slabs are to be lightly modelled to reflect the shuttering profiles used for them.

Geotechnical considerations

Hong Kong has an almost automatic association with geotechnical problems. The ground conditions are difficult and highly variable. Experience has shown that a particular behaviour can be expected from the completely decomposed granite which is the major sub-soil constituent in this part of Hong Kong Island. The need to control and monitor ground water and the lateral movement of excavations are of paramount importance.

At an early stage, after a brief assessment of retaining wall systems, a diaphragm wall enclosure was adopted principally for the following reasons:

(1) After grouting of the rock to wall interface, it forms a relatively watertight enclosure in which excavation can take place.

(2) It provides a strong, stiff construction which forms both the temporary and permanent retaining structure for a basement of this depth and allows a degree of flexibility in the location of temporary and permanent propping systems.

(3) It is a proven construction technique in Hong Kong.

Three stages of ground movement associated with the basement construction were identified from previous experience:

(1) Diaphragm wall installation.

(2) Dewatering.

(3) Basement excavation.

Estimated movements associated with the above were predicted using theories and computer techniques developed from our previous experience in Hong Kong. These estimates were verified by comparison with our site measurements of movements during construction of Chater Road Mass Transit Station 150m north of the site. Total vertical ground movements of the order of 50mm have been predicted.

Throughout the assessment period, different structural component forms and construction methods which give rise to the least ground movement, have been adopted. A key feature of this strategy is constructing the basements 'top-down', constructing the foundations and columns from ground level within large diameter caissons and then casting the permanent slabs as excavation proceeds, using them as the temporary works support to the diaphragm wall.

Sea tunnel

Hong Kong, with its limited space and high population, has long had a water supply problem. Indeed much of its potable water is imported from China. For many years it has been commonplace to use sea-water for flushing purposes in an effort to ameliorate the situation. In more recent years sea-water has been used as cooling water for heat exchangers with consequential energy savings and space saving, a significantly important commodity in Hong Kong.

The previous headquarters had a small sea-water pipe with an intake at Star Ferry Pier; it was perhaps inevitable that this redevelopment should have a system. The increased scale of the development has meant that the original system could not be reused. The final design is a 5.5m diameter rock tunnel 60m below ground, connected to an intake chamber at Star Ferry by an 11.0m diameter shaft and to the headquarters by a 4.5m shaft. It will provide water for the Bank and surrounding properties.

During the development period of the tunnel design many alternative vertical alignments were considered. However, the numerous underground services, the effective dam of Chater Station extending 35m below ground and a proposed twin tunnel railway immediately outside the site occupying the zone from ground to rock level, led towards a deep tunnel solution. A rock tunnel was adopted because of its obvious advantages of stable ground conditions, avoiding the need for working under high air pressures necessary if tunnelling in the decomposed granite above and minimizing the risk of ground settlements. Work has now commenced on the shaft at Star Ferry.

Construction progress

The construction period requires that erection of the superstructure frame must start during basement construction; indeed basement completion and superstructure completion are at similar projected dates.

Demolition to ground level of the past headquarters building was completed in November 1981. Diaphragm walling then commenced, the wall passing outside the existing basement structure on the northern boundary and inside the old retaining walls elsewhere. Soon after the start of walling, major obstructions were found along the line of the northern wall. The extraction of these obstructions delayed the completion of the walling and consequently the start of basement construction. To maintain the project completion date in 1985 the construction sequence has been modified. The original intention had been for the superstructure masts to be installed within a completed basement shell. They are now to be installed onto their foundations within 11m diameter access shafts formed at each mast location after completion of the diaphragm wall enclosure but prior to the commencement of basement excavation.

Walling work is now complete and excavation for caisson foundations and access shafts commenced. The monitoring programme is active and indicating that ground movements are generally slightly less than predicted.

Acknowledgments

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Credits

Client:
H. S. Property Management Company Ltd.

Architect:
Foster Associates

Quantity surveyor:
Levett & Bailey in association with Northcroft Neighbour & Nicholson

Services engineer:
J. Roger Preston & Partners

Management contractor:
John Lok/Wimpey Joint Venture

Structural steelwork contractor:
British Steel Corporation/Dorman Long Technical Services, Ltd. Joint Venture, UK

Cladding sub-contractor:
Cupples, U.S.A.

Module sub-contractor:
HMT Consort (HK) Ltd., Japan

Substructure sub-contractor:
Dragages et Travaux Publics, France

Appraisal of existing ferrous metal structures: Part 1

John Blanchard
Michael Bussell
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Edited by:
Poul Beckmann

Introduction

Growing disenchantment with much of the architectural use of modern construction techniques has led to increased support for the movement to preserve old buildings of architectural merit (in some cases very little merit!).

Many of these buildings occupy desirable city sites, but the use for which they were built may no longer exist or they have just become unattractive to potential tenants because of outdated services and fittings. The resulting low rents lead to decrease in maintenance which in turn leads to further deterioration.

In those circumstances refurbishment becomes not only a way of preserving the fabric of the building for posterity but can also sometimes be a financially attractive option.

Whatever the reasons, refurbishment of existing buildings has for some years constituted an increasing proportion of our workload and this trend is likely to continue.

Most of the buildings to be refurbished will have been built during the 19th century or earlier this century, the majority dating from between 1840 and 1940. This means that, at least in Britain, they will most likely have at least part of their loadcarrying structure built of cast iron, wrought iron or steel.

Any refurbishment scheme involves the question of whether the existing structure can be retained for the future intended use of the building, whether it needs strengthening or whether it is so far deteriorated or otherwise unsuitable (e.g. because of uneconomic storey heights) that all that can be done is to 'gut and stuff', i.e. demolish the entire innards and rebuild them whilst the façades are held up by temporary shoring.

The answer to this question comes largely from a structural appraisal. This is essentially a different exercise from design. Design is based on the fiction that the structure will be built according to our drawings and specifications and will behave according to our calculations; appraisal must attempt to deal with the facts of the structure as we find it.

The Institution of Structural Engineers' report, 'Appraisal of existing structures', provides guidance on the general aspects of structural appraisal, but does not contain very much detailed information on structures of the nature and age with which we are here concerned. It will therefore be assumed, in the following, that the initial stages of the appraisal are carried out, following the recommendations in the report, including a structural survey in the very early stages.

Identification of the metal

The first essential of an appraisal of an old metal structure is to identify the metal. The three basic varieties of ferrous metal which have been used in structures: cast iron, wrought iron, and steel, must be distinguished as their structural properties differ significantly. Cast iron is weak in tension and fails in a brittle manner. Wrought iron like steel has a tensile strength similar to its compressive strength but, lacking steel's homogeneous structure, it is prone to delamination.

The different characteristics of the metals and the techniques used to form them into structural sections, and to join these, often make identification possible 'by inspection'.

Knowledge of the age of the structure is also useful, as the three metals have in turn flourished as structural materials in the last two centuries. It is therefore sensible to examine the structure, and to seek documentary or other clues to its age. Sampling in the first instance is needed only to confirm the nature of the material. More extensive and costly sampling and testing is best left until an initial appraisal has shown whether a scheme for re-use or alteration is feasible in principle.

What to look for

Cast iron, as its name implies, was formed into sections in a molten state. This required the use of moulds, usually of sand or loam, which in all but the most ornamental quality castings resulted in a pitted or 'gritty' texture on the face of the iron. (This will by now often be masked by painting, so a wire brush should be used to expose the bare metal for inspection.) The method of forming was exploited to produce a wide variety of profiles. Columns were generally of circular profile (hollow more often than not and as such sometimes used as down pipes or even for steam heating), cruciform, or I-section. Baseplates and heads were formed integral with the shaft, the heads in particular giving scope for classical expression (leafy Corinthian or severe Doric), corbelled stiffeners, or hollow box-section: all were functional as the enlarged head provided bearing for beams. The latter were from the outset shaped and proportioned in a way that recognized the weakness in tension of cast iron, with more iron placed in the bottom flange than in the top. Early beam shapes were inverted 'Ts' or triangles, these giving the additional benefit of a convenient springing for the brick arching, widely used in mills and warehouses (before concrete became the obvious choice) for a 'fireproof' floor. Later I-beams retained the larger tension flange, often adopting the structurally-efficient 'fish-belly' profile in plan or elevation.

The problem of differential cooling of the iron, leading to locked-in tensile stresses, was widely recognized. One way to limit its effects was found by experience to be the use of generously-rounded roots at re-entrant corners. In contrast, the external corners of cast iron sections are frequently sharper than in wrought iron or steel.

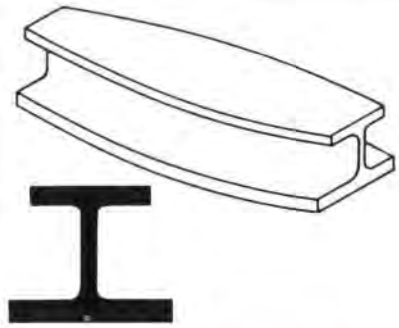
Connections between cast iron sections were by simple bearings or by wrought iron threaded rods and nuts fixed through preformed holes. Hollow circular columns were often cast in two semi-circular pieces which were then brazed together.

Wrought iron resembles steel in being formed into structural sections by passing billets through rollers. The earliest beams were built up from plate and angles riveted together, as these sections were all that the available rolling machinery could handle. Subsequently rolled beams became available, often being strengthened by riveted flange plates and web stiffeners. Its tensile superiority over cast iron led to the early use of wrought iron as chains, cables, and links for suspension bridges, and as tie-rods in buildings. The rods were frequently employed compositely with cast iron to form trussed beams and roof trusses.

Wrought iron can be distinguished visually from cast iron by its smoother rolled surface — assuming that not much corrosion has occurred. If more corroded, wrought iron tends to delaminate into thin sheets of nearly pure iron alternatively with slag which can be pulled away from the surface. Other distinctive features are the presence of rivets, and sections such as rods and cables which were never made in cast iron for structural use.

Distinguishing sound wrought iron from steel is more difficult as their production and structural forms are so similar. The maker's details (if stamped on the metal) may identify the material, likewise section descriptions. Wrought iron sections were never standardized, being decided by a particular ironworks. Steel sections were standardized from 1904 although many steel companies continued to offer their own 'specials' as well, prior to nationalization. 'Modern' fixings such as friction-grip bolts and welding are exclusive to steel (except in very rare cases of recent repair of wrought iron, which should be well-documented or obvious).

The illustrations summarize aids to the visual identification of ferrous metal elements.



Cast iron

Pitted or 'gritty' surface texture
Thick or coarse sections
Internal corners rounded:
external corners 'sharp'
Tension flange often larger
than compression flange
Flanges often 'fish-bellied'
on plan or elevation

Wrought iron

Smoother surface than cast iron unless
corroded, when delamination occurs
Joists rolled in modest sizes only:
larger sections built up from joists,
plates, and angles riveted together

Steel

Visually similar to uncorroded
wrought iron but larger sections rolled
Maker's name or section reference
often stamped on web
Standardized section sizes

Fig. 1
Characteristics of the various
ferrous metals as used in beams

The age of the structure

The identification of the metal can be aided or confirmed if the date of construction can be established, as the chronology of using iron and steel in structures is fairly well-defined.

Many older buildings or structures have the year of completion somewhere on the façade. Foundation stones and plaques are other unambiguous clues as to date. Documentary sources may often prove helpful. There may not, in the early stage of a job, be time to do more than ask the building owner and local authority for drawings and other records but this is always worth doing. The building or structure itself can often be dated to the nearest quarter-century by an experienced eye examining the architectural features and this should always be done, in case the building described in the records is an earlier or later construction!

Assuming that some idea of date has been obtained, Table 1 may be used at least to eliminate, or with luck to identify, the possible materials. It specifically covers British developments but will also be helpful as a guide for territories of the former Empire. The chronology is different in continental Europe, the USA, etc. Incidentally, it should not be assumed that metals were always produced in the country of use, even when these were major industrial powers. Belgian steel beams have been identified in at least one Edwardian building in central London – supported on English cast iron columns.

Care is obviously needed when (as is common) a structure has been altered in its lifetime or where structural elements have been salvaged and re-used. It should be possible to establish this by inspection, or documentary research.

How to make sure

Unless the evidence of visual examination or dating is conclusive, some sampling should be undertaken at the earliest possible opportunity. At this stage, the objective is merely to confirm or establish the nature of the material.

The simplest approach is to obtain a small specimen of the metal and send it to a specialist testing house where it can be chemically, or for cast iron, metallurgically examined and identified. Drilling swarf is adequate for chemical analysis whereas metallurgical examination will require a piece approximately 25 by 25 mm, such as can usually be obtained by sawing or by core drilling (flame cutting of cast iron is likely to produce dangerous notches and cooling stresses, and for steel and wrought iron it will require substantial increase of the sample size to allow removal of the heat affected zone; it is not recommended for identification sampling).

The testing laboratory should be instructed to identify the particular variety of metal, not just its generic name. This is because ferrous metals can have significantly different mechanical properties for the same chemical composition.

Table 1:
Key dates in the chronology of structural iron and steel in Britain

	Cast iron	*Wrought iron*	*Steel*
1792	First beams		
1794	First columns		
c.1800	First I-beams		
1840		First built-up beams (plates and angles)	
1860	Decline in use for beams	First rolled joist	
1877			Board of Trade approved use in bridge-building
1882-9			Forth rail bridge
1885			First rolled joist (Dorman Long)
1909	London Building Act gives design rules and permissible stresses		
1914	Virtually extinct in new work	Virtually extinct in new work	
1937			*BS 449* published
1959			Revised *BS 449*

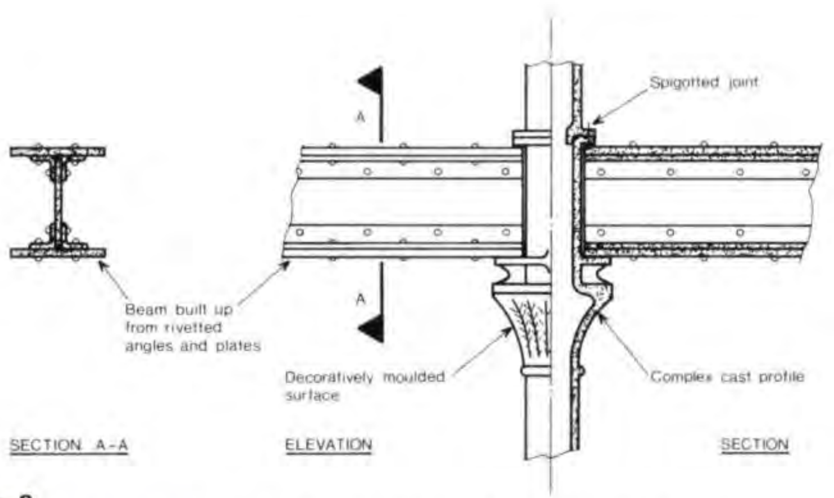


Fig. 2
Typical iron-framed construction: cast iron columns and wrought iron beams. Characteristic features are noted.

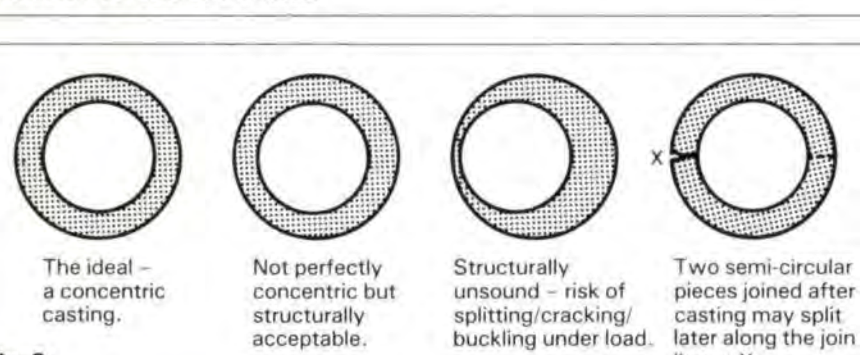


Fig. 3
Hollow circular cast iron columns are rarely found to be concentric and are often made up from two halves: problems can ensue.

Preliminary assessment: Comparison with contemporary design practice

Having identified the metal or metals, the next step is to get a rough idea of the loadcarrying capacity of the structure.

It must be appreciated that material specifications and the regulations governing design and loading have not always been as detailed, rigorous and 'numerical' as they are now. They did however recognize the characteristics of the materials, e.g. the brittleness of cast iron and the laminar nature of wrought iron, and made allowances for these.

One cannot therefore appraise an existing structure of cast or wrought iron by calculations based on today's steelwork code of practice, even if suitably factored to recognize a different basic 'working stress'. Nevertheless, some calculations will be needed, to establish the feasibility of re-use or alteration. For these to be relevant, the engineer requires an understanding of the original material specification, design regulations, and loading assumptions, and an awareness of how these differ from current practice.

Materials specifications

Cast and wrought iron were never subject to a standard quality specification as structural steel is today. Available information on strength would be furnished by the ironworks, each of which offered a variety of grades suited more or less to the needs of the market. Extensive independent testing was also carried out with the twin aims of improving the understanding of the materials' properties, and informing users. The results of the tests were published in several widely-used textbooks, being referred to by architects, engineers, and builders for guidance on strength, loadings, and design aspects generally. One of the best, Twelvetrees[1], quoted the values in Table 2 as typical in 1900.

It is important to recognize that these are *typical* figures, and that quoted strengths are *ultimate*. It is also noteworthy that cast iron has a very low elastic limit – some test results indeed suggested that permanent set occurred under any stressing. One other word of warning should be sounded on the tensile strength of structural cast iron generally (not just the malleable form): Twelvetrees noted elsewhere that an *average* value was 8 tons/in.^{2}, *but* quoted the results of 51 tests by Eaton Hodgkinson (a leading investigator) thus:

	Tons/in.^{2}	(N/mm^2)
Highest	10.5	(160)
Lowest	4.9	(75)
Average	6.8	(105)

It is evident that cast iron in tension always was frail and very variable material!

Design regulations

No statutory guidance on the use of metal in structures appeared before the London Building Act of 1909 (the so-called Steel Frame Act, which for the first time recognized that framed metal structures did not demand wall thicknesses as substantial as when the walls were loadbearing).

Prior to 1909, design of metal structures in the UK was based on experience and the use of factors of safety applied to the *ultimate* material strengths. These were recognized as factors encompassing both the ignorance of and the variability in the strength of materials and expected loadings, but they were nevertheless widely used. Twelvetrees[1] recommended the following factors:

	Dead load	*Live load*
Cast iron		
Beams	5-6	8-9
Columns	5-7	8-10
Wrought iron and steel		
Beams	3-4	5-6
Columns	4-5	6-7

The 1909 London Act (which was certainly known to those building outside London also) gave detailed rules on design, including loads, minimum metal thickness, and permissible ('working') stresses. These last are tabulated at the end of Part Two of this paper.

Tables were also given for permissible stresses on columns, taking account of slenderness and end fixity.

The figures were repeated in the 1930 London Building Act by which time (as virtually by 1909), the use in new work of cast and wrought iron had ceased. In 1937 *BS 449* appeared, being the first 'national' standard for structural steel: it included definitions of superimposed and wind load (preceding BS *CP3 = Chapter V = 1952*), design and detailing guidance, and a basic permissible tensile stress for mild steel of 8 tons/in.^{2} (124 N/mm^2).

This, and the corresponding values for compression, shear and bearing, was applied to mild steel complying with *BS 15: 1936* (typical tensile strength 28-33 Tons/in.^{2}, 20% elongation). High tensile steel (*BS 548: 1934*, typical tensile strength 37-43 tons/in.^{2}, 18% elongation) was rated at 12 tons/in.^{2} in tension or compression, i.e. 50% 'better' than mild steel. (The 1937 mild steel stresses are tabulated at the end of Part Two of this paper.

The first standard for girder bridges was published by the British Engineering Standards Assocation as no. 153 (*sic*!) in 1922-3. It had the same basic stresses for mild steel as *BS449: 1937* but naturally gave more attention to the various loading conditions on bridges, and surprisingly gave permissible stresses on cast steel, wrought iron, and cast iron (to be used in compression only for railway bridges, but allowed in bending for other bridges).

It is reasonable to assume that these standards, and their successors, would have been generally used in the design of steelwork (other than builders' lintels and other minor elements designed 'by eye') in the United Kingdom since their respective years of issue.

A note on the original 'modelling' of steel structures for design purposes may be of use; it was customary until 1937 at least (and indeed, it is sometimes still the case today!) that building frames were designed for dead and superimposed gravity loads, assuming simply-supported conditions at columns, and then analyzed for wind loading by assuming rigid joints, with points of contraflexure at column mid-heights and beam mid-spans. Joint detailing and actual construction were consistent with this 'semi-rigid' approach, but unless original drawings can be found or the joints exposed – not always an easy matter – it may be difficult to convince the building control authority that this approach provides any usable 'reserve of strength' to accommodate a proposed increase in dead or live loading.

Table 2:

Materials properties after Twelvetrees

	Tons/in.^{2} (as quoted)	(N/mm^2) (approx. equivalent)
Cast iron (average values)		
Tensile strength	8	(120)
Compressive strength	38-50	(590-780)
Transverse (flexural) strength	15	(230)
Shearing strength	6-13	(90-200)
Elastic limit	1	(15)
Young's modulus:		
Compression	5467–5879	(84500–9050)
Tension	4262–6067	(66000–93500)
Wrought iron		
Tensile strength	18-24	(280–370)
Compressive strength	16–20	(245–310)
Shearing strength	75% of tensile strength	
Elastic limit	10–13	(155–200)
Young's modulus	10000–14280	(155000–221000)
Mild steel		
Tensile strength	26–32	(400–495)
Shearing strength	70–75% of tensile strength	
Elastic limit	18–20	(280–310)
Young's modulus	13000	(201000)

Loading regulations

As with material stresses, no statutory guidance was published before the 1909 London Building Act, and once again the textbooks of the day are the nearest approach to a standard in the 19th century. The figures given varied and were invariably higher than to-day's standards require. Typically[1], 70-140 lbs/ft^2. (3.4-6.7 kN/m^2) live load was advised for a domestic floor, 168 (8.0) for public buildings, and 200-336 (9.6-16.1) for warehouses.

Past experience in appraisal of buildings of this vintage makes it clear, however, that many builders did not adopt such onerous figures for the majority of domestic and commercial timber floors (which were sized by experience and/or rule of thumb), and this is probably true also of many building structures with metal beams and columns.

The fact that many structures, as built, were clearly always incapable of supporting the over-generous design loads quoted in the textbooks and yet today exhibit no signs of overloading, has led building control authorities to be increasingly reluctant to accept 'on the nod' re-use schemes for which current live loading requirements would appear to be less than the original 'assumed' loading. It will therefore usually be necessary to establish member sizes and show by calculation that the existing structure is adequate.

The 1909 Act, and the later 1930 Act, required the following superimposed loads to be allowed for:

	lbs/ft.^{2}	kN/m^2
Domestic	70	3.4
Offices	100	4.8
Shop or workshop	112	5.4
Warehouse (minimum)	224	10.7
Roofs (less than 20° slope)	56	2.7
Other roofs (wind included)	28	1.3

Wind loading was to be taken at 30lb./ft.^{2} (1.4 kN/m^2) on the upper two-thirds of the building face.

A reduction of 5% of superimposed load could be made on supporting structures below the uppermost storey (except in two-storey buildings), increasing by 5% on each further storey to a maximum of 50%. No such reduction was allowed for warehouses.

The superimposed and wind loading requirements were eased slightly in *BS 449: 1937*, which also introduced the notion of designing slabs for a somewhat heavier superimposed load than applied on the supporting structure, but required allowance for partitions (where not shown on the plans) of 20lb/ft.^{2} (1 kN/m^2) on all structural elements. More recently, superimposed and wind loadings were defined in *BS CP3: Chapter V: 1952* and subsequent revisions).

Testing of material properties

After the preliminary assessment it is often felt desirable to obtain more accurate information on the mechanical properties of the metal(s).

Testing is an expensive and slow business, so it is essential first to establish that there is a real need for it, and then to decide how samples are to be obtained and what tests are to be made. Not least important is to be aware of what the test results will *mean* in relation to the existing structure – *before* starting *any* testing.

Is your test really necessary?

As noted earlier, there is little point in making tests if an initial inspection and appraisal has shown that the structure is in an unsound state already, or that it is likely to be grossly overstressed in its new use. Conversely, a 'young', well-documented steel structure – and often older structures too – may need little or no testing if they are in sound condition and will be stressed only to modest levels in the future.

It is of course important that the scheme is discussed with the relevant building control authority at an early stage, and that their requirements for testing are identified (together with our own). It may come as a surprise that some authorities are very dubious about the usefulness of testing as an indication of typical strengths in the actual structure, but this may be understood by considering the wide scatter of results, particularly for cast and wrought iron, previously quoted. Quality control in the 19th century was very much cruder than it is today, as was recognized by the generous factors of safety applied. There is thus no guarantee that sampling for testing, or even in situ load testing to failure (e.g. of elements typical of the building but unwanted in the proposed scheme), will give results that can be confidently regarded as 'average', still less as 'lower-bound', for the elements as a whole.

In such cases it may be more sensible and relevant to appraise the structure using (albeit conservative) permissible stresses such as those from the London Building Act given above.

Sampling: principles and expediencies

Because the properties of ferrous metals are not entirely constant, it is not possible to take a single known figure and apply it to an unknown structure. Similarly, it is not possible to take a single sample and test it, since there will certainly be variability within the structure. This variability will take two forms, statistical variability and bias.

It is generally assumed that the strength of a group of similar ferrous metal elements will vary in accordance with a normal distribution, and whereas this must be an approximation (allowing, for instance, a finite probability of having a member with negative strength), it is quite adequate for most circumstances. Once this assumption is made, it is then possible to use statistics to give an estimated strength.

Usually this is a 95% confidence limit based on test results, i.e. a figure below which no more than 5% of the actual strengths should fall. To find this, it is necessary to calculate the mean value and the standard deviation of the test results. The 95% confidence limit will then be a number of standard deviations below the mean, that number being a function of the number of samples taken.

Hence, if only two samples have been taken it will be necessary to use a value 6.3 standard deviations below the mean, whereas if six samples are taken this is reduced to two standard deviations. For an infinite number of samples, the figure is 1.65 SD, so that unless one wants to scrape the barrel or the material is likely to be very variable, there is not much to be gained by taking more than six samples. It should be noted here that, while this discussion has been concerned with strength, the same approach can be used for other material parameters such as elongation, charpy toughness, etc.

Life, however, is rarely simple, and whereas the statistical variations can be dealt with fairly easily by taking a sufficient number of samples, it is often far more difficult to deal with bias. Bias arises in a number of forms.

For instance, if two steelworks have provided steel for different parts of the structure, a statistical analysis based on one cannot be applied to the other, even if the members are nominally identical. If the situation is known to exist, or suspected, then separate sampling of each area should be carried out. Otherwise, it may be impossible to detect.

There are, however, various limits that can be given to help overcome some of the most likely sources of bias.

(1) Cast iron is a material whose properties vary considerably with position in the casting and with cross-section of the casting, since both of these affect the cooling rate of the molten material and hence the grain structure. Hence, for instance if measurements are made at the thickest cross-section the results will almost invariably be worse than those for other areas. Another consideration is that at areas of greater cross-section within a casting there is a far greater risk of porosity and other casting defects, due to the practicalities of foundry technology. Hence, if the natural tendency to take samples from the thickest area of a casting, (presumably to be best able to lose some of its cross-section), is succumbed to, the results may be unfortunately, and invalidly, low.

(2) It should not be assumed that strengths within a small member are the same as those in a larger member of the same material. For cast iron this has been dealt with above, for other materials the difference is likely to be less, but not negligible. This is because rolled materials have more work put into them in forming a thinner section, and this shows up as a higher strength.

(3) Beams and columns are not necessarily of the same material or source of supply. This should be obvious but is sometimes ignored.

This is clearly not an exclusive list of possible sources of bias and every effort should be made to discover and eliminate any such sources before carrying out sampling and statistical analysis.

Sampling techniques and choice of tests

For most normal purposes, unless it is proposed to weld to the existing structure, the most useful information is that which can be gained from a standard tensile test as covered by *BS18*, i.e. yield stress, ultimate strength, Young's modulus and elongation to fracture. This obviously involves destructive laboratory testing of samples cut from the structure. Ideally, samples 200 × 100mm should be cut, which will then be machined to the required shape by the testing laboratory. However, this has often proved difficult, and in some special cases, (and at extra cost!) it has been possible at one laboratory to get by with samples 100 × 50mm.

Removal of tensile samples is a cause of much headscratching. Techniques vary from hacksaws via interlocked drilled holes to flamecutting (which should only be used with an additional 1-15mm allowance around each cut face to enable removal of the heat-affected zone), etc. In taking the sample, the prime requirement must be not to weaken the structure dangerously, whether by reduction of cross-section, by introduction of stress-raisers, such as sharp corners, or by embrittlement due to the heat of cutting.

In addition to laboratory tests, it is often useful to carry out simple in situ non-destructive tests such as hardness surveys. The hardness is measured using, for instance, a portable Brinell hardness tester, which should be available from most independent test houses. (This measures the diameter of the imprint made, when a hardened steel ball is pressed against a smooth surface with a known force.) The figure obtained can be correlated very roughly with the ultimate tensile strength, and as such is useful as a measure of variability, if not much else. A hardness survey, on its own, is unlikely to give meaningful results.

Where it is intended to weld to existing steelwork, it will be necessary to carry out a chemical analysis to gain insight into the weldability. Again, this might be variable between members. *BS5135* implies that for unknown steels a value of carbon equivalent (measure of weldabilty) can be estimated from the carbon and manganese contents alone, adding a constant value for other residual elements. For old steels it is recommended that phosphorus and sulphur contents should be measured as well, as these can lead to welding problems if they are too high.

The chemical analysis required here can in general be done on a relatively small piece, say 25mm square, and the possibility of using a broken tensile specimen should be considered.

It should be noted that one should not normally contemplate welding wrought iron or cast iron, so the above applies to steel only.

When removal of sufficient tensile test specimens of cast iron is impractical it may be possible to carry out wedge penetration tests on discs 35-50mm in diameter. This test has been developed by, and is so far only used by, the British Cast Iron Research Association. Our experience with the method is limited to one project, where the samples could only be cut from enlarged nodes in a grid and therefore contained many cooling cracks and porosities which gave rise to very variable and low test results.

Interpretation

The approach to interpretation will be influenced by the age of the structure and the number of test results available. It is useful to plot the results as a histogram and first examine this by eye before attempting any statistical sophistry. This applies whether the tests carried out are purely of strength, or non-destructive 'strength-related'.

Cast iron and wrought iron are likely to show a wide variation in strength, reflecting the difficulties of maintaining consistent properties with the production process used. A small number of tests is therefore unlikely to provide a more reliable estimate of strength – and hence more advantageous permissible stresses – than contemporary regulations, e.g. the London 1909 Building Act.

If a sufficiently large number of test results are obtained, the 95% confidence limit can be calculated as previously described. This should then be divided by a suitable factor of safety (for cast iron a factor of 3 is suggested in view of its brittle nature; similarly, in view of the greater variability of wrought iron, compared with steel, a factor of 3 on the 95% confidence limit of tensile strength seems appropriate).

If the permissable working stress resulting from this calculation is higher than that quoted in the London 1909 Building Act, the agreement to its use should be obtained from the local building control authority. If it is lower, it should be used in its own right, and stresses for other conditions (e.g. shear or bearing) be obtained from the 1909 London values by proportioning.

Recent or *better-quality older steel* should exhibit a narrower variation in strength: the tests should also confirm whether the steel is of mild or high-tensile quality. The permissible stress adopted for appraisal should, it is suggested, be less than the minimum value of the elastic limit, and be not more than either 0.67 × 95% confidence limit of the yield stress or 0.375 the ultimate strength (again with the agreement of the building control authority).

This article, which will be concluded in the next issue of The Arup Journal, forms part of The Arup Partnerships' Seminar on steelwork, held in November 1982.

Theatre Royal, Plymouth

Martin Gates-Sumner
Mike Storey

Architects: Peter Moro Partnership

Fig. 1
The theatre on its corner site

Introduction

Plymouth's new civic theatre opened as scheduled on 5 May 1982 with a royal variety performance following a five-year design and construction period. The theatre represents the culmination of the redevelopment of the city centre following its devastation by bombing in the Second World War, and will enhance the City Council's promotion of Plymouth as the regional centre for the West Country and as a major tourist attraction in its own right.

History

The site on which the Theatre Royal now stands was originally allocated for a theatre in the master redevelopment plan for the city centre. Since that time it had been used as a municipal car park.

With the proposed demolition of Plymouth's only other live entertainment centre, the Palace Music Hall, there was much debate in the early '70s, on the size, form and policies for the new theatre. In 1974, a theatre was included in the brief of a limited competition for the commercial development of a large area adjacent to the Civic Centre including the present site. Peter Moro and Partners collaborated over the theatre element with the team of architects and developers which produced the winning scheme. The deteriorating economic situation led to this development being abandoned, but surprisingly the City Council decided to proceed with the theatre alone. The brief for the competition still reflected the indecision in the Council over the type of theatre to be built, offering the option of either a 750-seat repertory theatre, or a 1500-seat theatre for large scale and popular entertainment.

The theatre consultants, Carr and Angier, were then instructed to prepare a new and detailed brief embracing the need for the theatre to cater for both large- and small-scale productions, but within one auditorium. Peter Moro and Partners were commissioned in March 1977 to produce a design to this brief, and Ove Arup and Partners were appointed later that year as structural and services engineers, undertaking a multi-disciplinary design of a theatre for the first time. In addition, we recommended and were commissioned to undertake the co-ordination of the stage engineering and acoustic consultants' installations into our design.

The final decision to build depended upon an acceptable tender within the very strict cost limits set by the City. Costain Construction were successful and started work on site in April 1979.

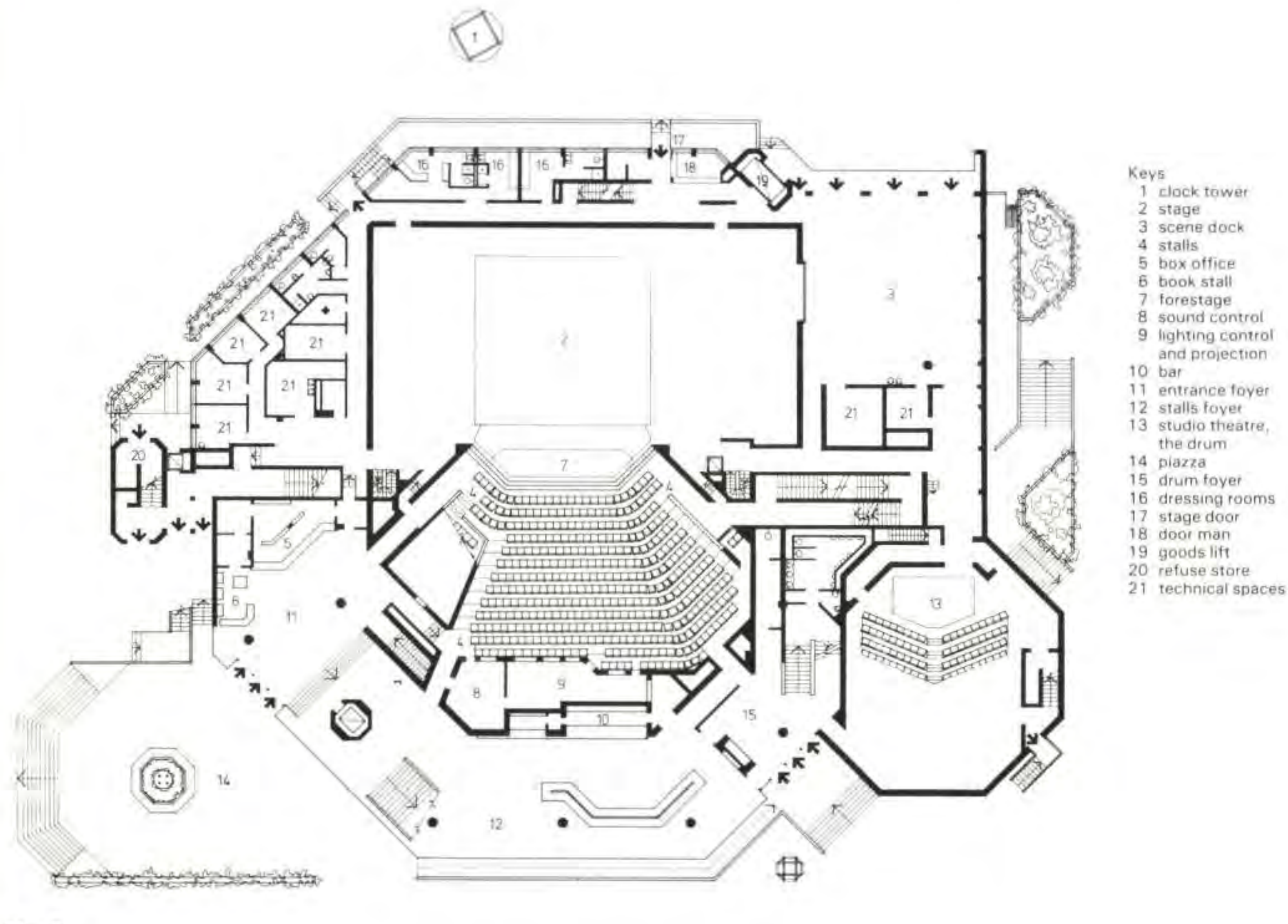


Fig. 2
Ground level plan

Fig. 3
Main entrance and foyers

Fig. 4
Foyer staircase

The brief

Carr and Angier recommended that a variable capacity auditorium be provided, ranging from a minimum of 750 up to 1200-1300 seats, together with a large stage, which would be suitable for the national companies without the usual touring compromises. The variable capacity was to be achieved by lowering a movable section of the ceiling and concealing the upper seating, but without detracting from the appearance that the auditorium in both forms was designed specifically for that form and no other.

At the same time, as 750 seats is a large house for the average repertory production, it was decided to upgrade a proposed rehearsal room into a fully self-contained studio theatre of up to 250-seat capacity, and for this to form a base for the resident Plymouth Theatre Company.

The auditorium planning and technical installations were required to cater for a full range of productions including drama, musicals, ballet, opera and concerts, and occasional conference use. The backstage areas were to meet the recommendations of the Arts Council for its proposed network of major regional touring theatres, dictating the size of the large structural proscenium opening, and requiring a large orchestra pit capable of accommodating a full orchestra. This was to be taken out of the forestage rather than the auditorium seating, resulting in a cranked safety curtain which is reflected in the profile of the flytower. Full wardrobe and production facilities were to be provided together with dressing rooms for over 100 performers, rehearsal facilities and administrative offices for both touring and resident companies.

Concept

The theatre is octagonal in plan, 52m across, situated on a corner site with pedestrian access from all sides (Fig. 1). The main auditorium, with its 29 × 15m stage and 28m high flytower, is at the centre of this octagon with four levels of public administrative or technical space around it, and a single-storey basement extending under the auditorium and stage left areas providing backstage facilities.

The auditorium is planned on three levels—stalls, dress and upper circles—accommodating 1271 seats, reducing to 768 when the movable section of the steel-framed, timber ceiling is lowered in front of the upper circle. The upper circle is symmetrical about the auditorium and building centreline, but the two lower levels are deliberately planned asymmetrically with a tongue of seating of the dress circle running down on the stage right side to stalls level.

Early acoustic model studies at the University of Cambridge strongly influenced the interior geometry and finishes of the auditorium. Reflection patterns from the variable height ceiling were studied, and an estimation made of the relationship between early energy (sound useful for intelligibility) and later energy (the reverberation important to sound quality). The auditorium design provides for clear speech, both natural or amplified, with further extension of the acoustic response by use of an assisted resonance system which prolongs reverberation electronically, allowing more scope for music performance.

The profile of the stepped ceiling has been determined by consideration of the acoustic response, sight lines from the upper circle and lighting angles for stage lighting mounted on bridges incorporated into the ceiling structure. Continuous line diffusers, supplying conditioned air to the auditorium, and cold cathode lighting run the width of the ceiling and emphasize its profile, while individual house lights are recessed into the ceiling planes.

Continental seating has been adopted allowing the use of side gangways only and so not requiring a major protected escape route through the front of house areas. Means of escape from all levels is provided by escape stairs on either side of the auditorium.

The front of house is planned on three levels interconnected by a 'grand' staircase and incorporates the box office, foyers, restaurant, buttery and two bars. The two lower levels are cantilevered out behind a structural glass wall displaying the foyers and staircase dramatically, particularly at night. The finishes are hard, as elsewhere in the building, softened only by an acoustic plaster ceiling and carpeting.

The studio theatre is also octagonal, 18m across, located about one corner of the main building and connected to the west end of the foyers by a common circulation space. It is planned as a single 9m high space with exposed services and lighting galleries, and removable seating.

Structure

The brief specifically required that the structural form should not restrict the planning of the internal spaces, and this approach has been followed throughout the building as far as possible. Early schemes considered the use of steel trusses for the auditorium seating structures, but a major reduction in the proposed building volume to meet the client's cost limits restricted the space available for structure and services and led to the development of an all in situ reinforced concrete design apart from the large span steel roofs of the auditorium, flytower and scene dock.

Foundations

The foundations are strip footings cast monolithically with the basement slab or individual pad footings, all founded on the Upper Devonian slate bedrock underlying the existing site fill. The presence of weathered clay bands in the slate and consideration of differential settlement restricted the allowable ground bearing pressure from 780 kN/m^2 to 1000 kN/m^2 depending on the size of foundation.

The basement areas are of watertight construction with an externally applied bituthene waterproofing membrane and were formed at three levels generated by the internal heights required and to limit the amount of expensive rock excavation. The 6m deep orchestra pit was excavated using controlled explosives and lined with a reinforced concrete wall and base prior to application of bituthene and construction of the pit proper.

Temporary dewatering was carried out during the construction by pumping from four shafts sunk alongside the pit excavation, lowering the ground water from its natural level at the underside of the basement slab.

At the start of construction, we were asked to report on the possibility of alkali-aggregate reaction occurring in the specified concrete, as this was then causing concern in Plymouth. While this was considered very unlikely as land-based aggregates were being used, it was decided to respecify a sulphate-resisting cement with a guaranteed low alkali content, the only one available, for the foundation elements and those most exposed to the weather. Unfortunately halfway through the substructure construction an industrial dispute halted the supplies completely, so the remaining stocks were only used on those elements considered to be at most, if any, risk.

Superstructure

The stalls are formed by a sloping flat slab with integral stepped risers, while the dress and upper circles are designed as stepped treads and risers spanning between raking beams on the radial support lines running through to the columns in the foyer areas.

The concrete side walls to the auditorium also support the circle structures, that at stage right spanning between the octagonal foyer lift shaft and the flytower over intermediate support as available while the stage left wall cantilevers in stages from the basement up to the rear of the upper circle.

The dress circle treads and risers are of simple design allowing the contractor to precast these elements, although this was done with limited success. The upper circle on the other hand is designed as an in situ beam grillage with torsion moments at the changes in direction of the seating taken out by the radial cantilever beams and ribs running parallel to the side walls introduced for that purpose. A stiff ring beam is used to connect the front ends of the cantilever beams and the side walls and supports a cantilevered front row of seating.

In the front of house areas, the architect's requirement for no perimeter supports, and the minimum of support elsewhere, resulted in a line of five octagonal columns and the foyer lift shaft as the only exposed vertical structure. The suspended slabs to the two upper foyer levels are designed as grillages of flat beams of minimum depth with interconnecting thin slabs, cantilevering out past the support line to the building edge and tied back into the dress circle structure. The first, wide flight of the foyer staircase is also used as an integral part of the structural support to the first level. The structural glass wall is supported at ground level but derives horizontal restraint from the suspended foyer slabs through glass fins with connections which allow for long-term deflection of the slabs.

All concrete in the front of house areas has a grit-blasted granite aggregate finish to complement the granite-faced blockwork of the wall surfaces. The need for the octagonal columns to be as slender as possible led to the selection of a 40N mix for all exposed aggregate concrete for consistency of colour, and this, coupled with the 10mm maximum aggregate size subsequently selected from grit blasting trials, resulted in some problems with drying shrinkage in the long sections of perimeter upstand required to be cast without joints.

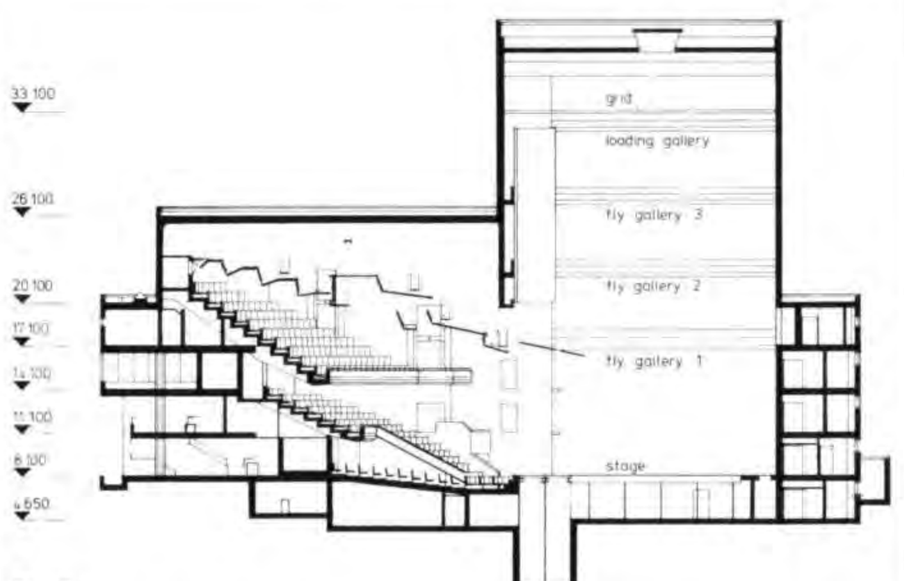


Fig. 5a
Cross-section: large auditorium

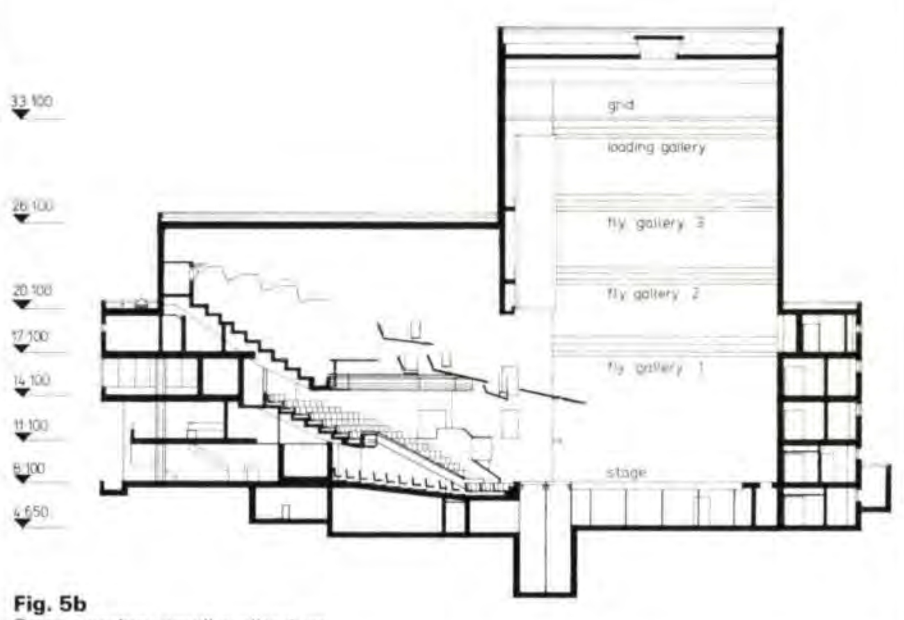


Fig. 5b
Cross-section: small auditorium

The administrative offices are suspended above the foyers between the ends of cantilever shear walls on the radial column lines, which are tied into a frame formed by the upper circle raking beams and beam strips in the office floor. The curtain walling to these offices is suspended from roof level again allowing for long-term deflections of the structure behind. Elsewhere column and flat slab construction is used with edge frames providing support to perimeter curtain walling or blockwork panels. Building stability is provided by concrete stair cores either side of the proscenium arch, the auditorium side walls and the blockwork infilled frame of the flytower. The 380mm thick cavity walls of the flytower also afford the necessary sound insulation from traffic noise while at the upper levels they are lined with 50mm woodwool slabs for sound absorption. The two-storey plantroom structures located either side of the flytower are separated from it, and the auditorium, by corridors or builderswork voids again for sound isolation.

The three main steel trusses of the auditorium roof span 24m from cantilever columns extending above the foyer columns to corbels on the face of the deep beam spanning the proscenium arch. To limit moments in these columns the trusses are seated on sliding bearings at the corbels. Bolted site splices at midspan enabled the trusses to be transported to site in two sections. Secondary beams support a double skin woodwool roof, which serves as the structural roof deck to the 'superfelt' roof system, and as an acoustic barrier. The lower level of woodwool is screeded on the underside in places to provide sound reflection.

The steel-framed ceiling elements, designed by a specialist subcontractor, are suspended from the roof steels. The three movable sections of the ceiling are driven by synchronized screw jacks mounted in telescopic towers. Access for maintenance is afforded by a network of catwalks and the veneered plywood panels of the ceiling itself.

The flytower roof beams support a single skin woodwool deck and are braced to resist the lateral forces generated by the stage suspension lines. A steel grid and three levels of side galleries are suspended from these beams.

Mechanical services

Ventilation and air-conditioning

Initial studies into the feasibility of air-conditioning both auditoria and the front of house areas, as proposed by the brief, had to be abandoned at an early stage when cost limits were exceeded and the major reduction in building volume severely restricted space for services. Air-conditioning of the main auditorium only was retained, and further studies on the principles of distribution resulted in the decision to supply from high level and extract from under the seats. Four plantrooms have been utilized to locate major items of plant as close as possible to areas being served and offering the minimum of intrusion into the planning.

A single zone constant volume air handling unit, located in the stage left plantroom, supplies conditioned air to continuous linear diffusers mounted in the ceiling planes and in the soffits of the upper and dress circles. The diffusers in the moving section of the ceiling are connected via plenum boxes and vertically suspended flexible duct to the fixed ductwork running at high level. Ductwork sized for low air velocities, laboratory testing of the diffuser assembly and extensive attenuation has ensured that the design noise level of PNC 20 + 2dB is met.

Air is extracted into builderswork plenums beneath the dress and upper circle seating including that connecting down to the stalls, and also at high level, to be either recirculated or discharged to atmosphere. The high level system also serves as the smoke extract from the auditorium in an alarm situation. Cooling is provided by a packaged liquid chiller of 457kW duty located in the basement plantroom with heat rejection from a cooling tower located in the service yard

Separate mechanical ventilation systems supply filtered, tempered air to many zones of the theatre including the foyers, toilets, kitchen, backstage areas and equipment rooms. Some cooling has been introduced in the general system to offset heat gains in dressing rooms. Two main services risers link the five levels of the building with distribution on each floor generally through the corridors, the underseat voids and above the auditorium ceiling. The studio is mechanically ventilated from a separate local air handling plant, again supply being at high level through central circular diffusers with perimeter extract at high and low levels.

Heating

Hot water and heating is provided by two atmospheric gas boilers with an output of 1560kW. These supply low temperature hot water heating circuits, a constant temperature circuit serving heater batteries in the ventilation systems, and two 3000 litre hot water storage calorifiers. Space heating radiators and convectors operate on two zoned circuits and are sized to offset fabric losses. The natural convective perimeter heating to the foyer areas helps to offset condensation on the glass wall and is supplemented by warmed supply air from the underseat ductwork systems. Extensive vibration isolation of plant and piped services has been installed in plantrooms and other critical areas.

Protective and public health services

The local authority's requirements for fire protection included a full hosereel system with an automatic sprinkler installation covering the basement, stage areas and scene dock with a drencher above the proscenium arch. No sprinkler coverage beneath the demountable stage area was accepted on condition that alternative infrared detection was provided. The building insurers later required the kitchen and production wardrobe areas to be sprinklered. In the event of fire all supply plant and the gas supply automatically shut down and extract fans expel smoke from the building, supplementing the natural smoke ventilation at the heads of escape stairs.

Other public health installations serve the foyer toilets stacked on five mezzanine floors, and backstage toilets and showers on all levels. Separate foul and rainwater systems run to the site boundary before discharging into a combined sewer.

Electrical services

Electrical supply

The main supply to the building is provided by the South Western Electricity Board at a pressure of 11,000 volts. Two transformers each rated at 800KVA, together with high voltage switchgear and metering equipment are located in a basement sub-station adjacent to the main intake switchroom.

General lighting

Lighting levels throughout the building are generally in accordance with the guidelines given in the CIBS Lighting code, although the main criterion throughout was quality of light rather than quantity. The general lighting in the building was considered in three distinct and separate areas: Backstage—dressing rooms, offices and workshops; public areas—foyers, restaurants and bars; and the two auditoria. The majority of the back-stage areas are lit by fluorescent luminaires with the levels being related to the various activities, i.e. workshops, offices, etc. Lighting in the dressing rooms is tungsten for general illumination and bare tungsten lamps for dressing tables. Lighting for the foyers, restaurants and bars is from fully recessed 'spill ring' luminaires with spherical spot lamps mounted on multi-circuit lighting/sound track for accent and display lighting. This track also carries matching loudspeakers for the public address system. All lighting in foyer areas is dimmer controlled.

Lighting within the main auditorium has been designed for the various dramatic and musical uses and its use as a conference venue in both its small or large format. A general background level is provided using fully recessed downlighters set into the various ceiling elements, both moving and

Fig. 6
Auditorium in large format

Fig. 7
Upper circle and lighting bridge

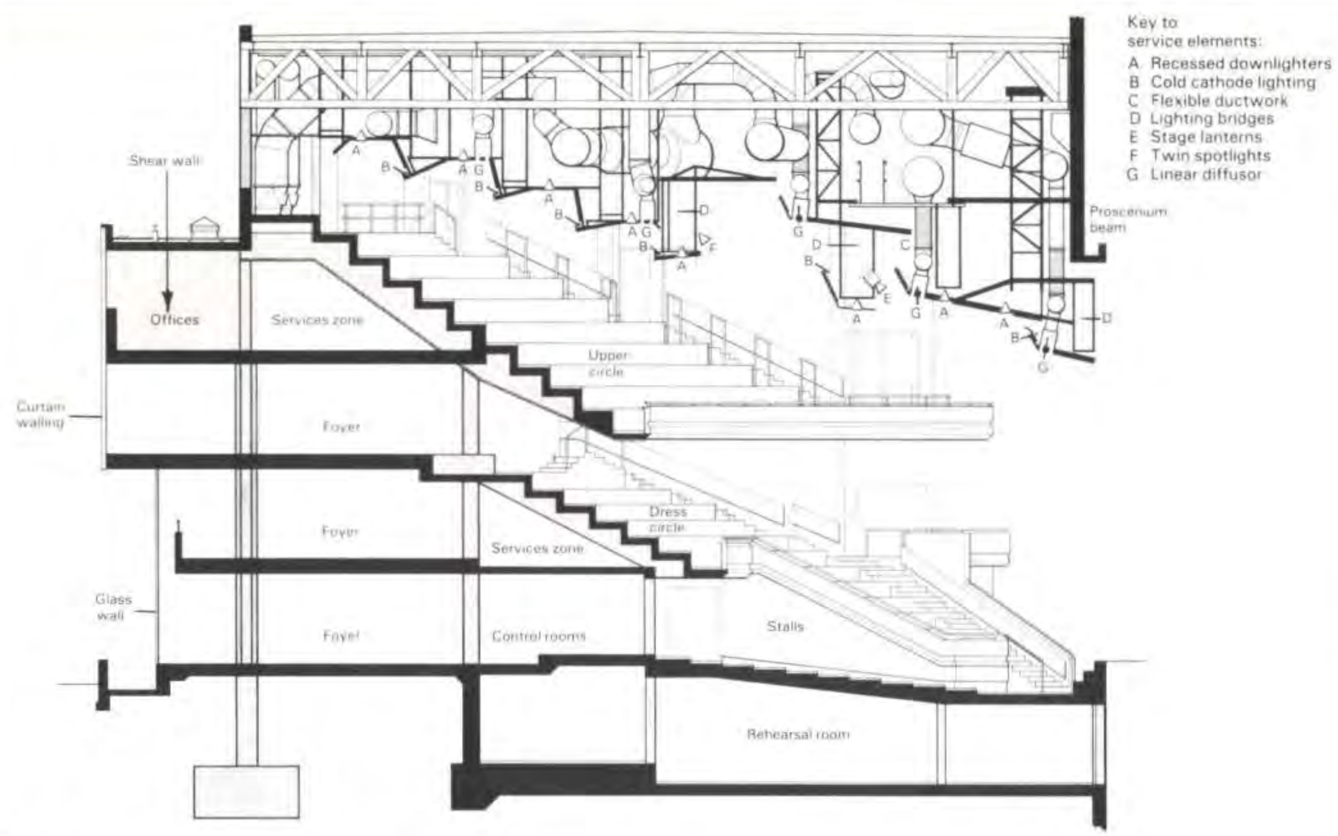


Fig. 8
Auditorium section showing structural frame and high level services

fixed, and under the dress and upper circle levels. To this can be added spot lighting from twin 300 watt halogen lamps mounted amongst the stage lighting above the ceiling to provide accent and sparkle, and/or continuous cold cathode strip lighting concealed along the edges of the ceiling elements to enhance and feature the wood veneered panelling of the main ceiling. The control of the auditorium lighting is through a 19-channel dimmer rack and control unit supplied as part of the stage lighting package enabling the entire installation to be preset in 10 different states. When all the lighting is in the full up position the measured illuminance in the auditorium averages 1200 lux, although obviously not intended to be used at this level.

The lighting in the studio theatre is basic with a single dimmer controlling tungsten downlighters.

Working lights

Stage working lights for both the main and studio theatre cover the stage and all other stage area working spaces, including the space above the ceiling in the main auditorium. There are two categories of working lights, the 'Whites' which are for manual use outside actual performance periods and the 'Blues' which give a very low level of illuminance, usually using blue lamps, for use during a performance.

Emergency lighting

The emergency lighting throughout the building is provided from a central 240 volt DC/DC battery system serving both the maintained and non-maintained load, totalling 12.0 kW. The maintained system covers all public areas and is in operation whenever the public have access to the building. Within the auditorium the system covers all stairs, aisles, sound lobbies and exit signs. Approval has been given by the local authority for blacking out all the maintained lighting in the main auditorium, with the exception of the exit signs, for up to 30 seconds for theatrical purposes. The non-maintained system comes into operation immediately there is a mains failure and covers the entire theatre. In an emergency both the maintained and non-maintained systems operate.

General power

Supplies are provided to the kitchen, mechanical plant, four lifts and the stage engineering equipment. Otherwise the power requirements of the building are minimal and restricted to 13 amp socket outlets.

Fire alarms

The fire alarm system is a multi-zone manual and automatic system. Automatic detectors are either smoke or heat actuated except in the high risk area immediately below the stage which is protected by highly sensitive infra-red scanners. Indication is provided at the stage door with a direct line to the local fire brigade. Whilst the public have access to the building the system is held in its manual mode but when the building is closed to the public the system is automatic. The changeover from manual to automatic is effected by micro-switches which are incorporated into the main access door locks, the main indicator panel indicating the system mode at all times. No audible alarms are activated in the auditoria in the manual mode, evacuation being supervised by the theatre staff after a public announcement.

Fig. 9
Ceiling structure and services installations

Fig. 10
Proscenium opening with orchestra lift lowered

Fig. 11
Studio theatre

(Photos: Harry Sowden)

Security system

The theatre has a comprehensive security system incorporating raid buttons at all cash points, door locks and contacts, breaking glass, window and bar grille contacts, all indicating on a central security panel in the stage door keeper's office with a direct link to the local police station.

Stage lighting

In the main theatre, the stage lighting is based on a 274 channel (dimmers) Rank Strand Galaxy memory control system operating: 185 No. 2.5 kW channels, 74 No. 5.0 kW channels, and 15 No. 10.0 kW channels. The studio theatre has a similar control system operating 116 No. 2.5 kW channels and 4 No. 5.0 kW channels. In the main auditorium a number of colour change outlets and independent outlets to be used to power special effects, etc. and supplies, provided for the main follow spots, are also installed.

Sound and communications

A comprehensive system of communications, essential to the running of a modern theatre, has been installed. This comprises a number of independent but inter-related systems offering the following facilities:

Show relay, stage calls, stage door calls, public address, stage sound, cue lights, talk back, closed circuit television, bar warning, hearing aid loop, staff 'bleeper' system and telephones.

Stage power

Items of stage equipment for which supplies and control wiring are provided include the two orchestra lifts, the proscenium towers, the safety curtain raising motor, 12 powered flying sets, a bulk supply on stage for imported revolves, lifts, etc. and the three lighting bridges incorporated into the moving ceiling. Cables to the bridges are mounted on folding cable trays fed from trunking systems run at high level in the auditorium.

Credits

Client:
Council of The City of Plymouth

Architect:
Peter Moro Partnership
(formerly Peter Moro and Partners)

Quantity surveyors:
Davis Belfield & Everest

Theatre consultants:
Carr & Angier

Acoustic consultants:
Sound Research Laboratories
in conjunction with Arup Acoustics

Main contractor:
Costain Construction Ltd.

Mechanical services sub-contractor:
Newman and Watson (South West)

Electrical services sub-contractor:
B. D. Copp & Co. Ltd.

Design for seismic loading in the North Sea: A state of the art summary

Edmund Booth

Introduction

During the early days of offshore development in the North Sea it was not usual to consider seismic loading. This was partly due to the feeling that structures strong enough to resist the high wind and wave forces of the area would self-evidently be adequate seismically and partly due to a lack of agreed data on appropriate earthquake loads. There was perhaps also the feeling that seismic loading was just one more unnecessary consideration with which to burden the designer.

More recently, there has emerged a fairly general agreement[1] from among all parties involved in the study and development of the North Sea that the area is one of low to moderate seismicity, which should be checked for when designing the very important structures to be sited there. It is now common to specify at least nominal levels of seismic loading to be considered when designing new structures located in the Norwegian sector. The understanding of the seismicity of the region is also improving rapidly, though generally agreed levels of loading have still to emerge.

Levels of earthquake loading

The basic consideration of earthquake loading involves an ultimate limit state load, analogous to ultimate wind and wave or gravity loads, with an associated return period of the same order as the life of the structure. Both DnV[2] and API RP2A[3] (in conjunction with ATC 3-06[4]) require such consideration for offshore structures and specify the return period and associated load factor to be used.

The ultimate limit state loading is defined as that loading beyond which the structure becomes unfit for its intended purpose. The collapse load or survivability load is the maximum loading the structure can sustain.

The difference between the two load conditions depends on the degree of redundancy in the structure, the nature of the loading and the inherent ductility of the materials used.

The aim of the ultimate limit state check is to ensure that the risk of structural damage due to an earthquake during the life of the structure is the same as that due to other types of loading. Rational methods exist[5] for choosing a load factor appropriate to the chosen return period to ensure this equality of risk (and hence produce an economic design) but as far as is known, such methods have not yet been used for the case being considered.

However, earthquakes are distinct from other types of environmental loads in two respects. Firstly, the ratio of long return period (say 10,000 year) earthquake load to short return period (say 50 year) load is much greater than is the case for wind and wave loads. This is due more to the effect of the probability of the closeness of the structure to the epicentre of an earthquake than to the distribution of earthquake magnitudes with time. Secondly, the extreme magnitude or survivability earthquake loading tests the structure's ability to withstand large amplitude cyclical loads. This is not necessarily ensured by designing for adequate strength against the 'ultimate limit state' earthquake, in the same way that designing a joint against ultimate wave loads does not ensure its adequacy in fatigue.

Both DnV and API RP2A require separate consideration of the survivability limit state. It is usual in buildings to account for this limit state indirectly by specifying special detailing rules, minimum levels of reinforcement and minimum amounts of vertical and horizontal ties to give overall continuity to the structure.

A third limit state may need to be considered for the effect on mechanical equipment. Horizontal accelerations at deck level are considerably magnified during an earthquake[10] and may adversely affect plant, with possible safety and economic consequences. It is noteworthy in this context that the 1976 Tangshan earthquake affected several steel jackets in the South China Sea, writing off all the plant, although the structures survived undamaged. Simple detailing rules rather than complex analysis is probably the most appropriate response to this problem.

Seismicity of the North Sea

The North Sea is an 'intraplate' region some distance from the tectonic plate boundaries where most earthquakes occur. This makes it more difficult to associate earthquake with obvious geological features such as fault planes. However, there is a growing body of both instrumental records and geological evidence of past earthquakes (see Fig. 1). NORSAR[6] lists over 40 events of magnitude 5 or greater that have occurred over the past 200 years in the North Sea and adjacent areas (50°-70°N and 5°W-15°E). Five earthquakes of magnitude between 5 and 6 have been recorded instrumentally in the North Sea since 1927 and a value of 6.4 is assigned to the 1904 Oslo fjord earthquake.

There is some general agreement[1] that an earthquake with magnitude as high as 7 could occur in the region. To put this in context, a magnitude 5 earthquake is generally considered to be the threshold at which damage occurs in a well-built, engineered structure.

NORSAR[6] have prepared zoning maps for the Norwegian sector of the North Sea which distinguish three levels of risk. The regions with the highest risk are assigned peak bedrock accelerations of 0.06g and 0.19g for 100 year and 10,000 year return periods respectively. This contrasts with an earlier study[7] for the Department of Energy carried out by Ove Arup & Partners which suggested an upper bound of 0.10g for a 100 year return earthquake.

On the basis of data resulting from the recent very significant improvement in the seismic detection network in the area[8], Browitt[9] of the Institute of Geological Sciences (IGS), Edinburgh, has tentatively reported a link between instrumentally recorded activity and known geological features. The minor earthquakes recorded over the last two years have been concentrated in the Viking Graben area (see Fig. 1) and an $M_B = 4.7$ earthquake occurred on the edge of this area in July 1982. The IGS work is still in its early stages, but if the susceptibility of the Viking Graben to minor earthquakes is associated with a propensity to more major events this could have important consequences for the oilfields (which include Statford, Brent and Gullfaks) in the locality.

Ambraseys[1] has suggested that a study should be made of the tectonic structure of the quaternary layers of the North Sea in an attempt to correlate the structure with known seismic activity, and it is understood that the Department of Energy may commission a re-examination of the seismicity of the region on this, or some other, basis.

Published studies of structural response

Various studies have been made of the likely response of offshore structures to seismic loading. Dowrick[7] investigated two typical concrete gravity platforms and two typical steel jackets standing in 140m of water and concluded that in the North Sea seismic effects were likely to be similar to or less severe than those due to wave forces when considering 100 year return periods. Selnes[10] studied similar structures and reached a similar conclusion. Possible situations in which seismic loading may however govern are the leg/deck connections of concrete platforms, very long horizontal deck cantilevers and articulated structures.

Work reported at Utrecht[1] suggests that the extreme 'survivability' earthquake may prove more generally critical in the design of North Sea platforms in 150m of water. Norske Shell[1] have also reported that the 100 year return earthquake may prove critical in the design of conventional piled and gravity structures in 340m water depth in the North Sea.

In addition to these investigations there have been a number of studies[11-13] of the response of concrete and steel platforms to the much more severe seismic conditions of the Gulf of Alaska and elsewhere.

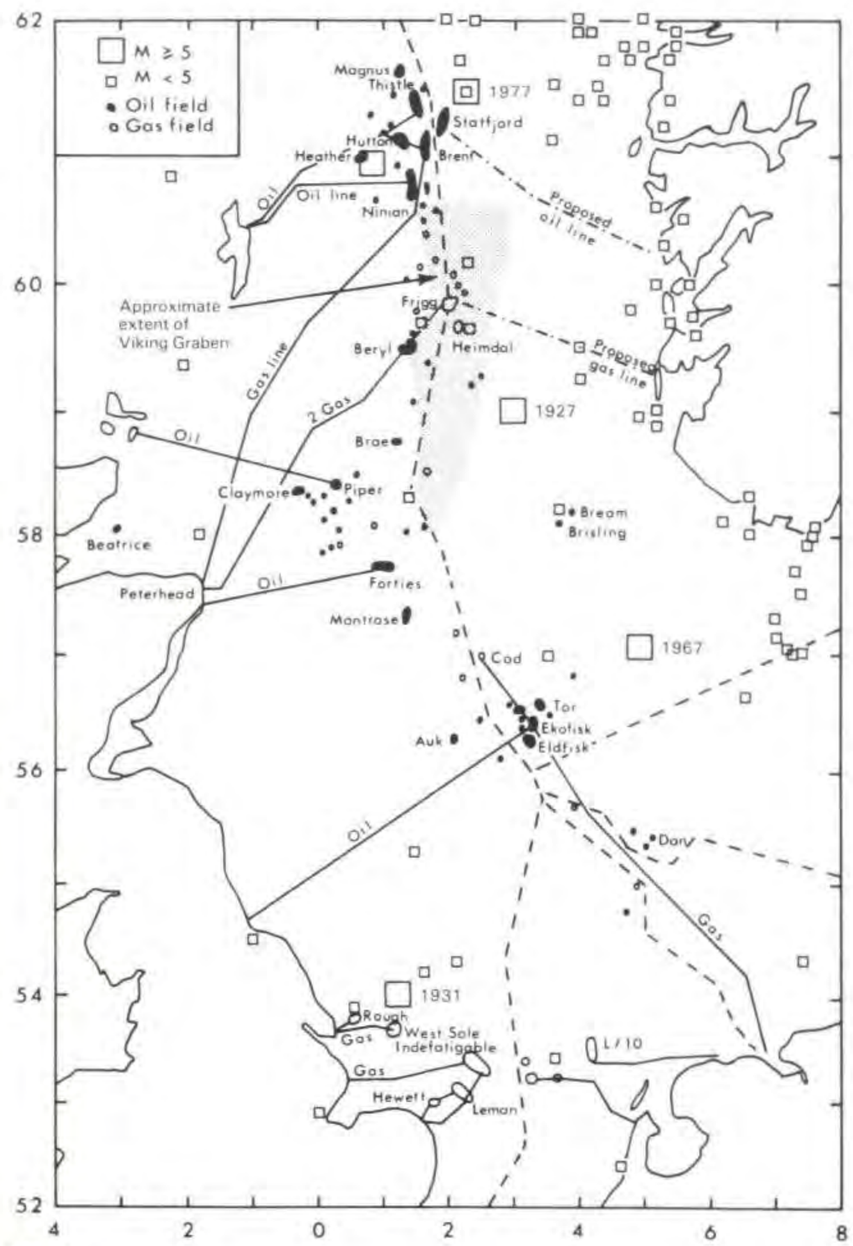


Fig. 1
Epicentral map of the North Sea, showing locations of oil and gas fields, and the Viking Graben (after Browitt[15])

The effect of earthquake loading on foundation material may also need to be considered. It is known that even the shallow slopes typical of the oil producing areas may become unstable under earthquake loading and there is evidence from off the north west Norwegian coast of earthquake-induced slides occurring in the past which have shifted massive amounts of material. There is also the possibility of liquefaction taking place. Both considerations would affect not only production facilities but also pipelines, which because of their length, might be more likely to be affected. Work has been reported on these geotechnical aspects[7, 14], but there appear to be no generally agreed answers at present.

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Acoustics in Arups 1965-80

Derek Sugden

Stephen Reiss wrote to Ove Arup in October 1965 asking whether Arups could give an opinion as to whether a malt house in the village of Snape—some five miles from Aldeburgh—could be converted into a concert hall for the Aldeburgh Festival.

The answer was yes, and the opening of Snape in June 1967 was the beginning of acoustic work in Arups.

Following that royal occasion—which was repeated in exactly three years time—Arup Associates became involved in many buildings which relied primarily for their success on the creation of an acoustic. In 1969, following a concert given by the London Symphony Orchestra (LSO) to raise money for the new Snape Foundation, Arup Associates were appointed to help the LSO find a building suitable for conversion into a rehearsal room. Eventually the London Philharmonic Orchestra joined in and the Southwark Rehearsal Hall Trust was formed and became Arup Associates' official client. Holy Trinity Church, Southwark, was the chosen building following acoustic tests with both orchestras playing in hard hats borrowed from John Mowlem. The Church was virtually destroyed by fire on the night before the contract was due to start for its reconstruction and restoration. It was opened in May 1975, with a concert given by the LSO and LPO, as the newly-named Henry Wood Hall and became, overnight, London's leading rehearsal hall and orchestral recording studio.

During this period the new Music Centre for the University of East Anglia was designed and built, being opened in 1974. In October 1975 the reconstructed Theatre Royal, Glasgow was opened as the permanent new home for Scottish Opera. In 1975 Frank Matcham's Opera House at Buxton was restored in six months and became the home of the newly created Buxton Festival.

Apart from these highlights of acoustic work in auditoria, a limited amount of work was done in setting acoustic standards and achieving quite rewarding results in open office design. This was particularly so at Penguins, one of Arup Associates' earlier office buildings right on the edge of London Airport.

Although during this period acoustic work was limited to work within Arup Associates, various reports and studies were done on existing buildings for the Decca Record Company. These included a report on the possibility of using the newly restored St. John's Smith Square as a recording studio, a report on the use of the Opera Centre in Commercial Road as a recording studio and two studies; one for the conversion of a skating rink and the other for the conversion of an old cinema into recording studios for large scale concerts and operas.

Two of the most interesting projects were a report on the famous Orchestra Hall in Chicago for the Chicago Symphony Orchestra and Sir Georg Solti, and the design of a Reggae studio for Federal Records in Kingston, Jamaica.

Fig. 1
The Maltings, Snape
(Photo: John Donat)

Fig. 2
Music Centre, University of East Anglia
(Photo: Richard Einzig)

Fig. 3
Henry Wood Hall,
(Holy Trinity Church, Southwark)
(Photo: Peter Baistow)

Fig. 4
Theatre Royal, Glasgow
(Photo: Arup Associates)

The formation of Arup Acoustics

During the '70s two papers were presented at the Institute of Acoustics and one or two members of Arup Associates – John Campbell, Peter Warburton and John Moss – began to take an interest in the subject, and one or two members of the practice took one- and two-day courses at Sound Research Laboratories (SRL). Through these lectures and courses a dialogue was developed with Peter Parkin at the Building Research Establishment (BRE) and Richard Cowell at SRL. At this time Peter Parkin was involved in the setting up of an acoustic research centre within the Martin Centre of the School of Architecture at University of Cambridge. Bill Howell, who was then the new Professor of Architecture at Cambridge, was deeply involved in the creation of this aspect of research work and his tragic death resulted in severe limitation of the work which eventually led to Peter Parkin taking up an appointment as Research Professor at the University of Southampton on his retirement from the BRE. At this time, Richard Cowell was the leading consultant in SRL and, together with Professor Parkin, was revising the standard textbook in this country 'Acoustics, Noise and Buildings' by Parkin and Humphreys.

During this period Arup Associates were not carrying out any instrument or measurement work in acoustics and usually engaged SRL for this and on many occasions were seeking second opinions and advice on many aspects of acoustic design from Richard Cowell. The close associations that developed during the '70s led quite naturally to discussions about providing a more organized and professional acoustic design service within Arups. This was influenced, if not stimulated, by many requests from other parts of the Partnership for advice in the widening field of acoustics and noise attenuation.

A paper was presented to the Ove Arup Partnership in the late '70s and in May 1980 the name Arup Acoustics was registered and a consultancy formed with Richard Cowell, John Martin and myself as Principals, and with Professor Peter Parkin as Consultant.

Fig. 5
Buxton Opera House (Photo: Martin Charles)

Fig. 6
Bedford School (Photo: Crispin Boyle)

Arup Acoustics now

Richard Cowell

When we fixed the brass plate (aluminium, in our case) in May 1980, we were very fortunate to have work to hand. I had been asked to continue some of my previous work on a sub-contract to my previous employers – Sound Research Laboratories Ltd. – and so maintain continuity of service to clients for the Theatre Royal, Plymouth, the International Conference Centre Broad Sanctuary and the Harrogate Conference Centre. I also continued with the acoustic design of Bedford School Hall and its sound reinforcement system. Ove Arup and Partners asked for a specification for advice on a suitable public address system within the National Exhibition Centre Hall 7.

Very soon, Professor Parkin was asked to advise on upgrading the speech reinforcement in Canterbury Cathedral following his work at Westminster Abbey. Derek Sugden was invited to carry out a review of the Royal Festival Hall acoustics working with Peter Parkin, who for many years has given guidance to the RFH in acoustic matters.

My apointment as consultant to the National Theatre on acoustic matters was also confirmed in the early days of the Practice. This resulted in an early visit to Germany to assess the acoustic implications of moving the 'Passion Play' from the Cottesloe Theatre to a highly reverberant Church in Cologne, and involved developments and staging within the National Theatre building and some staff training.

Joint venture project

In the summer of 1980 Arup Acoustics were invited by Foster Associates to carry out acoustic consultancy for the Hongkong and Shanghai Banking Corporation Headquarters in Hong Kong; this has been a joint venture with Dr. Tim Smith (an ex-colleague of mine from SRL) who is responsible primarily for the services noise/vibration control and design against wind noise. This project has called for a wide variety of studies and forms a major component of Arup Acoustics' work. Apart from the acoustic design for the new building we have been working with Professor Peter Grootenhuis advising on the necessary protection from noise/vibration from the Mass Transit Railway Short Island Line extension, which at one point will be approximately 1m from the basement diaphragm walls. To satisfy the relevant Government bodies and to avoid unnecessary disturbance to neighbours, construction noise and vibration have been monitored. The construction of a sea water tunnel from Star Ferry to the site involves noise and, in particular, blasting vibration. In close collaboration with Ove Arup Partners, Arup Acoustics are recommending guidelines for limits on charge sizes contributing to the specifications and will carry out monitoring when the trial blasting and tunnel blasting occurs next year. A check on the insurance policy has seemed appropriate!

More staff needed

In 1980 we were also invited to carry out acoustic consultancy for the Alexandra Palace Redevelopment and a study of the scope for improving conditions for music making in Christchurch, Spitalfields. The workload encouraged us to take on more staff. Paul Gillieron joined us after nine months and the workload expanded further. We were soon searching again for additional staff and Peter Mapp came to Arup Acoustics in June this year. We are still looking for a further consultant.

Our commitments in Hong Kong, particularly for the Bank project, called for a presence there. Initially we asked Dr. Lindsay Newmann, who previously worked for Ove Arup & Partners Building Engineering Group 6, to assist with communications in Hong Kong when she moved out there to work as a services engineer in the Hong Kong office in 1981. Her interest, involvement and experience in acoustics have grown to the extent that she is now able to handle much of the site monitoring of noise and vibration, approvals and negotiations with neighbours, supported by occasional visits by London staff.

The last 18 months have seen a rapid growth in the commitments of Arup Acoustics, perhaps best indicated by a review of the project work.

Auditorium design

In the field of auditorium design, apart from studies for the Royal Festival Hall and the National Theatre, we were asked by Hugh Creighton, acoustic consultant, for the Barbican Development and the architects, Chamberlin Powell & Bon, to help with commissioning tests, reports and second opinions on the acoustics of the Concert Hall. The recent opening of the Theatre Royal, Plymouth, has been an important landmark, as the building contains a number of interesting features, important to the acoustics, including 'passive' sound reinforcement under deep balconies, a ceiling which can be lowered, an orchestral enclosure formed from structural fabric stretched on screens, an inflatable overhead reflector and a 90-channel assisted resonance system incorporating sophisticated controls.

Proposals for renovation and development at the Everyman Theatre, Cheltenham, the Grand Theatre, Swansea, and at the Playhouse in Northumberland Avenue, London, are currently being studied. We are also acoustic consultants for the University Arts Centre in Swansea and for a Sedgwick Forbes development at Aldgate, London, which includes a 350 seat theatre/conference auditorium surrounded by sports facilities. Overseas acoustic tests and assessments were carried out at the Palais des Sports in Paris to establish the scope for its use by the Paris Opera and we are contributing to the design of small auditoria within the Al Shaheed, Baghdad, project (Ove Arup & Partners) and at Tabuk, Libya (with Theatre Projects Consultants Ltd.).

Acoustic design, and particularly control of noise and vibration from mechanical services are now recognized elements of office development. Designs for control of services noise are being developed for Royal London House (Ove Arup & Partners), Atlas House (Ove Arup & Partners) and Finsbury Avenue (Arup Associates) developments in the City. Prototype offices in Shellmex House, London, are subject to acoustic testing – our client is the GMW Partnership.

Proposed offices at Twickenham (Ove Arup & Partners), Wimbledon (Arup Associates) and Charing Cross (Ove Arup & Partners) are located over railways and the case for or against floated structures has been studied. We have contributed to Ove Arup & Partners consultancy to prospective tenants of office development at the Adelphi site in London with Arup Associates. We have also advised on aspects of office building for IBM in Hampshire, DEC Park, Reading and Wiggins Teape, Phase 2, Basingstoke.

The recently opened London International Financial Futures Exchange at the Royal Exchange Building in the City (with Ove Arup & Partners) posed interesting problems for Paul Gillieron, including insulation to protect neighbouring tenants from the very loud noise of the dealing operatings and mounting of mechanical services plant on the lightweight internal structure. Protecting of adjacent tenants from construction noise was particularly challenging.

Vibration studies

The prevalent problem of construction/demolition noise/vibration take up a great deal of our time. Piling vibration surveys and monitoring have been carried out at Epsom, Farnborough, Maidstone and Cheltenham.

Surveys of vibration caused by demolition work around the Lloyd's site in the City and in Hong Kong have, in each case, helped towards the definition of control procedures. The effect of vibration on green concrete was a particular issue during assessment of vibration during removal of river bed piles at the Thames Barrier. Structural transmission of noise from construction activity was checked during Phase 2 of the Cromwell Hospital development and measures were taken to protect wards in Phase 1. Improvements in techniques and instrumentation will allow Arup Acoustics to contribute effectively to the work of the Partnership in these areas.

The acoustics of sports halls are being tackled with the Sports Council through the Nicholas Grimshaw/Ove Arup & Partners scheme with Bovis. We are involved with

Fig. 1
Plymouth Civic Theatre (Photo: Ove Arup Partnership)

leisure centres at Hackney and Newport Gwent and two smaller school halls. In these cases the conflicts between the need for rugged or easily cleaned linings and the introduction of sound absorption, the quality of P/A systems and the need to keep noise in, are particular areas for concern.

The potential noise nuisance from leisure activities is brought home by two commissions we have had to contain noise from discothèques in Earls Court and in Dublin.

Our work with regional offices is expanding. At an early stage, we surveyed vibration and noise for mechanical plant at GREA Leeds via Ove Arup & Partners (Sheffield) and made recommendations for reverberation control in the Elswick Pool, Newcastle, via Ove Arup & Partners (Newcastle). We are now working with the Newcastle office on acoustic aspects of the Gateshead Administrative Headquarters. With the new Ove Arup & Partners Warwick Office a report is being prepared on the noise aspects of the proposed Coventry East By-Pass. Some calculations on noise propagation from the Chepstow by-pass have been carried out for Ove Arup & Partners (Cardiff) whom we are also assisting with acoustic aspects of the GREA/Wales Gas Development in Cardiff. A report was recently completed for Shropshire County Council recommending an acoustic design strategy for the proposed Telford Courts through Ove Arup & Partners (Manchester).

In the field of education, we have contributed advice on acoustic aspects of developments at Eton School (Arup Associates) and Clare College, Cambridge (Arup Associates), Bedford School (Arup Associates) and the French Lycée in Kensington (Ove Arup & Partners).

The control of noise and vibration potentially disturbing to research work has been investigated for three projects – Kuwait Institute of Scientific Research (Ove Arup & Partners), Merck Sharp and Dohme, Harlow (Ove Arup & Partners) and Pfizer, Sandwich.

Service noise control

Other commissions include two dance studios (The Place, Euston and Sundance Studios, Hammersmith), studies for redevelopment at Riverside Studios, Hammersmith and two hospital projects where the brief is concerned mainly with service noise control: Beaumont Hospital, Dublin (Ove Arup & Partners, Dublin) and Royal South Hants Hospital (Ove Arup & Partners). We are assisting Ove Arup & Partners with their responsibilities for service noise control on the Lloyd's development.

Fig. 2
Elswick Park Swimming Pool, Newcastle
(Photo: Ove Arup Partnership)

Fig. 3
NEC Hall 7
(Photo: Ove Arup Partnership)

Fig. 4
London International Financial Futures Exchange (Photo: Ove Arup Partnership)

There has been a growing demand for our services in the design specifications and supervision of electro acoustic installations with the arrival of Peter Mapp who has particular experience in this field. Outline design for sound reinforcement at the Kano Stadium, Nigeria, was developed with Ove Arup & Partners. Peter is currently developing a sound reinforcement system for the Public Enquiry concerning extension of the Sizewell Nuclear Power Station, which is taking place at Snape Concert Hall and is carrying out a full review of the sound and translation systems in the proposed International Conference Centre at Broad Sanctuary. Upgrading of the sound reinforcement system in the Barbican Concert Hall is an important commission for Arup Acoustics.

We have devoted a little time to investigating the scope for working through R + D on research contracts dealing with acoustic topics and have contributed to proposed revisions of standard specifications for control of noise from mechanical services. At this early stage, there appears to be substantial scope for Arup Acoustics to contribute immediately and usefully to standard specifications. The prospects for research projects have to be seen over a longer time scale.

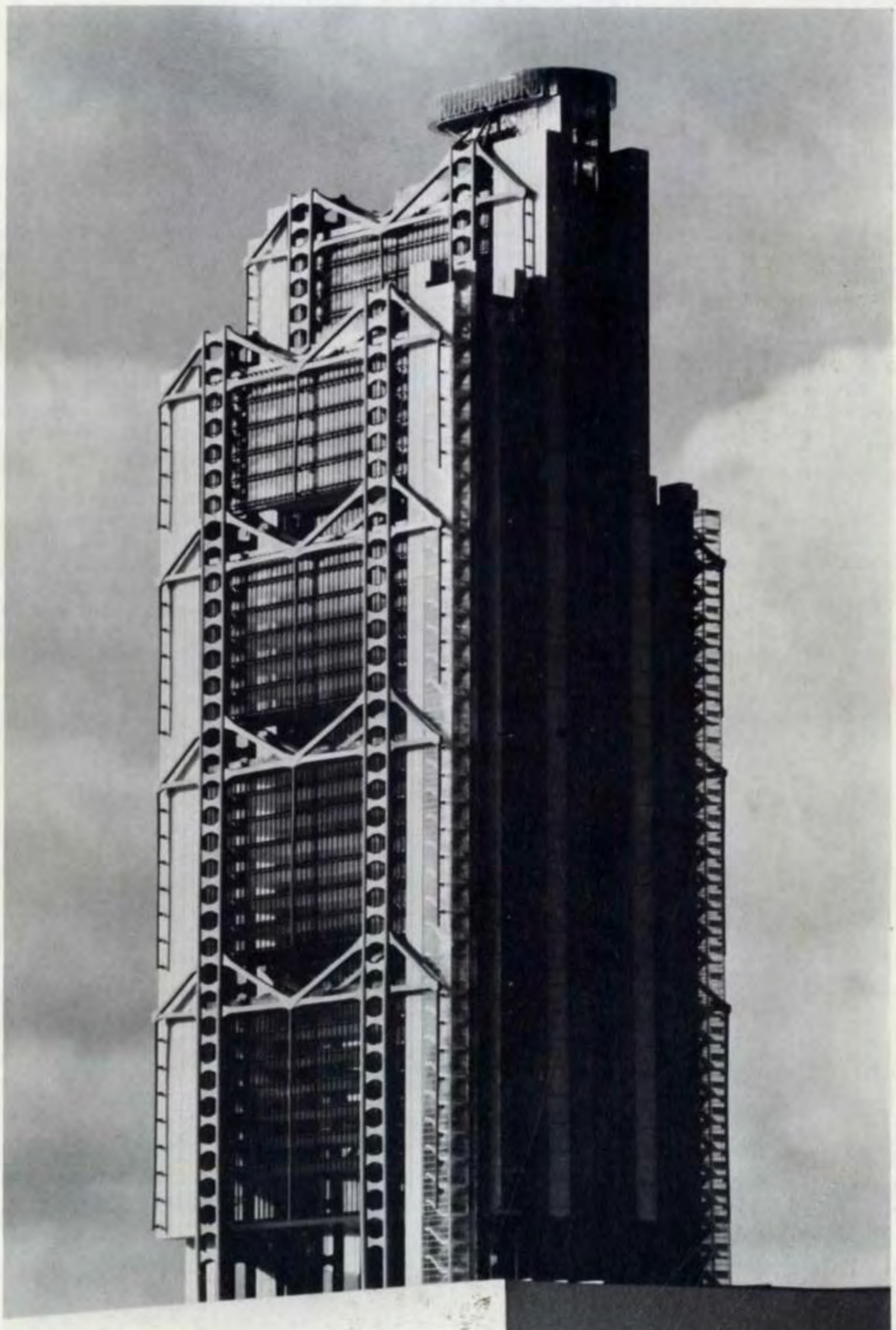
On its formation the Practice set out to cover all aspects of acoustics, noise and vibration within the construction industry. Without there being a specifically stated policy it set out with the idea of a small number of individuals who would specialize in different areas of acoustics. Paul Gillieron and Peter Mapp both read Physics for their first degrees—both therefore complement my first degree in architecture.

Our primary aims are clear—to develop effective and improving acoustic consulting services, teaching and research, and to enjoy it.

This embodies a concern to respond in particular to requests for help from within the Partnership and to encourage improved standards of acoustic design. We have no firm policy on the balance between internal work and work for outside clients except that we believe it healthy to retain a balance. We feel there can be substantial benefits from continuing to work with outside acoustic specialists with specific talents which complement and extend our own.

As with most other disciplines, the quality of consultancy depends on the calibre of the people doing the work. It is difficult to find the right staff and we foresee successful integration of new consultants in the next few years as central to our plans.

Prediction of workload is a hazardous business, but all the signs for Arup Acoustics are encouraging. There is a danger of overstretching the resources of such a small group, either by taking on too much work or by extending the subject matter sideways into the many potentially relevant acoustic disciplines too quickly. We aim for measured growth in the number and scope of our commissions.



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