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Jack Zunz
Michael Glover
Tony Fitzpatrick

This paper was presented and discussed at a meeting of the Institution of Structural Engineers, on Thursday 24 October 1985 and was originally published in The Structural Engineer, *September 1985.*

Introduction

Background

In 1929 Sir Vandeleur Grayburn, the Chief General Manager of the Hongkong & Shanghai Banking Corporation, decided that the Bank needed a new Head Office in Hong Kong. The Bank had occupied part of the site at no. 1 Queen's Road Central at the centre of Hong Kong's banking and financial district since the mid-19th century, and now instructed G. L. Wilson, a partner of Palmer & Turner, an architectural firm in Hong Kong, 'please build us the best bank in the world'.

Significantly the building, which was opened in October 1935, formed one of the most up-to-date and completely equipped buildings in the Far East (Fig. 1). Although much material and equipment came from England, the whole enterprise was completed in two years under the direction of Messrs. M. H. Logan and L. E. Amps, who were not only the designers of the structure but were also responsible for the management of construction.

There were several innovative features in the new building. Provision was made in the design of the roof to take a helicopter landing pad, the building was air-conditioned, and tubular steel scaffolding was imported from England. Above all, the structural steel frame was designed and constructed in a high tensile steel called *Chromador*, with a guaranteed minimum yield stress about 60% greater than ordinary mild steel. It was one of the first applications of this steel in buildings.

By the late '70s, the Hongkong Bank, as it had become known, had grown into one of the major and most important international banks at a time of Hong Kong's rising status as an international banking and commercial centre, and began considering the redevelopment of its headquarters to meet its increased spatial and symbolic requirements. In 1979 Ove Arup & Partners were engaged as structural engineers, together with a full building professional team, to report on the options and limitations for the redevelopment of the site on which the 1935 building, together with its additions, stood.

The report, which was submitted in February 1979, formed much of the background material for a limited architectural competition held between seven invited firms of architects between June and October 1979. Of the seven architects, two were from each of Australia, England, and the United States, and one from Hong Kong. The Bank was clearly intent on maintaining a distinctive presence on the Queen's Road Central site in keeping with the innovative standards set by its 1935 building. Foster Associates from London was selected. We had collaborated with Foster in formulating his proposals which, while responding to the brief in examining the options for partial or phased redevelopment, proposed a third preferred option of a phased regeneration of the site. The essence of this proposal was the opening up of a series of possibilities which allowed the client to construct either a totally new building or one which retained a variable amount of existing space, including in particular the old banking hall, opening up new options for an orderly development of the site. Technically, this implied a large-span building capable of being constructed over at least part of the 1935 bank. The large span also created the possibility of opening the ground-floor as a public space. By dedicating this area to the public, the plot ratio — i.e. the ratio of the area of the constructed building to the site area — could be raised from 15:1 to 18:1.

The resulting building which is described in this paper has, long before its completion in the latter part of 1985, created interest, speculation, admiration, and, of course, intense controversy. Not the least part of this has been the building's final cost which includes substantial sums for the special facilities required in an international bank's headquarters, together with all of the fit-out elements normally installed by tenants. Controversy makes news, but often obscures the challenge, the excitement, the pleasure in being able to participate in, and contribute to, something special, something which will ultimately be tested only by the client's and the public's appreciation and enjoyment.

Fig. 1
Hongkong Bank 1935 headquarters (Photo: The Hongkong Bank)

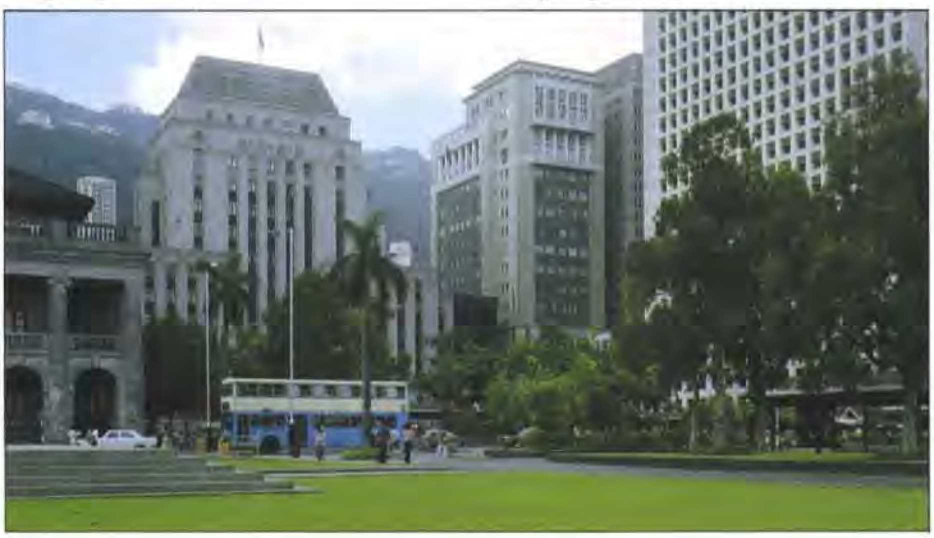
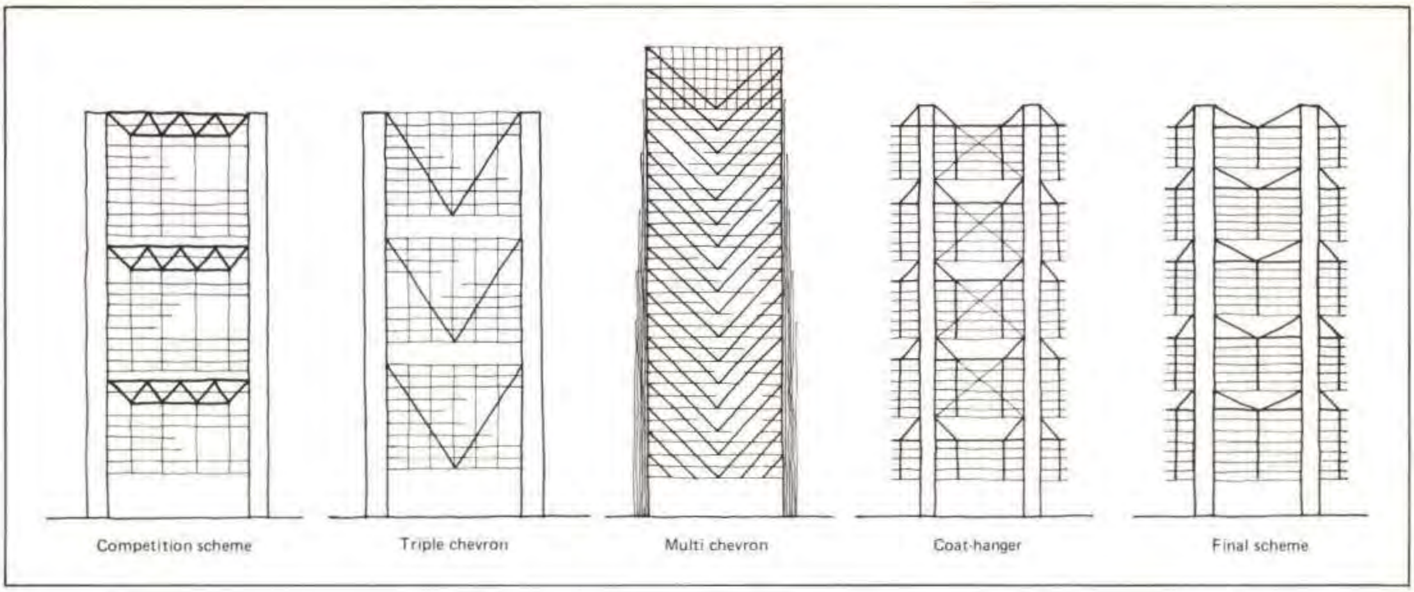


Photo: Ian Lambot



Preliminary concept

The range of options and flexibility inherent in Foster's competition scheme enabled the evolutions of the brief to take place simultaneously with architectural and technical design development.

Between October 1979 and January 1981, when the final concept was approved by the client's board, the scheme evolved through many designs which were synthesized, analyzed, and superseded, after evaluation by client, architect, and engineer, in any combination. The dominant challenge of this period was to maintain a positive response to, and collaboration with, an architect whose conceptual originality is matched only by his infinite attention and care for detail.

Gradually, the Bank's fundamental requirements emerged which pointed to the following elements being provided for in the project:

(a) a basement — to contain security and safe deposit vaults with secure access; loading and unloading facilities for bullion, as well as for the normal stores and equipment of an office building; plantrooms; public space for exhibition and general purposes.

(b) a superstructure — to contain a multi-level banking hall around a central atrium space; headquarters office accommodation, including the specialist departments associated with a major international bank; apartments and executive suites; recreation areas (including facilities for a swimming pool, subsequently omitted); restaurant and kitchen facilities; gardens and terraces at high level; a helipad and viewing gallery at the top of the building.

(c) and overall — a building providing maximum flexibility for changes in operational use and able to respond to the needs and demands of developing banking technology.

During this briefing and preliminary concept period, the client decided to redevelop the site completely. Phased redevelopment was rejected but modest banking facilities were to be established temporarily in the building known as the 'Annexe' occupying the westerly portion of the site.

The structural and civil engineering response to the preliminary architectural concepts included studies for a 180m high tower approximately 70m x 55m in plan, which evolved from the competition proposals through a scheme named the 'chevron' solution to the final design (Fig. 2).

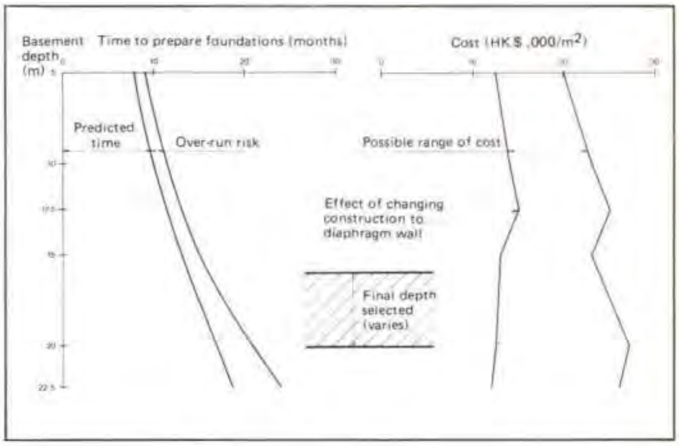
The common thread was to provide as much unobstructed floor space as possible, to prefabricate offsite because of congestion in Hong Kong's central area in general and

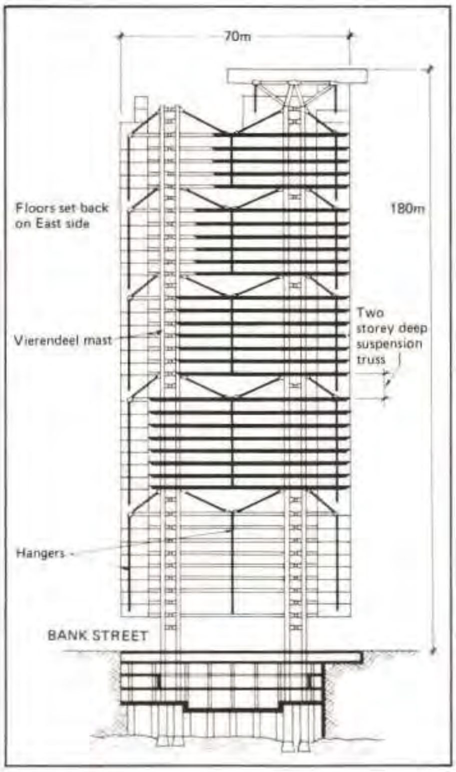
Fig. 2 Above
Scheme development

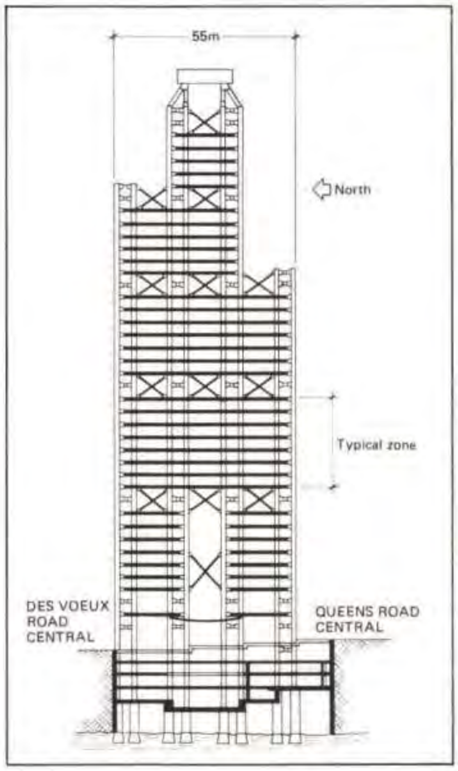
Fig. 3 right
Basement construction risk analysis

Fig. 4 below
East-west section

Fig. 5 below right
North-south section







on the site in particular, and to use the structure as the basis to give the building its distinctive architectural expression.

At the same time, studies relating to the extensive basement involved risk analyses including construction time and cost measured against increased depth (Fig. 3). Ground conditions in the central area of Hong Kong are difficult and not conducive to deep excavations, but the high cost of land, coupled with planning height restrictions, necessitated a multiple basement solution.

The final design emerged as a building comprising a 47-storey steel-framed superstructure of overall height 200m standing 180m above ground, with a four-level basement varying in depth from 16m to 20m (Fig. 4). A total floor area of $100{,}000m^2$ is provided by the structure, this being the maximum allowed by the current planning regulations. The main frame and foundations, however, have been designed to allow for the addition of some 30% extra superstructure floor area in the future, should the regulations change.

The requirements for gardens, terraces, pools and other recreation areas, combined with the service modules and the loading flexibility required by the banking operations, has resulted in the framework being designed for higher loadings than would be associated with more standard tall buildings.

Superstructure

The superstructure is rectangular in plan, approximately 54m x 70m overall, as indicated in Fig. 6. Two rows of four steel masts provide the main vertical structure of the building and extend from lowest basement level to the tops of the three 'slices' to form the cutback geometry (Fig. 5), which itself was a response to the planning regulations.

Two-storey suspension trusses at five discrete levels of the building span 33.6m east-west between the masts and cantilever 10.8m beyond them, dividing the building vertically into five zones (Fig. 4). Tubular hangers are connected to the central and outer nodes, and from the hangers are suspended the floors. With the ground-floor and basement structure being conventionally supported, this suspension structure arrangement creates a large column-free zone at ground-floor level, with all of the superstructure loads being carried by the eight masts.

Planning regulations also required that the massing of the building on the east side be reduced. This was achieved by setting back the floors between the masts on that side, the set-back increasing progressively up the building. The resulting floor plans at various levels are shown in Figs. 7 to 9.

The major potential for future increase in the area of the building lies in these east-side setbacks. The adjacent primary beams, plus the masts and foundations, were designed to carry the load from the total infill of this side of the building, making the structure essentially similar to the west side.

The two-storey spaces occupied by the suspension trusses become the focal point for each bank of floors. They contain the reception, conference, and dining areas, and lead onto open terraces which form both refuge areas in the event of fire and recreation areas in normal times, with some of the best views in Hong Kong as a bonus. The structure has been designed to carry the load from mezzanine floors should these be introduced at a later date.

The layout on all floors is essentially the same, being dominated by the large central open space to provide maximum flexibility in layout and operational use. Services, lifts, and stairs, are located on the east and west sides, with the lifts above level 13 serving only the double-height floors. Travel between these levels is by escalator, resulting in faster average journey times and a greater feeling of 'openness' and communication between floors. The central slice of floor in the lowest superstructure zone is omitted to form an atrium between levels 3 and 13 (Fig. 6), around which are located the public access banking areas with the new double-height Banking Hall being at level 3.

A glazed curved soffit seals the base of the atrium at level 3 affording a clear view into the building above from the plaza floor, which forms an at-grade public thoroughfare linking two of Hong Kong's major pedestrian routes.

An important feature of the atrium is the arrangement of sunscoops, one external and one internal, that reflect daylight into the building and so light the internal atrium space.

Towards the top of the building are the senior executive suites, together with the Chairman's apartment.

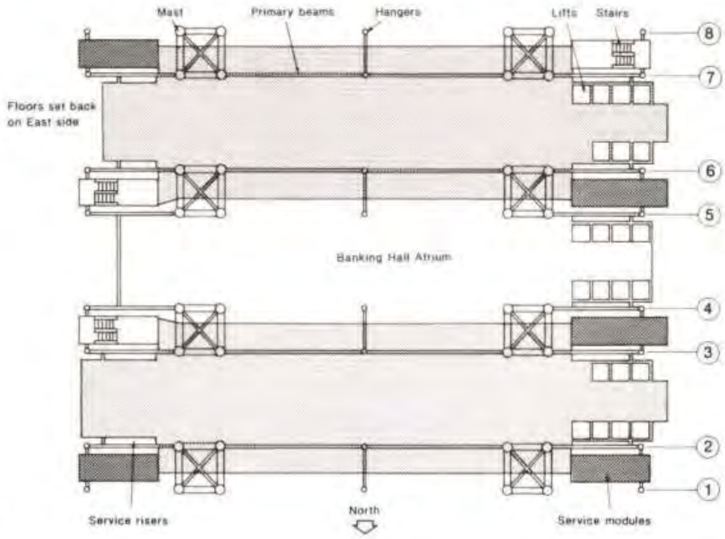


Fig. 6
Levels 3-11
floor plan

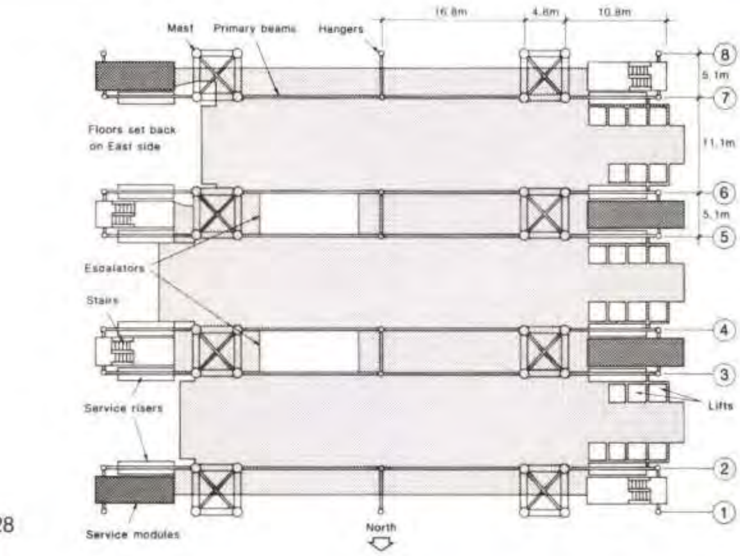


Fig. 7
Levels 13-28
floor plan

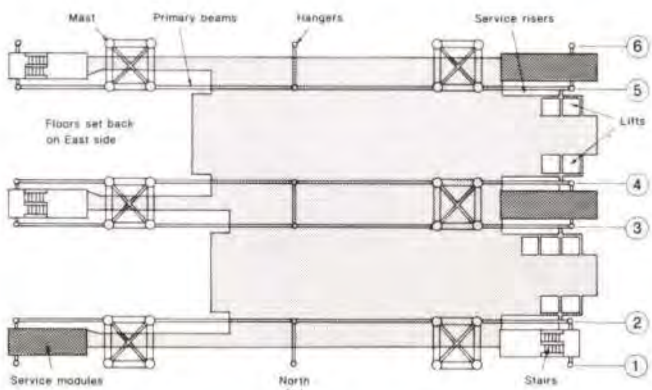


Fig. 8
Levels 30-35
floor plan

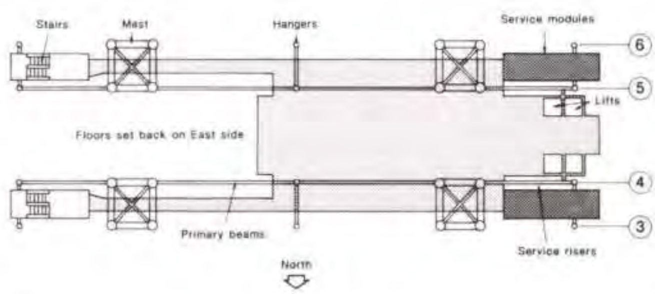


Fig. 9
Levels 37-41
floor plan

Above level 43 the building structure inverts to form the 'top of the building', a group of floors containing dining and executive areas surmounted by a helicopter landing pad.

Substructure

The planning of the basements reflects that of the superstructure, with vertical services distribution, lifts and stairs concentrated on the east and west sides (Figs. 10 and 11). Generally four levels deep, the basements reduce to a single level on the west side because of the retention of the annexe building during the first phase of construction.

The first basement level is essentially a public-use area containing safe deposits, offices, and public and VIP reception areas. The main entrance is via escalators from the east side, while on the west, small vehicle access is provided down a ramp into secure areas for cash and bullion deliveries. The three lower levels house the vaults and the main plant for the building, and access for large vehicles is provided by two 17 tonne lorry hoists on the east side which lead to the loading docks at the lowest level.

The basement structure is built within a 1m-thick perimeter diaphragm wall extending some 25m to rock, grouted at the base to form a watertight seal to minimize ground water drawdown outside the site during subsequent excavation within the wall enclosure. The floor structure within this perimeter is solid reinforced concrete flat slabs spanning onto columns generally on a 7.2m x 8.1m grid. The finishes to the basement are as-struck exposed concrete for floor, soffits, and columns. Internal columns are cased steel stanchions founded on single concrete shafts extending to rock. The foundation to each of the eight main

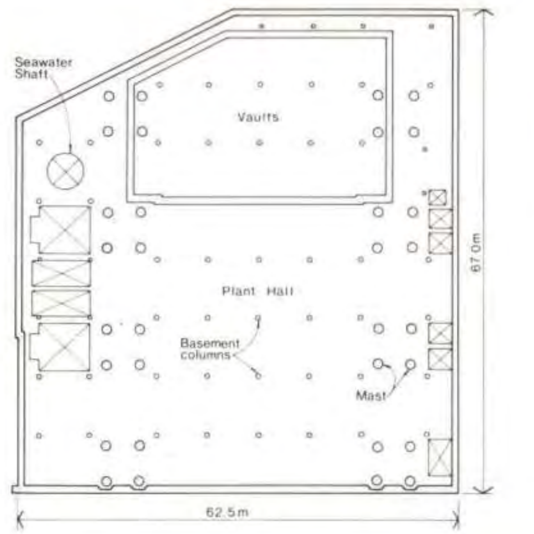


Fig. 10
Basement plan

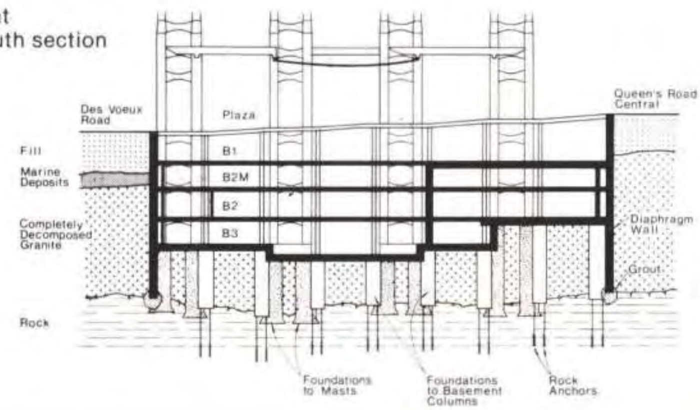
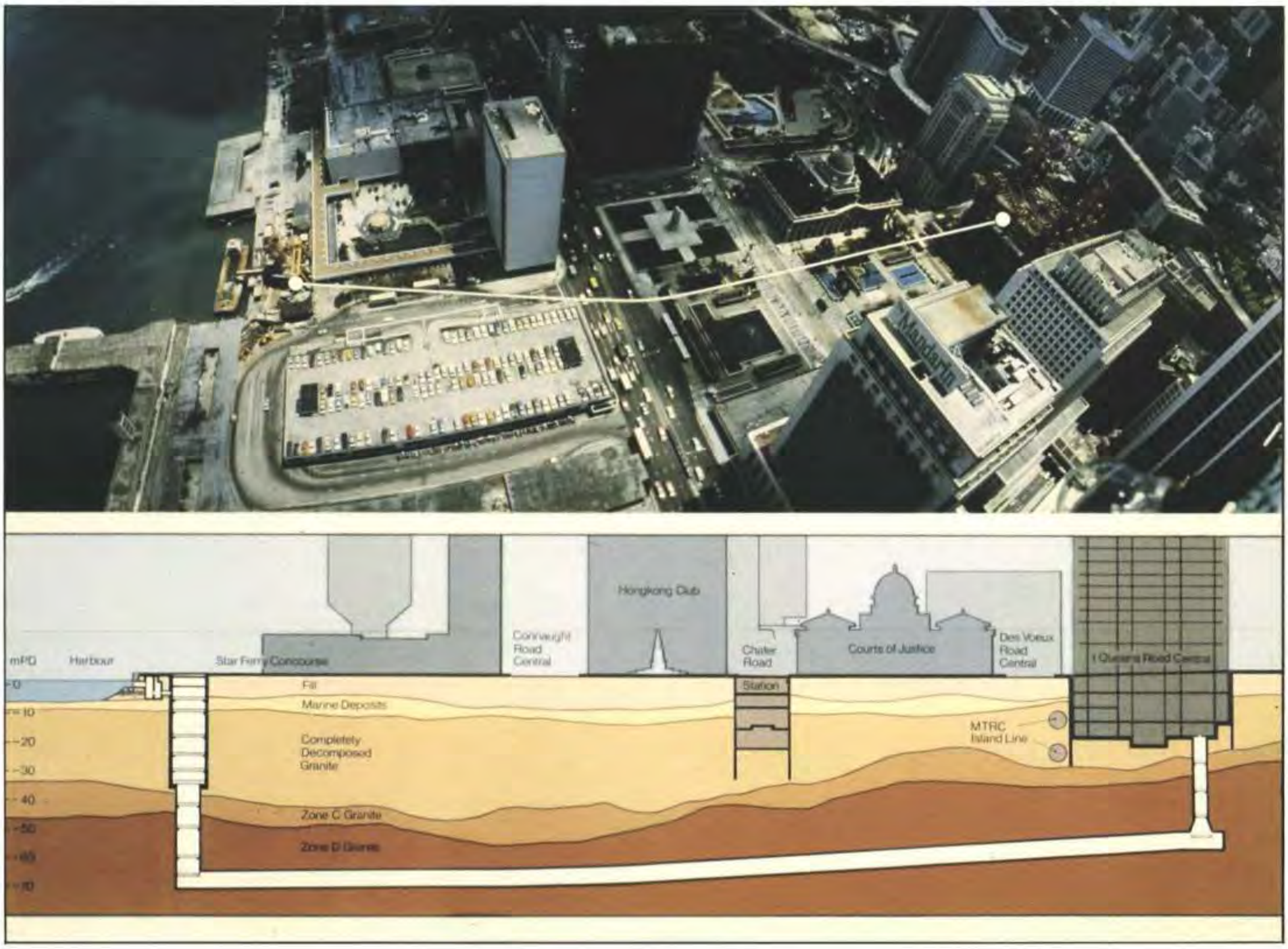


Fig. 11
Basement
north-south section

Fig. 12 below
Seawater tunnel alignment



superstructure masts is formed by a group of four concreted caissons, one under each mast leg, with bell-outs in the granite bedrock.

Seawater tunnel

Located in the south-east corner of the lowest basement level is the entrance to the seawater tunnel. 6m in diameter and driven in the massive granite at a depth of up to 75m, the tunnel contains 700mm diameter pipes bringing seawater from the harbour for the building's air-conditioning and flushing systems.

To bring the seawater the 350m to the Bank from the harbour requires traversing two major roads, a deep underground railway station, and one of Hong Kong's most famous landmarks, Statue Square (which is also one of the few remaining open spaces in the centre of Hong Kong). To avoid the major disruption of surface works in these areas, a tunnelled solution was adopted.

The minimum depth for the tunnel was dictated by the enclosing diaphragm walls to the existing underground station and the future island line underground railway tunnels, as indicated in Fig. 12. However, by a moderate further increase in the depth of the tunnel, it could be driven entirely in massive granite, thus avoiding the construction restrictions of using compressed air and minimizing the excavation supports needed.

The final line of the tunnel is shown in Fig. 12, the depths being determined by the quality of granite and the need to maintain a constant drainage fall.

At the harbour end of the tunnel a circular shaft rises some 75m to the surface. The upper section of shaft located within the completely decomposed granite is 12m in diameter and was formed by diaphragm walling; it houses the filtration and pumping equipment for the system. The lower section of shaft is 8m in diameter.

Intake and outfall pipes connect to the top of the shaft, and water is carried through the system in three 700mm pipes, one flow, one return, and one standby. All parts of the system are similarly equipped with backup units to ensure continuity of operation during maintenance and in the event of breakdown.

The tunnel itself is circular in shape with an unreinforced concrete lining of internal diameter 5m. The system is illustrated in Fig. 13.

Contract organization

The organization of the construction contracts was through an overall management contract, whereby contracts were let for sections of the work, known as packages. This arrangement allowed the tendering of individual contracts on a competitive basis as soon as that portion of the design was complete, thus ensuring minimum delay before execution of the works while retaining the cost benefits of competitive tendering.

As the design for each package progressed, extensive prequalification exercises were carried out to shortlist those contractors best suited to carry out the works. Because of the scale of the works and the desire to obtain the best quality and price available, contractors from around the world were interviewed, resulting in truly international and competitive final tender prices.

The scale of the prequalification and tendering process varied with the scale of the package to be undertaken. For example, the main structural steelwork package involved the initial selection of about 30 international companies, the vast majority in joint venture, followed by a prequalification submission from nine selected joint ventures from which the final tender list of five was selected. By comparison the much smaller package for the diaphragm wall was tendered to three local established and experienced contractors without any selection or prequalification stages.

The packages for the structural works were awarded to a wide range of international companies, the main steel frame being fabricated in the UK, the service modules in Japan, the cladding in the USA, and the substructure being awarded to a Hong Kong subsidiary of a French company. The principal contractors are listed at the end of this paper.

The programming and coordination of construction was carried out by the management contractor, who was appointed on a competitive tender basis early in the project and worked with the design team to ensure that the designs were informed by considerations of programme and 'buildability'. The management contractor was responsible for the overall quality control of contractors' works and for the provision of temporary works, and provided the on-site common user services such as hoists, electrics, security, toilets, etc.

Ground conditions

The site

Most of Central District is built on reclaimed land, and 1 Queen's Road Central is no exception. Historical records show that the original foreshore crossed the site, but reclamation works carried out over the past century have resulted in the harbour now being about 350m to the north (Fig. 14).

Extensive site investigation works were carried out in two phases during 1980 and 1981, consisting of 13 and 36 boreholes, respectively, up to 48m deep. These show a top layer of loose fill up to 7m thick which, to the north of the original shoreline, overlies sandy marine deposits up to 4m thick (Fig. 11). Groundwater was generally 2m to 3m below ground level.

The major soil stratum on the site is the completely decomposed granite (CDG), which

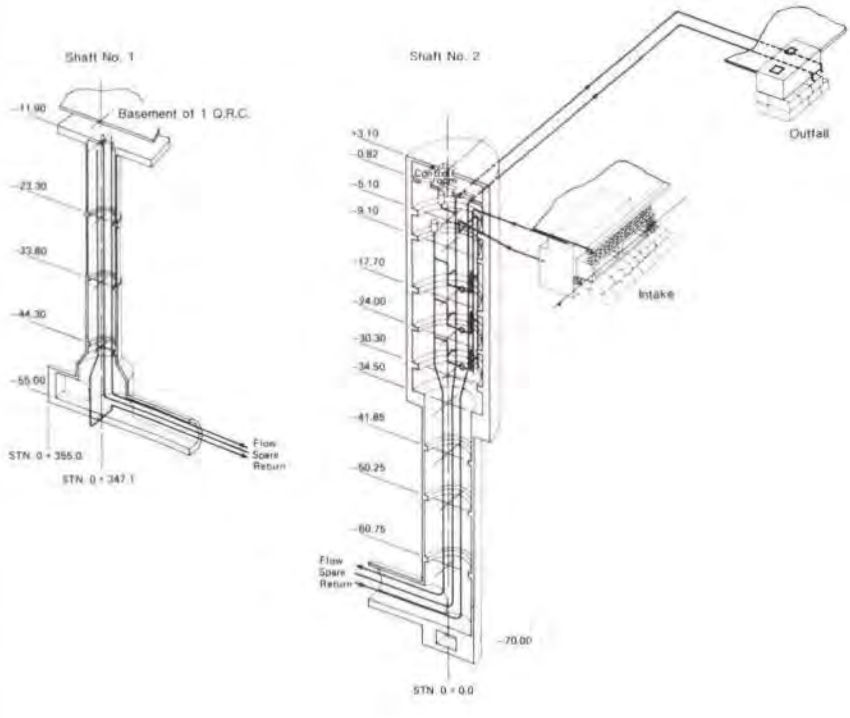


Fig. 13
Seawater tunnel shafts

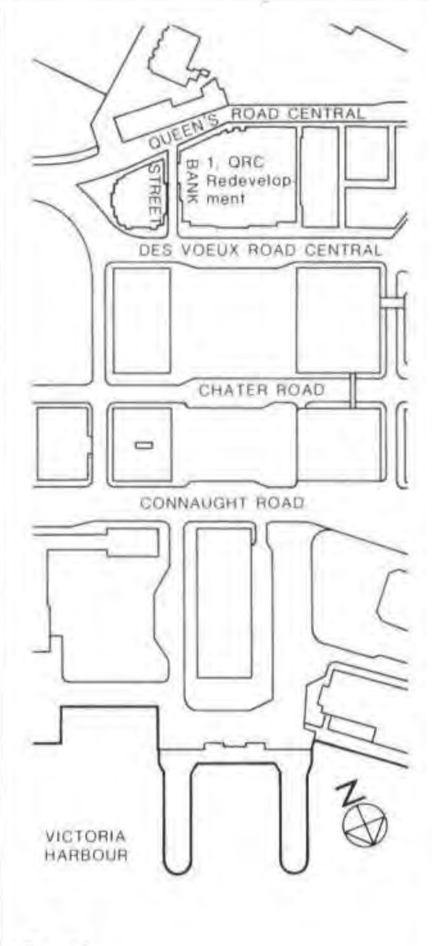


Fig. 14
Site plan

underlies the fill and marine deposits and varies in thickness from 15m to 27m. The more weathered material in the upper 6m to 9m of the stratum is a medium dense silty sand, grading into a dense to very dense gravelly sand as the degree of decomposition decreases with depth. Rockhead level between the CDG and the base coarse-grained granite was found to be highly undulating.

The site is in the centre of an extensively redeveloped area of medium- to high-rise bank and office buildings, and an extensive desk study was undertaken to determine as much as possible about their foundations. A noteworthy building in the immediate vicinity of the site is the Courts of Justice which, constructed *circa* 1900, is one of the few remaining examples of Hong Kong's old colonial style of architecture.

Preliminary assessments

Previous experience in Hong Kong had shown that large ground movements could occur during the process of forming deep excavations in the completely decomposed granite, due both to stress relief on the sides of the excavation and to the lowering of the external groundwater level by dewatering within a site. During construction of a nearby underground station, for example, the Courts of Justice building was observed to settle by up to 180mm. Throughout the planning and design phases it was therefore clear that the overriding constraint on the substructure construction would be to limit ground movements to an acceptable level.

The roads around the site are some of the busiest in Central, and any event which threatened their closure would have considerable repercussions. Major services run beneath the roads, and many of these services are old and in a poor state of repair and therefore susceptible to damage by ground movements.

Adjacent buildings and structures were also a cause of concern. Although some of the buildings are supported on piles, the founding levels are quite shallow compared to the depth of excavation planned, and to the south of the site, on the opposite side of Queen's Road Central, there are old retaining walls which are not in a particularly good state of repair. Large ground movements from the new construction could therefore have caused significant damage in the surrounding area.

Initial studies assessed the use of sheet piles, temporary strutting, and full dewatering as in traditional basement construction. The prediction of large ground movements resulting from this construction approach led to the decision to use a top-down method of construction inside a diaphragm wall 'box' around the site. The wall would provide both a stiff restraint to the sides of the excavation and, by grouting the base, a relatively watertight enclosure to minimize groundwater drawdown outside the site. Then, in addition, by constructing the permanent slabs as the excavation proceeded and subsequently digging underneath, much stiffer lateral supports could be provided to the walls during excavation than would have been feasible with temporary strutting.

Ground movements

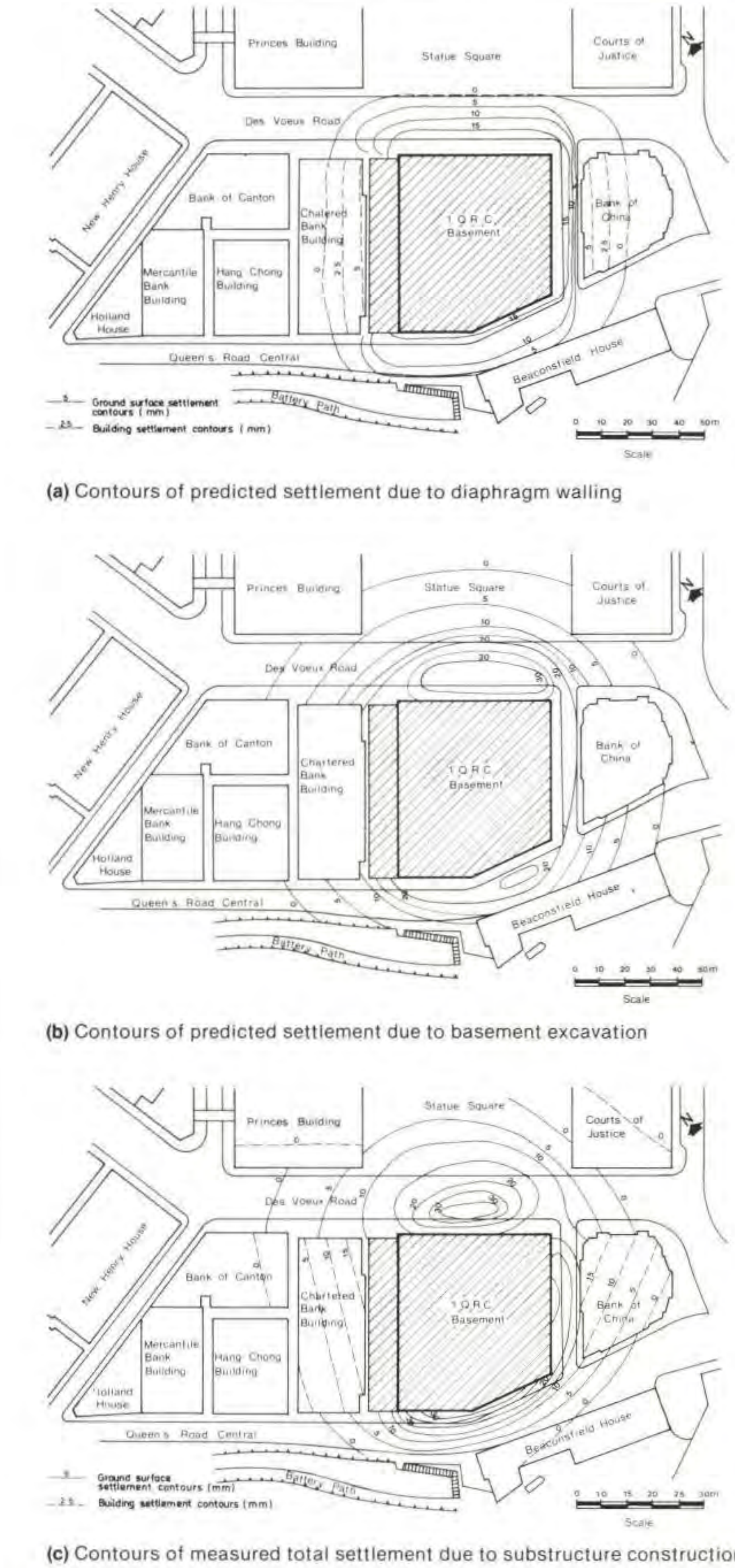
Introduction

The final arrangement of the basements is shown in Figs. 10 and 11. The principal concern of the ground movement analysis was to assess the likely effects of the construction of the deep basements on the adjacent buildings and services and to integrate those findings with the development of construction procedures and controls. The analysis was also used for the design of the diaphragm walls, since the loadings generated during 'top-down' construction cannot be estimated reliably by those established empirical analysis methods associated with open excavations.

Analysis

Ground movements were to be expected from both the installation of the diaphragm wall itself and the subsequent excavation within the box. Both phases of construction would, however, result in some lowering of the external groundwater level, which in itself causes ground settlements.

A flow model was therefore first established using the records from previous nearby excavations and the results of pumping tests carried out during the site investigations. This was used to assess the external drawdown due to dewatering inside a diaphragm wall enclosure, and the effects of grouting the base. This showed that, while



(a) Contours of predicted settlement due to diaphragm walling

(b) Contours of predicted settlement due to basement excavation

(c) Contours of measured total settlement due to substructure construction

Fig. 15
Settlement contours

unacceptably high drawdowns would result from an unsealed wall, satisfactory improvement could be obtained by grouting the base at the interface with rock. By this means it was expected that lowering the groundwater inside the site to rock level would cause not more than 10mm settlement outside the site.

The effects of installation of the diaphragm wall itself were assessed principally from back-analysis of previous projects in the area. From these past projects it was recognized that ground movements were influenced by the size of panel to be excavated, the slurry level in the trench, and the surrounding groundwater level. The method of construction selected had to contain the movements associated with these factors and, at the same time, be practical.

Following a series of laboratory tests, a maximum panel length of 6m was specified, together with a minimum excess head of 3.5m of the slurry level in the trench over the external groundwater. Although this excess head could practicably only be gained by deliberate dewatering of the adjacent ground, the degree to which the groundwater was lowered was the same as was expected to occur during the basement excavation.

Computer analysis of the excavation sequence was carried out using a model which represented the soil by a linear elastic continuum acting between active and passive limits. Soil properties were initially estimated from the site investigation data, and the model was refined by back-analysis of previous projects. Alternative construction proposals were then tested using this model to arrive at the final sequence.

The model was also subsequently used during construction when, following excavation to first basement level, the ground movements measured up to then were back-analyzed. This showed that settlements were somewhat less than had been anticipated, and hence some of the restrictions on excavation could be relaxed.

Effects of construction

Contours of predicted settlement due to diaphragm walling and basement construction are shown in Figs. 15a and b. Greater settlements were predicted to the north of the site because of the softer soils there, arising from it being reclaimed land. Two analysis models were used to represent the different soil properties on the north and south sides of the site, with the predictions for the east and west sides being obtained by interpolation.

Principal concern was, of course, directed at differential settlements; the predictions illustrated in the figures were considered acceptable and were not expected to cause any structural damage to adjacent buildings. Minor cracking was anticipated in the roads, although the services beneath were not expected to suffer damage.

A comprehensive monitoring installation had been placed around the site before construction commenced. This comprised standpipes and piezometers to monitor groundwater levels, inclinometers in the diaphragm walls and in the ground to measure lateral movements, plus building and ground settlement levelling points, totalling some 100 instruments and stations. These were supplemented by the use of previously installed monitoring points, plus the addition of some 50 temporary settlement stations installed by the statutory authorities.

The schedule for the reading of individual instruments varied according to the stage of construction, with, for example, piezometers adjacent to a diaphragm wall panel under excavation being read every 6 hours, while those remote from the site were read twice weekly.

Fig. 15c shows the total settlement contours at the end of basement construction, and it can be seen that settlements were generally less than predicted. Two principal factors accounted for this. Firstly, the settlements due to diaphragm walling were reduced because of the generally very good control exercised over slurry levels and the associated excess head over groundwater, and secondly the soil mass proved to be stiffer than expected, particularly to the south of the site. The grouting to the base of the diaphragm wall also was very effective and reduced considerably the dewatering of the external ground during basement excavation.

Foundations

Description

The eight superstructure masts are restrained laterally at first basement level but are otherwise structurally independent of the suspended basement slabs. Wind load is taken through the first basement slab to the diaphragm walls and thence transferred down to the rock, with the panels acting compositely through shear-linkage provided by the capping beam and slabs. Lateral restraint is also provided at the lowest basement level where the masts are prestressed to caps integral with the lowest basement slab.

Each of the four columns of a mast is founded on a single reinforced concrete shaft extending down to rock, with the shaft diameters of between 2.5m and 3.5m and bell-outs within the rock of up to 4.1m diameter. The rock bearing stress was limited to 5000 kN/m^2. These 'primary' foundations were constructed as caissons dug by hand in the traditional Hong Kong manner, with blasting employed at the base for the bell-outs within the massive granite.

Because of the high loads on the foundations, and since each column has only one shaft, great care was taken to ensure that the rock beneath each founding level was sound. Penetrations below notional rock-head level extended to 7m in some cases before the whole of the bell-out area was considered satisfactory, and three probes were finally drilled below each base to ensure that no weaker layers existed beneath.

Columns for the basement structure were similarly founded on single concreted caissons, although these 'secondary' caissons were straight-shafted and smaller (2.1m diameter) than the primaries and were only keyed into the rock.

Construction sequence

Following completion of the diaphragm wall enclosure, the top-down construction sequence required that the foundations to the internal basement columns be first constructed from ground level so as to allow installation of the columns. These would then support the concrete slabs as they were cast progressively from the first level down, excavation being carried out beneath the constructed slabs.

In parallel with the construction of the basement slabs it was necessary to commence erection of the superstructure. Since the masts extend to the lowest basement level this required the advancement of significant areas of the basement excavation around the masts to allow their installation.

An arrangement consisting of 10m internal diameter access caissons was adopted. The access caissons were sunk around each mast from ground level to the lowest basement, and from their base were then dug the individual caissons to form the mast foundations. Fig. 16 shows the access caissons at the start of excavation.

This sequence resulted in the first mast sections being erected on schedule in January 1983, some 11 months after the start of diaphragm wall excavation.

Rock anchors

The design concept which created the large column-free space at ground-floor level also created a problem in the basements — hydrostatic uplift. With design groundwater level at grade to allow for flood conditions and all of the superstructure weight concentrated in the mast locations, there is simply not enough dead load in the basement structures to resist the water pressures.

An investigation was carried out into the use of an active dewatering system since, with the diaphragm wall cut-off, only relatively small volumes of water would need to be removed to reduce the pressure. In consultation with the client's future building management team, however, it was decided that the possible problems outweighed the potential advantages, and a system of permanent rock anchors was chosen.

The anchors are located in the column foundations and are fully accessible for monitoring and, if necessary, restressing.

Fig. 16
Access caisson excavation (Photo: The Hongkong Bank)

Elements of the superstructure

Masts

The components forming the mast elements are shown in Fig. 17. Each mast comprises four tubular steel columns interconnected by haunched rectangular beams at storey-height intervals of 3.9m to form an overall Vierendeel structure. The masts are 4.8m x 5.1m centre to centre on plan.

The elevational geometry of the haunched beams is constant throughout the height of the building, while the tubulars decrease in diameter up the building height to reflect the decrease in axial and lateral loads. The column diameter remains constant on all masts within a zone and the steel thickness is varied to suit the individual loads.

Tubulars at the base of the building are typically 1.4m diameter with a maximum wall thickness of 100mm, reducing to 800mm in diameter and 40mm in thickness at the top of the building. Flange thicknesses in the haunched beams are generally less than 50mm, but towards the base of the building and within the suspension truss zones, increase to 100mm.

Suspension trusses and hangers

A typical suspension truss arrangement is illustrated in Fig. 18. The truss structures comprise rectangular elements connected to each other and to the masts by pins passing through end clevis plates into large spherical bearings located within thick gusset plates. The horizontal truss elements also form the primary floor beams at the stability levels. Generally, the depth of the inclined elements is 500mm and that of the horizontal elements 900mm.

Gusset plates are generally in excess of 100mm thick and reach a maximum thickness of 175mm. The bearings are maintenance-free radial spherical plain bearings, with the inner ring made from hardened steel with its spherical surface hard-chromium plated. The axially split outer ring is lined with PTFE foil, and seals are fitted in the side faces of the bearings. Over 650 bearings were used, ranging in size from 150mm to 600mm and with a maximum load of just under 20 MN. The pins were manufactured from machined EN19R solid bar, and the largest is 380mm in diameter.

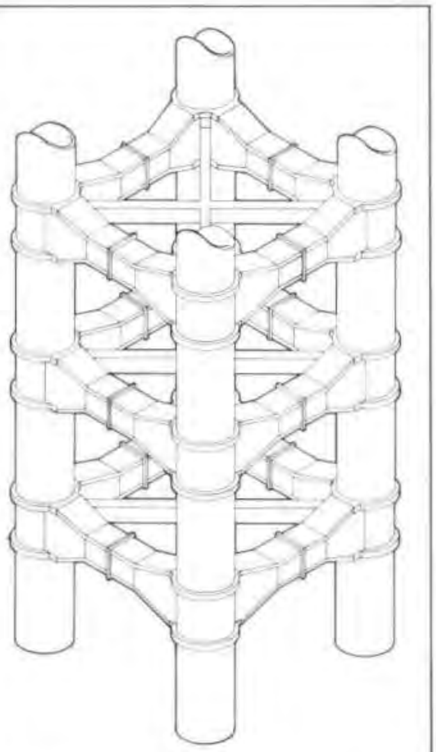


Fig. 17
Mast components

The individual truss elements comprise two thick plates joined by thinner web plates, the thicker plates being placed top and bottom in the horizontal elements for bending stiffness and vertically in the inclined elements to simplify connection detailing.

The main vertical hangers consist of thick-walled tubes ranging in diameter from 200mm to 400mm, with wall thicknesses of up to 60mm. Site connections are made with internal screwed couplers, since general site welding of such members would be impractical and external connections would break the smooth finished lines of the hangers. A central solid coupler engages the threaded ends of the open hanger sections, with opposite threads each end.

North-south cross bracing

North-south stability is provided by two storey-high cross bracing which links the masts together on the east and west sides of the building at the suspension truss levels (Fig. 5). A three storey-high set of cross bracing is located between levels 5 and 8, within the Banking Hall atrium, to cope with the larger lower zone.

The bracing is located on the inner line of mast tubulars only, which, although inducing eccentricity forces and concentrating the loads into single joints, has significant benefits in terms of the spatial impact of the bracing. The form of the bracing is similar to that of the suspension trusses, and bearings are used at all the connections to the masts. The central node of the bracing is a fully continuous connection.

Floor structure

The typical floor structure consists of a 100mm-thick reinforced concrete slab cast in situ on profiled metal decking and spanning 2.4m between secondary beams. Hong Kong regulations do not allow the metal decking to be used as principal slab reinforcement, however, and hence conventional reinforcement was placed in the slabs and the decking effectively used as permanent shuttering only. The slab also forms the horizontal diaphragm link between the masts.

The secondary beams act compositely with the floor slab and typically span 11.1m between primary beams, which in the general office area then span 16.8m between the mast and the central hanger.

Secondary beams are typically rolled section I-beams 400m deep with headed shear studs shop welded to the top flange. Internal primary beams are 900mm-deep welded plate girders with flange thickness varying along the span up to 50mm. The beams are haunched at the mast end to mate with the diaphragms within the columns to which they are welded to form a moment connection. A pinned connection detail forms the joint to the hanger.

All slabs and beams in the office areas are designed for a live load of 5 kN/m^2 to allow storage and filing to be located anywhere on a floor. For assessing the loads on the main frame, however, only 25% of the floor area within a zone (plus certain specific storage areas) is taken at this loading, the remaining 75% having the statutory office live load of 3 kN/m^2. This results in a significant load reduction on the main frame and foundations while still providing maximum flexibility for planning of the office floors.

Analysis of the superstructure frame

Introduction

Verification of the design of the superstructure primary framework required extensive structural analysis which may be divided into three main categories:

(a) Static analysis to demonstrate strength and compliance with the statutory requirements of the Hong Kong Building Code.

(b) Dynamic analysis to predict the performance of the building under wind loading.

(c) Static analysis to predict the cumulative deflections of the framework during the construction process.

The use of database techniques and pre-and post-processors was central to the analysis procedures in order both to speed the production of information and to avoid the introduction of error during the preparation and retrieval of large volumes of information.

Among the forces considered in the design of the building were those due to earthquakes and tsunamis, where the event postulated by the insurance company was a wave 30m high travelling at 30 knots. The location and form of the building, however, is such as to generally protect it from this really quite unlikely event. For earthquakes, although Hong Kong may be considered as a region of low to moderate seismicity, the very high wind loads resulting from typhoons were found to be the dominant design case.

Computer modelling

The analysis system used was an in-house development of the PAFEC finite element program, mounted on Arups' mainframe computer. The system operates around an accessible database containing details of the model and analysis results, and has extensive pre- and post-processing facilities.

The detailed static analysis of the framework was performed on a three-dimensional space frame skeletal finite element model illustrated in Fig. 19. The basic model comprised elements for the masts, suspension trusses and north-south cross bracing for one half of the structure, and contained over 3200 nodes and 3000 elements.

All of the column and beam components of each of the eight masts were modelled explicitly as single elements, with bending and shear stiffness parameters calculated to give the correct overall sway stiffness for the assembly. The procedures for calculating equivalent properties were developed after extensive theoretical and finite element modelling of typical joints to establish the effects of the non-prismatic haunched beam profile and of the large size of the joints relative to the spans. The procedures were automated by development of pre-processor programs to prepare the member property sections of the input data. The stiffness of a typical subframe was later verified by the testing of a full-scale prototype, described in a subsequent section of this paper.

The floors at each level interact in a number of ways with the main frame steelwork. Diaphragm action connects all the masts together for overall movements in the north-south and east-west directions and for torsion about the vertical axis, and locally the floors tend to restrain plan rotations and distortions of the masts. In addition, the concrete plates act compositely with the truss members in axial resistance at upper and lower stability levels.

It was considered impracticable to assess and model precisely the various interactive effects between the steel frame and the concrete floors, particularly as the two sides of the building have, in the setbacks of the upper zones, floor diaphragms of very different degrees of completeness. In addition, the modular ratio is uncertain and, in any case, varies with the duration of loading, e.g. dead and wind loads.

For this reason, simple models of the floor diaphragm were employed which provided reasonable bounds to the effects of the varying diaphragm completeness and stiffness using either beam elements or membrane plate elements. Rigid link methods were eliminated at a very early stage in favour of more realistic element modelling because the rigid links introduced unrealistic force interactions in conjunction with the main north-south cross bracing elements.

The information available in the computer database files enabled the production of calculations for the mast elements to be fully automated. Purpose-written post-processor programs were used to calculate the stresses at critical sections of columns and haunched beams for all the design load combinations and to check those stresses against the Code-allowable stresses. These results were output on calculation pages suitable for submission to the checking authorities.

Static behaviour

The gravity load support system divides into five separate zones vertically through the building. The loads from each are taken by its suspension trusses to the masts, which transmit the load directly to the foundations. In general, the suspension trusses contain a top boom between the masts. However, these top booms are omitted for architectural reasons on the building facade frames and at the top of each mast. This leads to significant differences in the force actions in the trusses at these locations. The masts at the top of the building zones required particularly stiff Vierendeel beams to control the deflections of the trusses.

Ability to increase the floor area of the building at some stage in the future was provided by so designing the frame and foundations as to allow the east-side floor setbacks (Figs. 7 to 9) to be infilled. All design loadcases therefore had to consider both the setback '1985' geometry and the 'filled-in' future building.

This capability for expansion resulted in symmetry of the vertical structure to support the 'filled-in' vertical loads but asymmetry of gravity load in the 'setback' 1985 building. This symmetry of structure and asymmetry of gravity loading cause the structure to deflect laterally during construction and result in the coupling of translation and torsion in the natural modes of vibration. Both these effects were examined in depth and are described in subsequent sections of this paper.

The deflected shapes of the building under equivalent static wind load in the two principal directions are shown in Figs. 20 and 21.

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 top deflection of the structure under the equivalent static wind load specified by the Hong Kong regulations, 1.2 kN/m^2 at ground level up to 4.3 kN/m^2 at a height of 140m and above, is approximately 300mm in both principal directions.

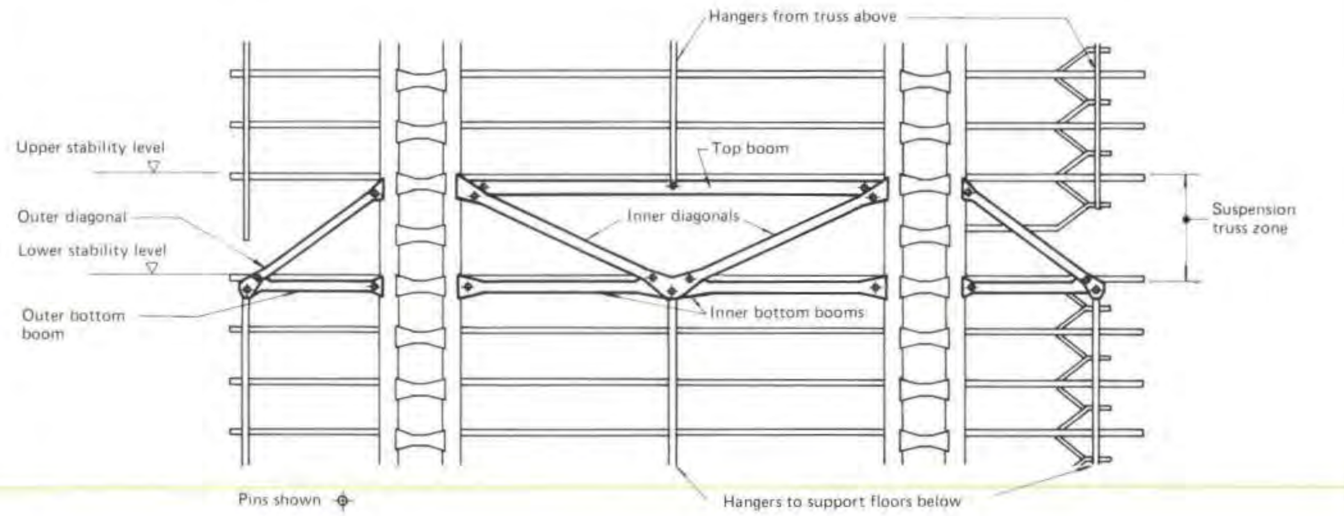


Fig. 18
Suspension truss components

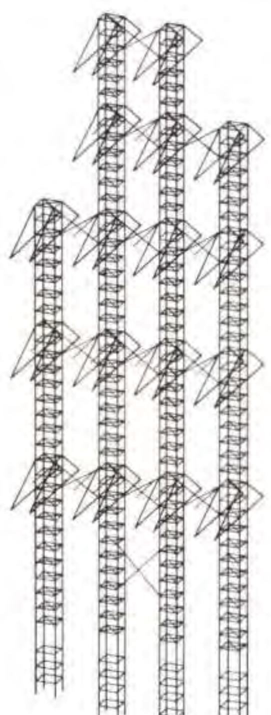


Fig. 19
Superstructure analysis computer model

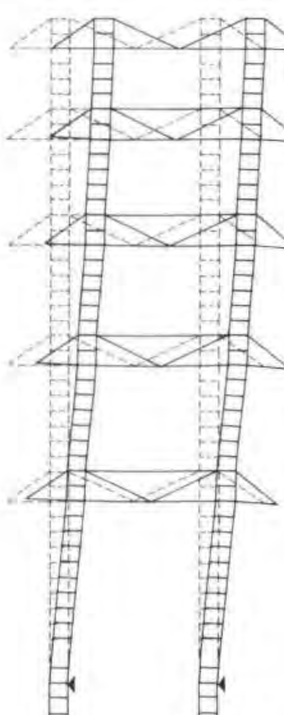


Fig. 20
East-west wind deflection profile

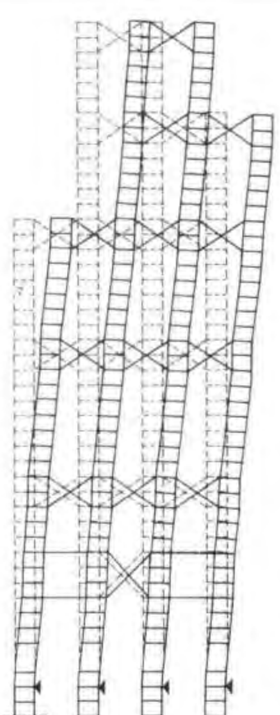


Fig. 21
North-south wind deflection profile

Fig. 22
Wind tunnel test of Waglan Island

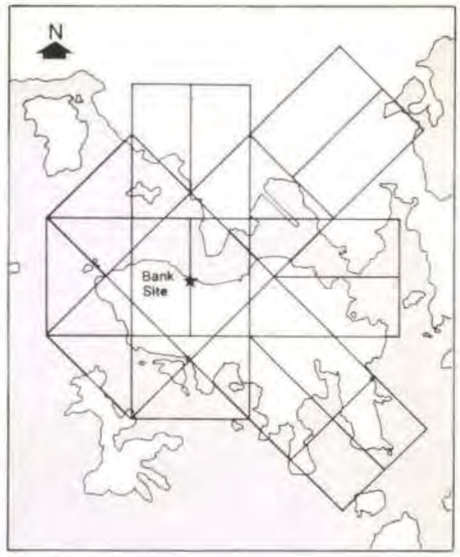
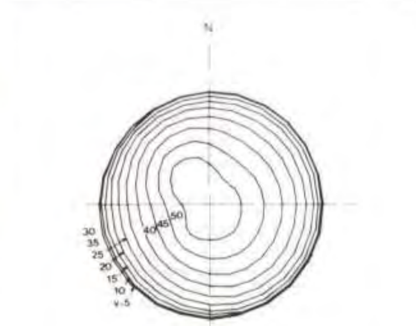
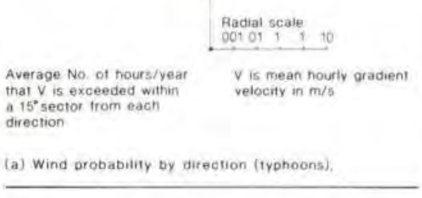


Fig. 24
Layout of topographical model



(a) Wind probability by direction (typhoons).



(b) Wind probability by direction (non-typhoon).

(c) Predicted extreme winds, all directions.

Fig. 23
Wind climate of Hong Kong

Overall stability

In stability terms the structure may be simply visualized as a five-storey unbraced sway frame, the columns of which (i.e. the masts) are themselves unbraced sway frames. The concepts of *BS449*, using effective lengths and permissible stresses for the design of individual column elements, were not considered appropriate to demonstrate overall frame stability for such a structure, and therefore a 'first principles' assessment was made.

The elastic critical load factors for the frame for sway and torsional modes were estimated using the methods of Horne[1] and of Roberts[2], assuming the floors of the building to be infilled to the fullest possible extent. The lowest elastic critical factor was estimated as 8.4 for sway in the east-west direction. It was intended that an amplified sway approach would be adopted for the design of the elements using design wind loads derived from the wind tunnel tests. In the event the statutory lateral loads were considerably larger than the amplified design wind loads, and the final justification was based on the nominal statutory loads.

Construction analysis

It was recognized from an early stage that the movements of the building due to the progressive application of load during construction would be an important factor in the achievement of an accurate final geometry of the building frame. The step-by-step construction of the building was simulated by a series of computer analyses using detailed skeletal models of the main frame at various stages of completion based on the full model described above.

Deflections at all the key points on the masts and suspension trusses were extracted from the analyses for each stage and written to independent databases for subsequent processing by purpose-written programs. The analysis enabled options for correcting the geometry of the frame during construction to be explored.

Details of the construction stages and the use to which the results of this analysis were put are described in subsequent sections.

Consequences of accidental loading

Although the Hong Kong building regulations do not contain any mandatory requirements concerning accidental loading, the consequences of accidental or malicious damage to the structure were investigated. The following aspects were included in the study:

(a) Assessment of the quantities of explosive necessary to remove particular members of the structure

(b) Assessment of the ability of the structure to support overload caused by pressures due to an explosion (key element approach)

(c) Assessment of the ability of the structure to withstand the notional removal of any primary member (alternative load path approach).

The form of the building itself limits the possibility for malicious damage since none of the vertical hangers, potentially the most vulnerable elements, is accessible from ground level and the outer zone hangers are not accessible from inside the building. The only elements accessible at ground level are the masts, and the configuration of four columns connected together strongly at each floor level is particularly robust; one column (and, in many places, two columns) could be removed from a mast without causing collapse.

The key element assessment was based on the accidental pressure of $34\ kN/m^2$ specified in the UK Building Regulations, although it is unlikely that such pressures could develop in the large volume of each floor space in this building. It was established that the vertical hangers could resist the load caused by the pressure acting over the floor area they support at any level.

The notional removal of any single primary element has limited consequences. In the inner zones the removal of a hanger or truss member causes damage which is contained within the zone in question. Alternative load paths were identified in which the loads could be sustained by bending of the floor beams without the need to invoke the additional effects of catenary action. In the outer zones the substantial secondary structural elements, particularly the module and riser frames, provide a range of alternative load paths both across any individual level and vertically to undamaged zones below.

Wind tunnel testing

Background

The wind engineering study for this project is believed to be one of the most extensive ever undertake for a building project. Hong Kong is an area regularly subjected to typhoons, hence wind loading was a crucial factor in the design of both the building frame and the glazing and cladding. In addition, it was recognized from the outset that, for such a tall and prestigious new building, it would be necessary to demonstrate at the design stage that its in-service performance, in terms both of occupant re-action to wind-induced sway and of environmental wind comfort, would be acceptable.

The complexity of the Hong Kong terrain

and the geometry of the building, coupled with the need for the best possible predictions, led to the adoption of wind tunnel testing as a primary tool in the wind engineering study. To this end the Boundary Layer Wind Tunnel Laboratory (BLWTL) of the University of Western Ontario, Canada, under the direction of Professor Alan Davenport, was commissioned to undertake the study. This team has been involved in the design of a large proportion of the tallest buildings built in the last 20 years throughout the world.

Hong Kong wind climate
In order to obtain accurate predictions from the testing, a complete reassessment of the wind climate of Hong Kong was required. Hong Kong is subjected to a mixed population of winds. Typhoons are associated with the strongest winds and dominate the design of structures for strength and safety. However, they occur infrequently (on average three times a year) and non-typhoon winds which occur for the majority of the time are more relevant for the comfort-related aspects of the design.

The evaluation of wind records from different sites in Hong Kong is a difficult task because the complex topography causes substantial local variations in wind speeds, these effects varying with wind direction.

Correlation between different sites was facilitated by the use of a large topographical wind tunnel model of the Hong Kong area.

In view of the variation in surface measurements the climate study attempted to develop gradient height wind climate models for typhoon and non-typhoon winds, where the gradient height is defined as a level sufficiently far above tne surface for topographical changes to have no effect on the wind speed.

The wind records at Waglan Island formed a particularly important part of the assessment. Located some 6 km off the south-east tip of Hong Kong, the island is small, exposed and steep sided, and the anemometer at the top provides reliable measurements for the full range of wind directions. The problem of relating the measurements to gradient height was overcome by modelling the island in the wind tunnel (Fig. 22) in order to obtain corrections for the anemometer reading for each wind direction. The anemometer has been relocated three times since its initial installation and each position was calibrated.

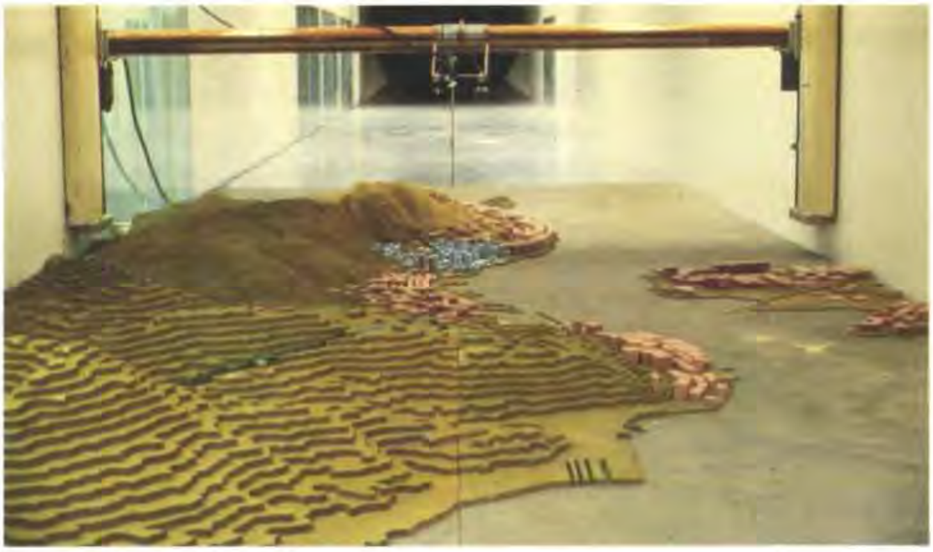


Fig. 25
Topographical model in wind tunnel

Typhoon model
The basis of the climate model for typhoon winds was a 'Monte Carlo' simulation of typhoon occurrences in the South China Sea. This was undertaken to increase the database on which predictions could be made. The method employs a mathematical model of the gradient windspeed field as a function of primary parameters for a typhoon such as central pressure difference, tracking speed, and direction. Statistics relating to these parameters and also to annual occurence rate and minimum approach distance to the site were evaluated from historical records.

The Monte Carlo method synthesizes a large number of typhoon events, the overall statistical properties of which conform to the historical data. The extreme value distribution for typhoon winds is then derived from the simulated windspeed records.

The predictions were supported by extreme value analysis of typhoon windspeeds (suitably corrected) from Waglan Island.

Non-typhoon model
The non-typhoon model was based primarily on upper level data from the Kings Park Observation Station, again supported by records from Waglan Island.

The results of the climate study are illustrated in Fig. 23.

Topographical study
Having established the gradient level wind climate for Hong Kong as a whole, a further study was undertaken to evaluate the effects of the complex topography on wind speeds at the site of the Bank building.

This was accomplished by testing a detailed 1:2500 model of the Hong Kong area in the wind tunnel. The extent of the model is shown in Fig. 24. It was designed such that the central octagon could be realigned in the wind tunnel and the appropriate upstream and downstream sections incorporated for each wind direction, at 45° intervals.

Measurements of the vertical profiles of wind speed and turbulence intensity were made for the central district of Hong Kong and also for other sites of interest, notably the surface anemometer sites (Fig. 25).

From the study a total of five representative upstream wind categories were identified for the Bank site, and these are illustrated in Fig. 26. The strong influence of the surrounding topography in sheltering the site is apparent, particularly for winds from the south coming over the Peak. This is, of course, a direct reflection of the original reason for the founding of Hong Kong, whose sheltered harbour made an ideal resting place for ships to wait out typhoons and was the origin of the name 'Hong Kong', meaning 'fragrant harbour'.

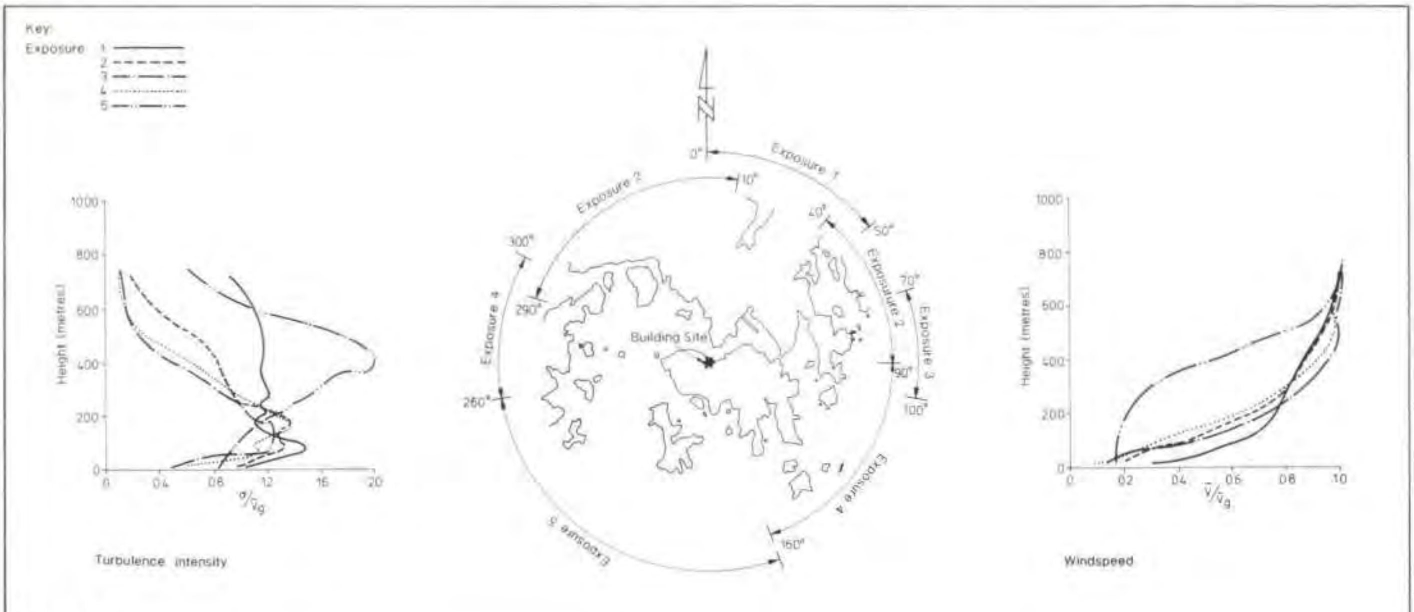


Fig. 26
Central Hong Kong wind profiles

Fig. 27
Proximity model

Fig. 28
Surface pressures model

Proximity model
For the testing of detailed models of the new building it was necessary to simulate on a larger scale the wind profiles over the site which had been established from the topographical study.

For this a 1:500 'proximity model' was constructed containing a detailed representation of all the surrounding buildings and other features within a 600m radius of the site. The proximity model was mounted on a turntable in the wind tunnel and the upstream terrain modelled by 'blocking' as illustrated in Fig. 27. This was varied with wind direction to simulate the appropriate incident wind properties, and particular care was taken to match as closely as possible both the mean velocity profile and the turbulence properties of the wind stream as indicated by the topographical study.

The sensitivity of the measurements on the new building to changes in the local built-up environment was investigated by repeating measurements with hypothetical future developments added to the proximity model.

Surface pressures model
The peak external pressures and suctions on the surface of the building were investigated using a 1:500 scale rigid model instrumented with 520 pressure taps. The model faithfully represents the complex geometry of the full-scale building, and its construction required a very high degree of accuracy, as illustrated in Fig. 28.

This model of the building was tested in the 'proximity model' for the incident winds at 10° azimuth intervals. The peak pressure and suction coefficients found for each direction were integrated with the directional probabilistic wind climate models to predict the 100-year return period pressures. This study showed that the statutory design wind pressures for cladding would be adequate and indeed conservative for this building, even after allowance was made for internal pressures and for the variability and uncertainty inherent in the predictions.

Force balance study
While the rigid pressure model could be used to provide estimates of the mean hourly load on the building as a whole by integration of local pressures over the surface, the uncorrelated nature of local peak pressures prevented such an integration being carried out for overall peak dynamic loads.

An aeroelastic model study was considered, but it was not undertaken because of both time constraints and the difficulty in constructing a model that would represent accurately both the geometry and the dynamic properties of the proposed building. Instead the force balance technique was adopted to measure the fluctuating aerodynamic loads at the base of a rigid model of the building. The dynamic response was then calculated by model methods, a computer model rather than a physical model being used to simulate the dynamic properties of the building.

A light rigid foam model at 1:500 scale provided by the architect was mounted on BLWTL's five-component dynamic base balance at the centre of the proximity model.

Tests were carried out at 10° azimuth intervals to determine the mean-hourly and fluctuating r.m.s. values of the five base aerodynamic force components. Spectra of the fluctuating loads were measured at key azimuths. The variation of base aerodynamic moment with azimuth is illustrated in Fig. 29 and a typical spectrum (as measured, model scale frequency) is shown in Fig. 30. The analysis of these measurements to predict the dynamic performance of the building is described later.

Environmental tests
The investigation of the wind environment around the building became a major area of study. Preliminary experiments had confirmed the likelihood of a strong flow of wind through the open space at the base of the building resulting from the pressure differential between the windward and leeward faces. A number of possible aerodynamic ameliorative measures were investigated by means of a series of wind tunnel tests.

A 1:200 model of the base of the building was constructed, and the flow patterns previously found using the 1:500 pressure-tapped model were simulated at this larger scale. Various arrangements of canopies and partial closures of the plaza area were investigated in an interactive design procedure with the architect. Extensive investigations were also made of the feasibility of jet curtains at the north and south ends of the plaza opening to control the flow of wind below the building.

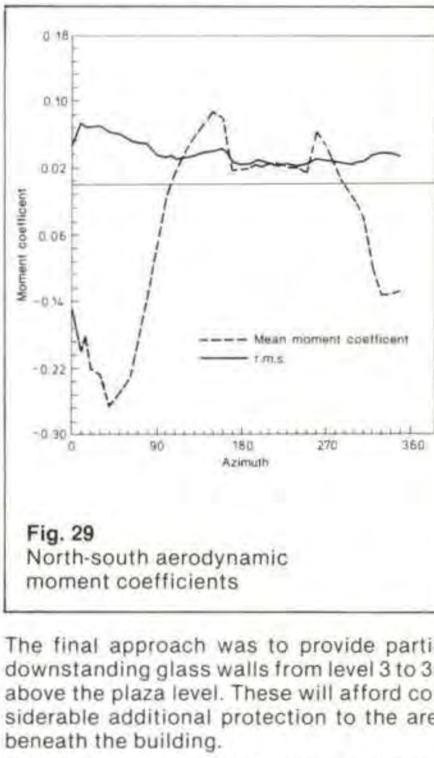


Fig. 29
North-south aerodynamic moment coefficients

The final approach was to provide partial downstanding glass walls from level 3 to 3m above the plaza level. These will afford considerable additional protection to the area beneath the building.

The environmental test results were judged by comparison with wind tunnel tests on the environment around a model of the existing Bank building. The knowledge of the actual conditions that had existed around the building was used to calibrate the wind tunnel test results.

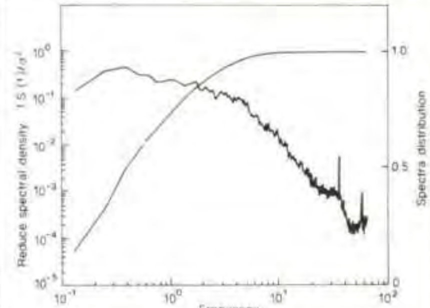


Fig. 30
Base moment spectrum

Dynamic analysis

Modelling

A dynamic analysis was performed to predict the natural period of vibration of the building and the corresponding modal properties for use in the dynamic response calculations. A large skeletal finite element model was considered unnecessarily cumbersome for this purpose and a simpler model was adopted.

The building conveniently divides into five zones, each with a characteristic mass distribution, as can be seen from the elevations (Figs. 4 and 5). The building was therefore idealized to a five lumped mass system, each mass corresponding to one zone and having three degrees of freedom (two horizontal translations plus twist about the vertical axis).

A flexibility matrix for the centres of mass was obtained by analyzing the fully detailed skeletal finite element model, described previously, under a series of unit loads. The magnification of sway by P-δ effects was allowed for by increasing the terms of the matrix globally by the appropriate factor.

The mass matrix was assembled directly and the 15 degree of freedom eigensolution was obtained using a small purpose-written program.

Fig. 31 shows the modeshapes and periods of the fundamental modes obtained in this way. It can be seen that the eccentric mass distribution causes significant coupling between translation and torsion, particularly in modes 2 and 3.

Calculation of dynamic wind responses

The dynamic responses of the building to wind were predicted by a modal analysis in which the wind tunnel force balance measurements were combined with the predicted dynamic properties of the building. The force balance measurements take the form of mean and r.m.s. fluctuating aerodynamic force components of x and y base shear, x and y overturning moment, and torsion, for the full range of wind directions. As-measured spectra, such as illustrated in Fig. 32, were smoothed for noise and corrected for mechanical resonances and electrical filtering characteristics before being used in response predictions.

Calculations to predict the dynamic responses were based on the principle that the mode-generalized forces were proportional to the overturning moment and torque measured in the wind-tunnel tests. For translational modes this holds, provided they are approximately linear in elevational shape, which was reasonably so. The mode-generalized torque is strictly only proportional to the measured torque for a rigid body torsional mode, and this is a less realistic assumption for the building than that of linear elevational modes. However, the assumption is conservative and was sufficiently accurate for the purpose.

The mode-generalized forces so defined were obtained in terms of the mean, r.m.s. fluctuating and spectral components of measured base moments, to determine the mean, background r.m.s., and resonant r.m.s. components of the required responses. The fluctuating contributions from the three fundamental modes were considered to be uncorrelated when combined.

In making these predictions a sensitivity analysis of the changes in response associated with variation in certain key parameters (e.g. damping) was undertaken. This work included the results of research into the measured response of as-built structures subjected to wind, seismic, and nuclear loading. (The analysis procedures are described in more detail in reference 3.)

A computer database was then generated containing deflection and acceleration responses and base loads in mean, background, and resonant components for 15° azimuth intervals and wind speeds at 2 m/s increments. These components were combined to form responses of design interest (e.g. peak base shear force, resultant peak acceleration) and plotted as shown in Fig. 32.

Probabilistic predictions

Having established response/azimuth/wind speed relationships the wind climate models were introduced into the calculations in order to make predictions of the frequency with which chosen response levels were exceeded.

To illustrate the procedure, from Fig. 32 it is possible to establish for each azimuth the value of gradient windspeed V_g required to cause a specific response level to be exceeded, and Fig. 23 shows the number of hours/year that V_g values are exceeded (by azimuth) in the typhoon and non-typhoon models. A further program therefore carried out a 'round the clock' integration for those responses of design interest, thus generating a second database containing the number of hours/year that specific response levels are exceeded.

Data of this form was used as the starting point for assessing peak dynamic loads, fatigue effects, and occupant comfort considerations.

Peak dynamic loads

The results of the directional wind climate integration for peak shear forces and moments were converted to return-period format as illustrated in Fig. 33.

The derivation of design loads from wind tunnel test measurements that are compatible with the level of safety implied by traditional Code of Practice wind load assessments is a complex area and was examined carefully. In particular, decisions had to be made on how the reference or 'characteristic' load effect should be defined and on what partial factor to apply to the chosen characteristic value to achieve an appropriate level of safety and reliability.

In making these decisions due allowance had to be made for:

(a) The variability in the expected load effect due to the inherent variability of the wind climate and the random nature of wind loading

(b) Cumulative uncertainties in response prediction due to uncertainties in the accuracy of:

- wind climate models
- wind tunnel test simulations
- structural properties
- modal analysis

A full discussion of how these were approached is contained in reference 3.

The conclusion of the investigation was that the design loads derived from the wind tunnel test measurements and climate study were significantly lower than the statutory design loads derived from the Hong Kong Wind Code. Unfortunately, the tight design programme and the need to release information for construction did not allow time to negotiate a reduction in the design wind load for the building with the statutory authorities. The building was therefore designed in strength terms to resist the full statutory load.

Certain savings in steel weight in the main frame, where elements had been sized to enhance the lateral stiffness, were, however, possible in the light of the favourable dynamic performance of the building as indicated in the following sections.

Fatigue effects

The fatigue load on the building was estimated by assuming that the overall response of the building could be idealized as a narrow banded random process. Under this approximation, the amplitudes of the dynamic response follow a Rayleigh distribution and the cycling rate is equal to the natural frequency of the building.

The calculation of fatigue load involved the following steps:

(a) Evaluation of the number of hours/year that given levels of r.m.s. fluctuating responses are exceeded, utilizing the integration of the wind climate models with the dynamic responses predicted by modal analysis

(b) Calculation of the amplitudes within each r.m.s. level according to the Rayleigh distribution

(c) Calculation of the number of cycles of response/year for which each amplitude is exceeded.

The fatigue lives of the most critical components of the building frame were estimated, using fracture mechanics techniques, to be of the order of 10,000 years. This long life is not surprising when it is taken into account that:

(a) The structure was designed and detailed to resist the statutory wind loads, which were found to be conservative in this case;

(b) Peak dynamic loads are associated with strong typhoons which occur infrequently; the non-typhoon climate is comparatively mild.

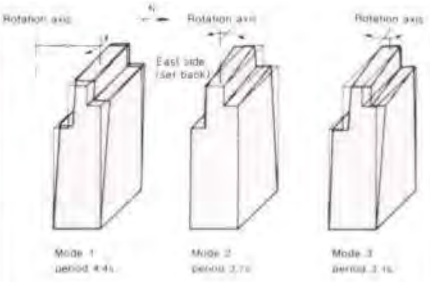


Fig. 31
Fundamental modes of vibration

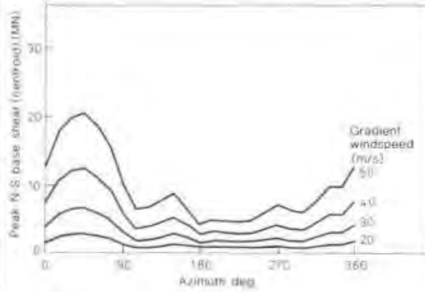


Fig. 32
Predicted peak north-south base shear v. azimuth

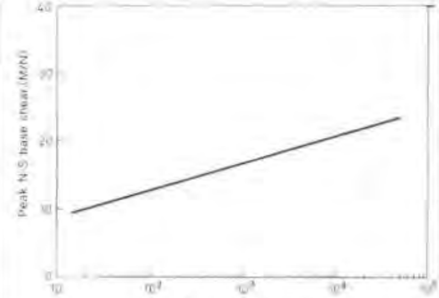


Fig. 33
Predicted peak north-south base shear v. return period

Occupant comfort

The predictions of peak resultant acceleration at the top floor of the building are shown in Fig. 34, together with the acceptance criteria proposed by Davenport[4], the National Building Code of Canada[5], and the draft international standard ISO/DIS 6897[6].

It can be seen that the expected performance of the building under non-typhoon winds is very good and will be satisfactory even in typhoons, when in fact most people leave their offices and such stringent criteria are no longer applicable.

Full-scale monitoring

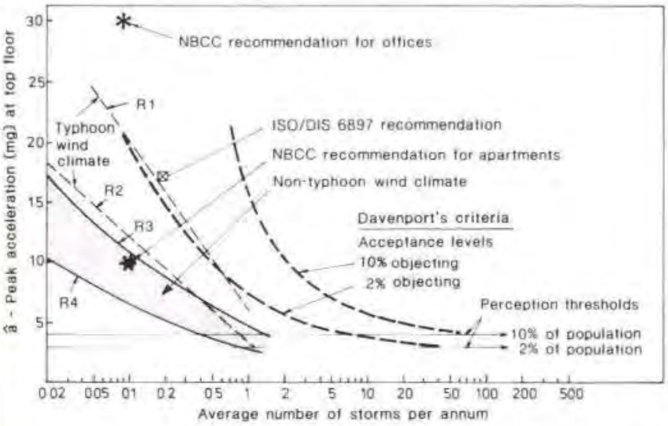
It is intended that the wind responses of the completed building will be monitored. The design of the instrumentation system is currently being finalized; measurement of wind speed and direction, together with accelerations near the top of the building, will be recorded. The results will enable the natural frequencies, damping ratios, and dynamic response of the building, to be measured directly and so add further to the full-scale information that gives confidence in the state-of-the-art methods employed in predictions such as those described here.

Prototype testing

Introduction

The testing to destruction of full-scale prototypes was used to provide the design team and the client with the ultimate assurance of the performance of important elements of the building. This approach also was fundamental to the design process adopted for this project, which made much use of models, mock-ups, and then prototypes of production elements.

The main frame structural steelwork prototypes consisted of four typical key details, illustrated in Fig. 35. The concept was to fabricate the test pieces using the pro-



Responses

Wind climate	Location in Building	
	30m from centroid	Centroid
Typhoon	R1	R2
Non-Typhoon	R3	R4

Fig. 34
Predicted peak accelerations at top of building

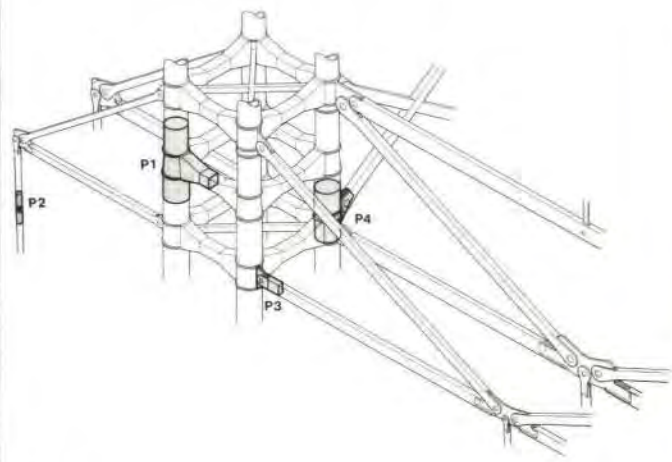


Fig. 35
Location of prototype elements

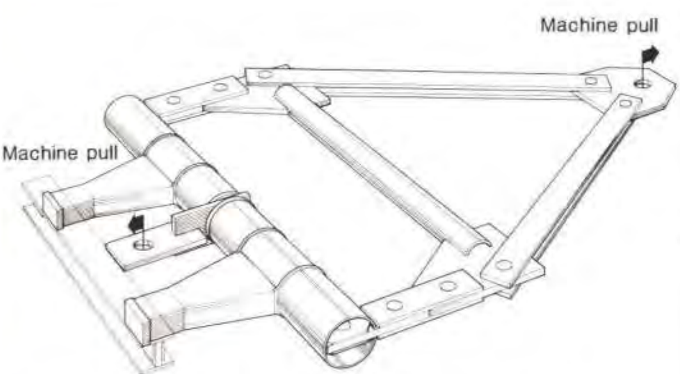
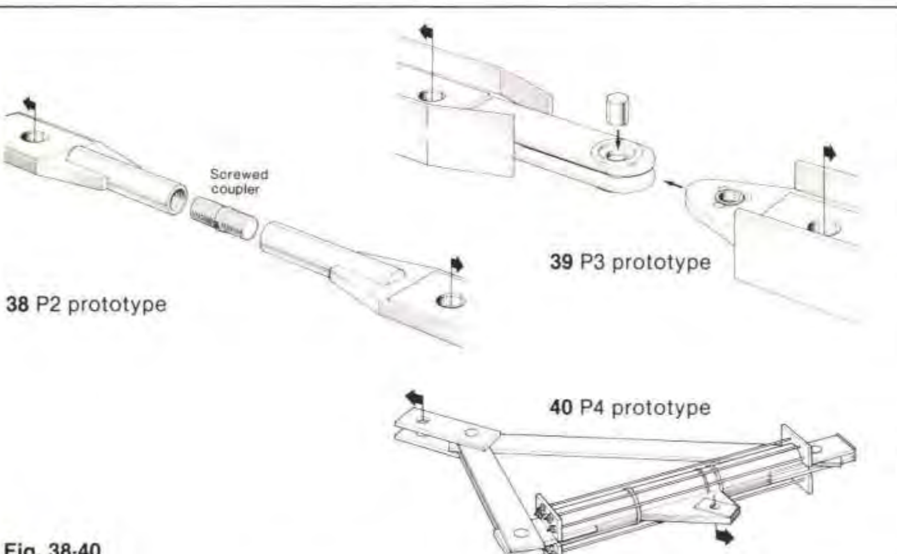


Fig. 36
P1 prototype

Fig. 37 right
P1 prototype failure



38 P2 prototype

39 P3 prototype

40 P4 prototype

Fig. 38-40
Prototypes

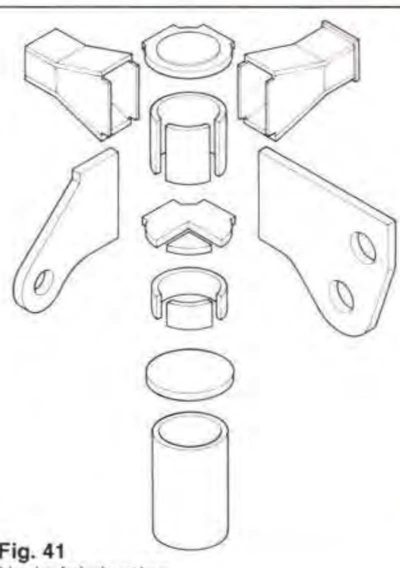


Fig. 41
Node fabrication

cedures set down for the production elements and then to load them as nearly as possible with the expected design forces, firstly in the working load range and then to destruction. Although the tests were intended primarily to confirm final designs, in some instances alternative details were tested and the final design decisions taken in the light of the test results.

Apart from demonstrating the validity of design calculations, the prototypes also made it possible to obtain information on performance and constructability that could not have been obtained easily, if at all, from mathematical analysis. Examples of such information are:

(a) Permanent deformations at working load, which are influenced by residual stresses, tolerances, and fit-up

(b) Serviceability of components after overload

(c) Post-elastic behaviour and ultimate strength, which are difficult to assess for details with a high degree of indeterminacy

(d) Potential problems associated with fabrication and site erection.

The tests were carried out using the British Steel Corporation's 1250T Avery test machine at the Britannia Works, Middlesbrough. This machine was built in the 1920s, originally to test prototypes of elements for the Sydney Harbour Bridge.

The following sections outline the scope and objectives of each main frame prototype test and the results that were of particular interest. All the test pieces were instrumented with strain gauges, deflection transducers, and brittle lacquer coatings, to give detailed measurements for subsequent back-analysis.

While the most important structural prototypes made were of the main frame steelwork, prototypes were also manufactured of the service module structure, the glazed soffit, the prefabricated steel stairs, and the connecting walkways across the atrium, although these were generally not tested to destruction or instrumented as extensively as those of the main frame.

Prototype P1: Vierendeel beam/column subframe

The Vierendeel beam/column subframe of the main masts is one of the primary building blocks of the structure. For this prototype two identical test pieces were fabricated and tested 'back to back' as illustrated in Fig. 36. The loading arrangement simulates the effect of wind shear on the masts.

The pieces were representative of the mast construction at the upper levels of the building; the column was 900mm in diameter x 40mm thick, the beam flanges were 25mm thick and the webs 12mm.

The test sequence started with three cycles of loading to 1.2 times the design working load in order to effect a shakedown of residual stresses. On first loading, yield strains were reached locally well below the working load. However, on the third load cycle, reasonably linear behaviour was observed in all the transducers up to the proof load.

The test was intended to verify the overall elastic sway stiffness of the subframe, and its ultimate strength, and to investigate the local stress concentrations at the column-beam junction.

Moiré fringe photography was used in addition to deflection and strain gauges to estimate the deflections of the subframe. This technique utilizes photographs taken of the surface of the test piece, which is covered with a paper having a fine and regular pattern of dots. Interference fringes occur when photographs taken at different load steps are superimposed; the deflections are calculated from the positions of the fringes. This method proved successful for the haunched beams and the centreline of the column, but it did not prove possible to evaluate the distortions around the curved surface of the column.

The elastic sway stiffness of the subframe was found to be within 5% of the stiffness predicted by the simplified 'equivalent properties' method used for the main frame analysis. A detailed finite element back-analysis was also performed, and this is described in more detail later.

The ultimate strength of the subframe was reached at 2.75 times the design working load. Failure occurred by inelastic buckling of the beam flange induced by the change in direction at the end of the central parallel section, as seen in Fig. 37.

Prototype P2: screwed hanger couplers

With little practical building experience available in the use of large threaded couplers, prototype P2 was set up to examine the performance of two different thread forms (an Acme thread and a Unified thread) and the effects of reducing the engagement length from one diameter to 0.87 diameters. The effects on the connection of non-concentric loading of the hangers was also investigated, together with possible site erection problems, including assessment of the torque required on the coupler to assemble the connection.

Two test pieces were fabricated, each consisting of 250mm diameter, 30mm-thick tubular hangers with their couplers. The arrangement is illustrated in Fig. 38. Detailed pre- and post-test metrology was undertaken, including the making of plaster replicas of the thread forms.

A series of tests was carried out with the two prototypes loaded in axial tension. The conclusions were as follows:

Both thread forms performed satisfactorily and gave a connection that was stronger than the tube sections being connected. Both couplers could be unscrewed and appeared undamaged after testing to the ultimate load of the tubes. The Acme thread was selected for the production elements because of its greater resistance to handling damage.

Reducing the coupler engagement to 0.87D did not cause the connection to fail prematurely.

A nominal eccentricity of loading of 25mm affected local first yield in the tube wall but did not reduce the ultimate axial resistance of the connection.

It was found to be very difficult to turn the coupler to form the connection unless the two hanger components were very accurately aligned. It was found to be much easier to do this with the hangers in the vertical position rather than horizontal, and it was decided to use this orientation for site assembly.

Prototype P3: bearing truss connection

The suspension truss members are connected to the masts and to each other using radial spherical plain bearings as previously described, and prototype P3 was set up to examine the following features:

(a) The overall performance of the spherical bearing and pin

(b) The stiffness of the connection

(c) The degree of permanent deformation developed at working load, which would influence the 'play' in the connection as members underwent reverse cycle loading

(d) Problems likely to be encountered during fabrication and erection.

A typical bottom-boom-to-mast connection was selected for the test, consisting of a central tongue plate containing the bearing connected to the forked end of the boom by a solid pin, as shown in Fig. 39. The tests revealed high strains around the perimeter of the holes in the central and outer plates on first loading due to the bedding-in of the spherical bearing and the pin. The play in the joint was examined at stages by releasing the machine tension and applying a nominal compression load, measuring the relative deflections between the two sides of the joint. The play was found to increase from 0.4mm after 35% of the full design load had been applied, to 0.8mm after 120% had been applied.

The ultimate load was found to be 2.3 times the full design load. At this stage the hole in the central plate had elongated by some 7mm. After the test the spherical bearing was removed and found to have been virtually unaffected by the ultimate load of the connection. The pin had suffered a small amount of permanent deformation.

The high local stresses found on first loading were probably associated with initial out-of-alignment of the holes in the outer plates, which had been noted in the pretest metrology. This was attributed to the fabrication procedure which was therefore changed for production elements so that all welding was completed before the final machining of the holes.

A design detail was also modified. Small retaining plates had been welded to the prototype to retain the spherical bearing within its hole. A bolted detail was adopted in the production elements to prevent the possible introduction of defects, embrittlement, and stress concentrations, due to welding in the area of high strain around the hole.

Prototype P4: north-south cross bracing

The P4 prototype represents a typical connection between a north-south cross brace and a mast. Elements representative of the upper zones of the building were tested. The column was 800mm in diameter and 40mm thick; the thickness of the tongue plate was 100mm and that of the diaphragm 80mm.

The principal intentions were to identify stress concentrations at the structural discontinuities in the detail and to verify, in conjunction with finite element analysis, the force actions assumed in the design of the highly indeterminate welded connections between the massive tongue plate, the three internal diaphragms, and the column tube.

Fig. 41 shows the total components forming this connection and illustrates the complexity of the junction.

In the compression test the column was prestressed using eight Macalloy bars around the outside perimeter. It was only possible to take the compression test to 1.14 times the maximum working load, corresponding to the safe limit for the Macalloy bars. No signs of distress were observed at this load.

The bending test was taken to 2.97 times the maximum working load, this being the maximum safe capacity of the test machine. The test piece was still capable of taking further load but was approaching its ultimate load, as inelastic buckling of the column outside the stiffened tongue plate zone had begun.

Back-analysis

Back-analysis of the prototype tests was carried out as a supplement to the tests in order to examine more fully particular areas to which it had not been possible to gain access on the prototypes with strain gauges.

Detailed finite element models of the P1 and P4 prototype tests were therefore set up to simulate the elastic behaviour in the tests,

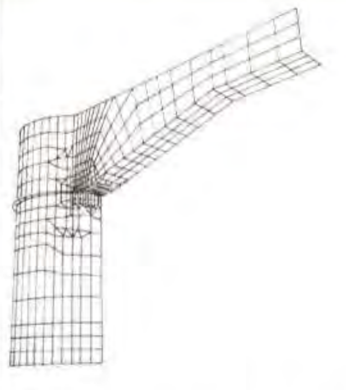


Fig. 42
Finite element mesh for P1 prototype

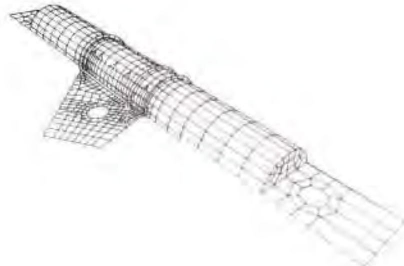


Fig. 43
Finite element mesh for P4 prototype

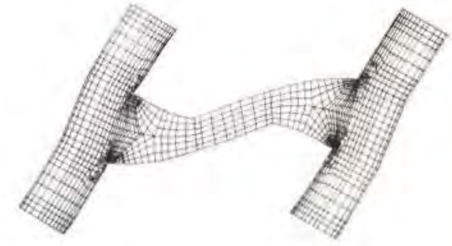


Fig. 44
P1 prototype analysis deflections

giving a complete picture of the stress patterns and deformations which it had been possible to sample at only specific points on the prototypes.

The meshes for the two models are illustrated in Figs. 42 and 43. The models are composed of combinations of eight-noded quadrilateral and six-noded triangular facet shell elements, modelling bending and membrane actions. In areas where the notional span-to-thickness ratio of the plates was small, elements were used which, in addition, modelled through-thickness shear deformation.

The use made of back-analysis is described below for the P1 prototype model.

Overall deformations. The simulated deflections of the P1 prototype test loading are shown in Fig. 44. Analysis of the finite element predictions assisted in resolving differences between predictions for overall sway made from different measurements on the prototype. The finite element model gave very close agreement with the Moiré fringe deformation patterns for the haunched beams and the centreline of the column, and also for the direct diagonal measurements between the tip of the column and the beam. Measurements from transducers mounted on an independent frame agreed less well, and this comparison tended to confirm an observation that the frame had moved during the test.

Local stress concentrations. Fig. 45 shows a section through the junction between the beam flange and the column. The top and bottom fibre principal stresses are plotted as found by the full finite element analysis and by a plane stress sectional analysis. The results from strain gauges at four locations on the prototype are illustrated for comparison. This gave confidence in the finite element representation and enabled the worst stress concentrations in the area to be predicted.

Corrosion protection and fire protection
Requirements
The all-steel superstructure frame of the building required systems for fire and corrosion protection which would satisfy the client's requirements for a 50-year design life, including those elements of the building that are effectively external, while being suitable for application on complex member geometries and compatible with the site erection procedures. Although the building is fully air-conditioned, significant intentional down-time, e.g. at weekends, was also to be allowed for in the design.

Fire-rating was set at 2 hours for all primary superstructure, with a 1 hour rating on the staircases. Statutory regulations required only a 2 hour floor fire-rating in the superstructure for those floors immediately above and below the suspension trusses, with a 1 hour rating on intermediate floors. The client, however, decided to extend the 2 hour rating to all floors both for insurance advantages and to allow total flexibility in the location of fire-rated document storage rooms. Fire-rating in the basements is 4 hours.

The building is fully sprinklered with extensive fire detection systems, and the glazed staircases are provided with an external drencher system. Halon protection is also provided in some areas.

Corrosion protection: external system
Extensive research showed that paint-based corrosion protection schemes could not be relied on to provide the required longevity for the external areas of the main frame steelwork, in a situation where inspection generally could not be carried out and maintenance would be extremely difficult. This led to the adoption, in principle, of a cementitious-based protection system.

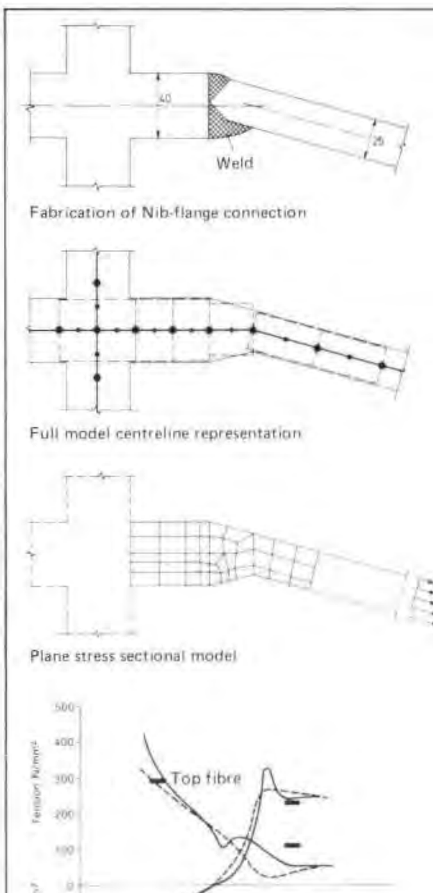


Fig. 45
P1 prototype stress concentration study

Although a traditional coating of 50mm of concrete was considered suitable in corrosion protection terms, it presented severe problems in terms of weight, dimensions, and application to the shapes involved. An alternative was therefore developed by the addition of a polymer, styrene butadiene rubber, to an otherwise normal sand-cement mix. This material, which became known on the project as CBC (cementitious barrier coat), has excellent water vapour permeability properties, such that a 12mm layer ensures corrosion protection equivalent to that provided by 50mm of concrete.

For ease of application on the varied shapes, and to ensure good compaction, the material was gun-applied in a similar manner to traditional gunite. Comprehensive test programmes were carried out both to confirm the performance of the material and to establish application procedures in conjunction with the contractor. The system finally adopted was to blast clean the steel before application, followed by a high-pressure water wash to remove salt contamination. CBC was then applied in two 6mm layers, with 12mm-long melt-extract stainless steel fibre reinforcement (5% by weight) in the lower layer.

Application was generally in two phases, with the bulk of the material applied in an off-site yard, leaving areas around site joints, temporary fixings, etc., to be applied on site. Handleability of the material proved to be good, with only small areas of damage due to handling of the large mast elements.

In the early stages of the site work, rather more was actually applied on site than had been intended, partly because of difficulties in the start-up of off-site application and also because of the areas of leave-offs required for subsequent attachments and temporary fixings being greater than had been anticipated. This caused problems in regard to both operations and programming. The situation improved considerably as the subcontractor became more familiar with the system at higher levels of the building.

To achieve a level of protection at the pinned joints equivalent to the external scheme, a system was developed with the contractor, consisting of a proprietary rubber gasket with an outer silicone seal. Thus protected, the bearings themselves are maintenance-free.

Corrosion protection: internal system
The system of CBC was originally intended for use on the external steelwork only. A paint-based scheme had been specified for the internal elements of the main frame and galvanizing for the internal floor steelwork.

Once the facilities to apply the CBC had been established, however, it became cost-effective to use the same system on the internal as well as on the external elements of the main frame.

In such instances the use of CBC represented an overspecification, and for some internal main frame elements, where other considerations prevailed (principally, hangers and truss members), a paint-based scheme was adopted. Considerable advantage could here be gained by blast cleaning the steelwork off-site but not applying the paint until after erection. Products suitable for application over lightly rusted, blast cleaned surfaces were therefore investigated, and an aluminium-filled epoxy paint was finally selected. The elements were coated with a basic primer off-site after blasting and subsequently hand-prepared by wire brushing and water washing on site prior to application.

The floor beams, both primary and secondary, were galvanized and chromated in the UK prior to shipment. In practice, because of the high silicon content of the base metal, coatings well in excess of the minimum thickness specified were achieved, at the expense of slight problems with brittleness of the coating. This required a higher than ideal level of touch-up, which was carried out using a zinc-rich chlorinated rubber paint.

The chromating treatment was both to minimize the risk of white rusting during transit and to allow for the occasional areas of overlap, where the external scheme of CBC was to be applied over the galvanizing. This occurred where part of a member was internal and part external, it not having been practicable to galvanize only a portion of a member. The joints between the galvanized beams were made with sheradized friction grip bolts.

Away from the main steelwork the same basic requirements for corrosion protection applied, but the actual schemes used varied considerably because of both the experience and capabilities of the individual contractors and the types, locations, and visual requirements of the elements. Galvanizing or flame-sprayed zinc formed the basis of most schemes, sometimes overcoated with further paints to achieve the required level of protection and aesthetic finish.

Fire protection

Fire protection to the main frame elements is provided by a ceramic fibre blanket fixed to a stainless steel mesh wrapped around the member. The system was adopted because of its great flexibility in application, enabling even the most awkward geometrics to be covered easily. The system was tested to *BS476: Part 8* in a full-scale test on a section of mast, including the external corrosion protection layer of CBC.

This fire protection system was used on all elements to which the external corrosion protection system had been applied. Internal floor steelwork was fire-protected using a proprietary board system, and a sprayed vermiculite system was applied to the underside of the floor slabs to provide the 2 hour rating.

Intumescent coatings were used on some visually exposed elements, notably staircases and mullions, where their final appearance was preferred and the sizes and locations of the members made a board or sprayed solution impracticable. 4mm and 6mm thicknesses were used for 1 hour protection, and in a few locations a 13mm layer was used to give a 2 hour rating. A fire test was also carried out on a stair to assess both the performance of the intumescent coatings and the inherent fire rating of the structural arrangement.

The reinforced concrete substructure slabs were so detailed as to achieve the required 4 hour rating, while the fire protection to the basement steel columns was provided by the precast concrete casing.

Ancillary structures

Glazed soffit to level 3

The Banking Hall atrium is sealed at its base at level 3 by a curved clear-glazed soffit spanning 21m across the central bay of the building (Fig. 46). The structure consists of steel cable-stayed catenary beams at 2.4m centres, with the typical section being a 250mm-deep fabricated 'V' (Fig. 47). The glazing lies on the catenary surface and consists of two sheets of clear glass in aluminium frames spanning between the beams, the top sheet being 10mm thick toughened and the lower 12mm thick laminated.

The catenary action is broken by the penetrations of the escalators which pass through the glazed soffit to form the main entrance to the Banking Hall, and here the loads are carried by trimmer beams. These use the typical curved member as a bottom chord, with a horizontal twin-boom Vierendeel top chord and vertical bracing in between.

After fabrication, the first catenary beam was erected with its cable stays at the works and fully loaded, and its deflected profile was then accurately measured. The stiffer trimmer beams at the escalators, which are simple spanning members, were fabricated to meet this deflected profile so as to ensure that the installed soffit had a regular shape with no discontinuities.

Fabrication of the structure and glazing frames was carried out in Austria, and the beams were delivered in three sections and joined on site using internal connectors to maintain their smooth profile.

Service modules and risers

One of the fundamental concepts for the building was for as many components as possible to be prefabricated so as to take advantage of factory control techniques and to speed construction. One of the most complete prefabrication contracts was for the service modules, which contain all of the on-floor air-handling plant and toilets in the superstructure. The modules were fabricated and completely fitted out in Japan, and full plant testing and commissioning were completed before delivery of the units to site.

A total of 139 modules is provided in the building; four modules service each floor at the lower levels, reducing to two at the upper levels where the building plan reduces (Figs. 7 to 9). The modules are independent structural boxes spanning between connections to the outer hangers and inboard floor beam supports, and each module measures 9.6m or 12m long by 3.6m wide by 3.9m high. Their design required full spatial integration of architectural planning, services routes, plant equipment and access, together with coordination of the structural support frame, external wall, acoustic lining, and fire

Fig. 46
Glazed soffit to level 3 (Photo: The Hongkong Bank)

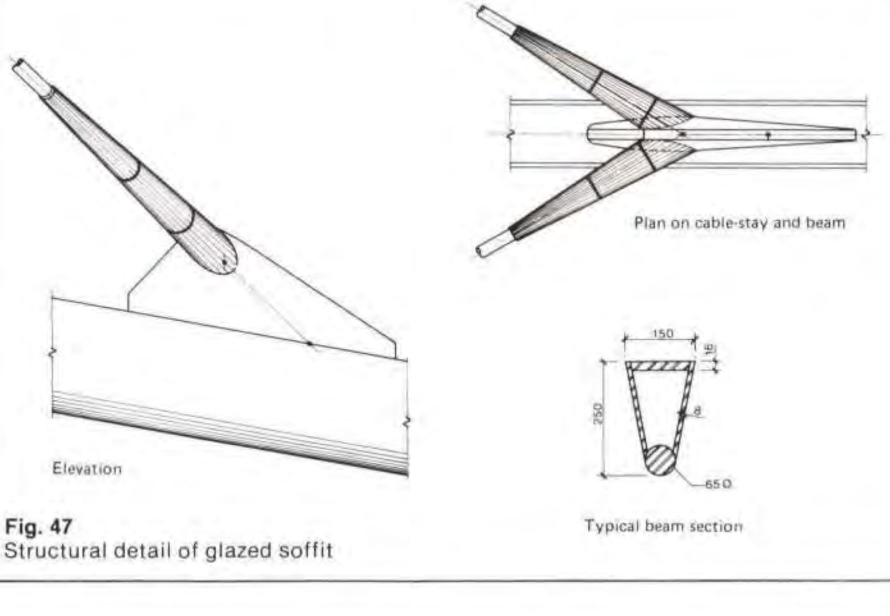


Fig. 47
Structural detail of glazed soffit

Photo: Ian Lambot

protection. The detail design was developed by the contractor in conjunction with the design team using model studies, full size mock-ups, and prototypes.

Initial design solutions for the module structure included stressed skin boxes. These were found to have severe drawbacks, however, including difficulties in fire protection and in forming the many penetrations required. The final structural solution adopted was a simple steel trussed box with lightweight steel deck floors.

The installation weights of the modules were generally between 30 and 40 tonnes, although special modules at the top of the building containing the boilers and standby generators had installation weights of up to 50 tonnes. Road transport restrictions limited delivery of the modules to the site to night hours, and the construction programme allowed only a 1 hour period of craneage hook time to lift each module from the low-loader to its final position on the building. The modules were lowered onto levelling jacks supported on the module below to enable final adjustments and connections to be made after releasing the module from the crane.

Included in the modules contract was the prefabrication of the vertical service risers for the building. These are located adjacent to the modules and the stairs and contain all of the vertical services distribution for the building. The services were prefabricated inside steel frames two and three storeys high, thus minimizing the number of connections on site. A total of some 8km of riser frames was installed in this way.

Cladding and curtain walling

Visual expression of the building's structure is a fundamental architectural consideration. Early studies included extensive investigations of various methods of providing corrosion and fire protection to fully exposed external steel members which would then remain unclad. Possible corrosion protection solutions included the use of weathering steels and corrosion-resistant steels. The use of molecularly bonded stainless-clad steel was also investigated in detail, but there were significant technical difficulties in the material development. Fire protection studies for these schemes included intumescent coatings, flame shielding, and water filling. It was concluded at that time that such total exposure of a large structure would be impractical given short timescales available, and the solution was adopted whereby the structure would be fully expressed, but clad.

The design of the various cladding and curtain walling systems is complex. in addition to the stringent visual criteria, the varying plan geometry of the building and the exoskeletal structure create a great number of different details and many penetrations through the curtain walling.

Having established the criteria for appearance, form, and structural performance, the design of the curtain walling systems was developed through the evaluation of models, mock-ups, and prototypes, with the design team working in close cooperation with the contractor in Hong Kong, London, and at the fabrication works in the USA.

The typical curtain wall consists of storey-height 12mm-thick fully tempered clear glass between vertical aluminium mullions. The mullions are suspended from each floor via sand-cast aluminium brackets, with a vertical movement joint at the junction to the mullion below. The typical mullion is stiffened by the addition of a Vierendeel truss to span the double-height floor spaces.

The cladding to structural members comprises 6mm-thick aluminium sheets with extruded aluminium edge sections and stiffeners which are plug-welded to the sheet. The cladding panels are fitted to the structure by bolting to prefixed stainless steel channel sections, which are fixed onto stainless steel studs welded onto the steelwork. A layer of cement-impregnated tape is located between the channel and the carbon steel structure and a silicone seal applied around the heads of the connecting studs to prevent bimetallic corrosion.

The service modules and risers are clad in aluminium, laminated honeycomb panels. All of the external and visible aluminium surfaces received a sprayed, pigmented (onyx grey), fluoropolymer finish, applied at the works before shipment, which gives the building its final appearance.

The stair shafts are fully glazed using structural silicone sealant to secure the glass to the internal mullions. All such structural silicone was applied in controlled environmental conditions. Particular attention was paid to surface preparation and cleaning, and regular sample adhesion testing was carried out.

Steelwork fabrication

Preproduction

The fabrication and erection of the whole of the superstructure main frame and the concrete floors was let as one contract, which also included provision of the main cranes for the site.

Before fabrication commenced, detailed assessments were made with the steelwork contractor of alternative details and fabrication and erection procedures, to ensure that as fast a construction programme as possible could be realized commensurate with maintaining the required high quality standards. Every effort was made to maximize prefabrication so as to obtain the benefits of fabrication shop conditions in both quality and dimensional accuracy; within the constraints of handling weight and shape the elements to be erected on site would be as large as possible.

The organization of the total project into a series of separate construction 'packages', e.g. steelwork, cladding, ceilings, etc., required that each section of the work be erected to tolerances compatible with the fixing adjustment being provided in the follow-on contractors' works. The dimensional acceptability of the main frame and floors was therefore strictly specified in terms of interface tolerances at handover to other trades. Before finalizing construction details the steelwork contractor then undertook an extensive review of the proposed fabrication processes and erection sequence to determine the elemental tolerances necessary to ensure that the erected structure would be within the specified overall tolerances.

Materials

The principal steel type used in the masts and trusses was equivalent to *BS4360* grade 50D with high fracture toughness properties. A modified grade 50D steel with good through-thickness tensile properties was used where details resulted in stresses in that direction, and grade 43D was used in elements where stiffness, rather than strength, determined member properties.

Crack-tip opening displacement (CTOD) testing was used to demonstrate toughness characteristics initially, and Charpy V-notch testing subsequently employed as a control test during production.

Steel in floor beams was generally grade 50C.

Fabrication

Masts were fabricated as four separate elements typically 7.8m high, each element consisting of a column section (known as a 'tubular') with four half-beams attached as illustrated in Fig. 48. The maximum weight of these elements at the base of the building was approximately 46 tonnes.

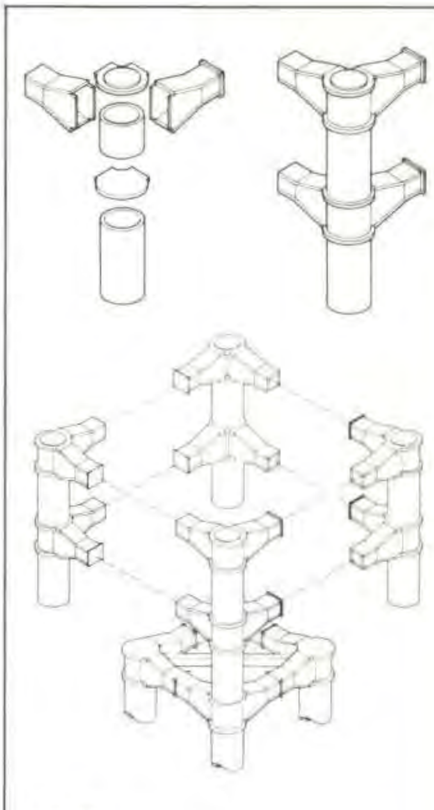


Fig. 48
Mast fabrication

The tubulars were fabricated from open-ended 'cans' welded to diaphragms which coincided with the flanges of the haunched beams. This arrangement enabled the accurate assembly of cans that could vary in wall thickness and ovality, and provided a means of welding the beam flanges to the column without inducing through-thickness stresses in the can wall. On site the top diaphragm provided a positive platform for accurately locating the next element.

The majority of can elements were formed from plate using cold or hot rolling, although a number of the thick-walled, smaller diameter cans were formed by the back extrusion technique. In order to ensure that no detrimental strain-ageing effects were developed during cold rolling, a thorough investigation was undertaken into the possible metallurgical behaviour. All shop welding of longitudinal and circumferential welds was carried out using the submerged arc process.

Although it had been originally expected that the flanges of the beams would be formed from plate welded at the change in direction between the sloping and horizontal surfaces, in the event it proved possible to form all the flanges by cold bending. An elaborate jigging arrangement was developed to ensure accurate fitup of the beams onto the tubulars for welding, and after welding, extensive checks were performed to determine the degree of machining required on the ends of the tubulars to achieve the overall tolerances.

Possibly the most complex fabrications in the whole of the superstructure frame were the truss nodes, which were built up from a number of plate elements as shown in Fig. 41. At their most complex, they included large gusset plates in both directions to pick up truss and cross bracing elements, in addition to haunched beams on the other faces. The forces from the gusset plates are then transferred through a series of horizontal diaphragms into the circular section.

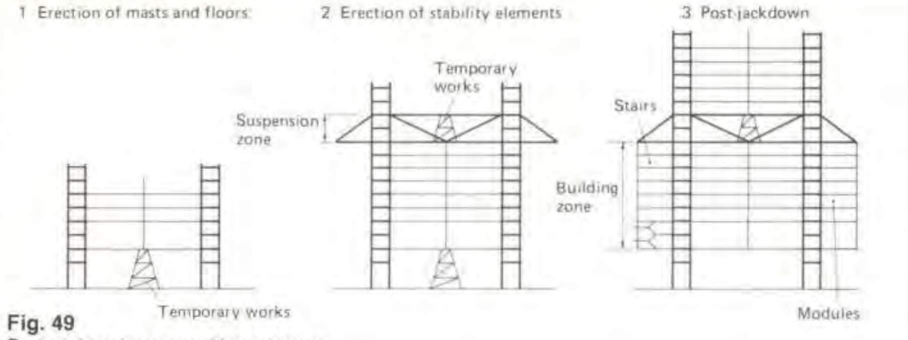


Fig. 49
Superstructure erection stages

Hangers were generally fabricated from extruded seamless pipe and internally threaded at their ends to take the screwed couplers. The connection between the hanger and the bottom boom of the stability truss was fabricated from a solid forged billet.

Floor secondary beams were cambered prior to erection, with the site connections to primary beams being generally bolted joints with grade 8.8 bolts. The welded plate girder primary beams were not fabricated to a camber because of the relatively low values of dead load deflection.

Welding
Prior to the commencement of fabrication an extensive programme of welder and procedure qualification was undertaken for manual metal arc and submerged arc welding.

22 basic welding procedures represented the various combinations of process, position, and preparation, that would be used in fabrication and subsequently for erection. All the basic procedures were centrally qualified to exacting standards of strength, hardness, and toughness. Each fabrication centre subsequently requalified selected procedures appropriate to the work to be undertaken.

Only one type of manual metal arc electrode and one submerged arc wire/flux combination were used throughout the contract. The control of steel composition and arc energy facilitated the use of only two welding preheat levels, ambient and 80°C.

Joints between can and diaphragm were generally partial penetration welds with machined bearing over the remaining area, since the members remain substantially in compression. The exceptions to this were at the suspension truss and cross bracing levels, where full penetration welds were used. The site welds between the flanges of the half-beams were also generally partial penetration welds except at truss and bracing levels, while the web connections were always full penetration welds with backing strips, since shear is generally the dominant mode of action at this joint.

For site welding a training programme was established to qualify welders locally for site work which, in its combination of type and required quality, was quite rare in Hong Kong. Key personnel from the erector were sent to the UK for training and qualification and, on return to Hong Kong, in turn trained others. Qualification tests were witnessed by Lloyd's inspectors. The programme proved very successful, with eventually some 50 welders on site producing work with not only high productivity but also low defect rates.

The size and configuration of components made radiography impractical for general weld testing, particularly on site, and butt welds were therefore examined ultrasonically. The scope and frequency of inspection and testing, together with defect acceptance criteria, were specified for all welded joints, varying in intensity according to the degree of importance of the weld.

Superstructure movement analysis

Requirements and objectives
The large floor-spans, non-symmetrical load distribution, and the 'zonal' nature of the structural action, required that accurate assessments be made of the movements of the main structural elements of the building both through the construction period and during its use. Following finalization of the erection sequence a detailed movement analysis was therefore carried out, with the following objectives:

(1) To provide a means of assessing the suitability of the proposed erection positions and levels of the suspension truss elements and the masts

(2) To predict the vertical and horizontal movements of each section of the primary structure during the construction sequence after its erection

(3) To predict maximum credible building movements under the imposed loads applied to the completed building

(4) To provide information on the fixing adjustment required in non-structural components.

The steelwork contractor carried out parallel analysis of the deflections expected during the erection of each zone. The effects of weld shrinkage in the site joints in masts and suspension trusses were not included explicitly in the movement analyses but assessed separately.

Erection sequence
The erection of the building structure was basically zonal, each bank of floors and its suspension trusses being essentially structurally complete before handover to following trades. Within a zone the central area floor steelwork was erected closely behind the masts, rather than waiting until the suspension trusses for that zone were complete. This gave significant programme advantages.

Until the suspension trusses were complete, therefore, the hangers had to act in compression and be temporarily supported from below. For this a trestle was placed under each pair of hangers, the trestles being supported by the work platform for the erection of the lowest zone and by the suspension structure below for higher zones. Following completion of welding in the suspension elements, the trestles below were removed during the 'jackdown' operation, whereby the structure was initially jacked-up off the trestles to remove the supporting packs and then let down until it hung freely.

Steelwork erection above continued within prescribed limits before completion of suspension trusses and jackdown of the zone below. Concreting of the floors was carried out after jackdown. Completion of the suspension trusses also allowed the erection of the outer hangers, floor elements, and stairs within a zone. Module erection was completed in a narrow time window and marked the essential structural completion of a building zone. The stages are illustrated in Fig. 49.

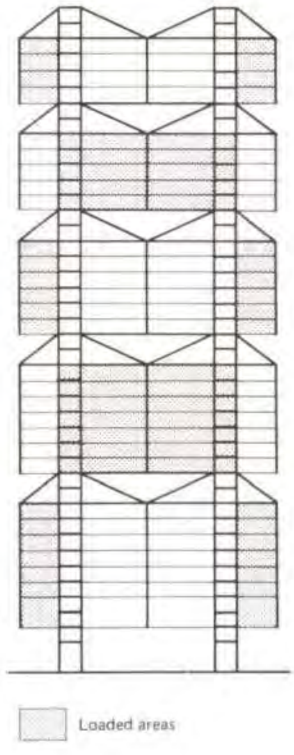


Fig. 50
Patterned live loading

The lowest, most slender hanger sections were stiffened to increase their compressive capacity to support the 'jackdown' loadings. Jacking forces and displacements were closely monitored during the operation to give warning of potential overloading of the hangers. The load/displacement records gave an indication of the stiffness of the structure, and once locked-in loads from welding were overcome, the structure behaved linearly: the measured stiffness agreed closely with predictions.

Analysis stages and models
In order to assess the movements in lower zones due to construction in higher ones, the sequences of structure erection, component installation, and fit-out were broken down into 12 discrete steps, each step representing a significant stage in the erection and load-out of the building. Analytical models to represent the stage of completion of the structure at each of the 12 steps were then chosen, each being a skeletal half-structure based on the main analysis model.

Loads
Assessments were made of the degree of completion of the building at each stage to establish the loads applied in the analysis. Wherever possible, manufacturers' estimates of the weights of components were used, particularly mechanical plant.

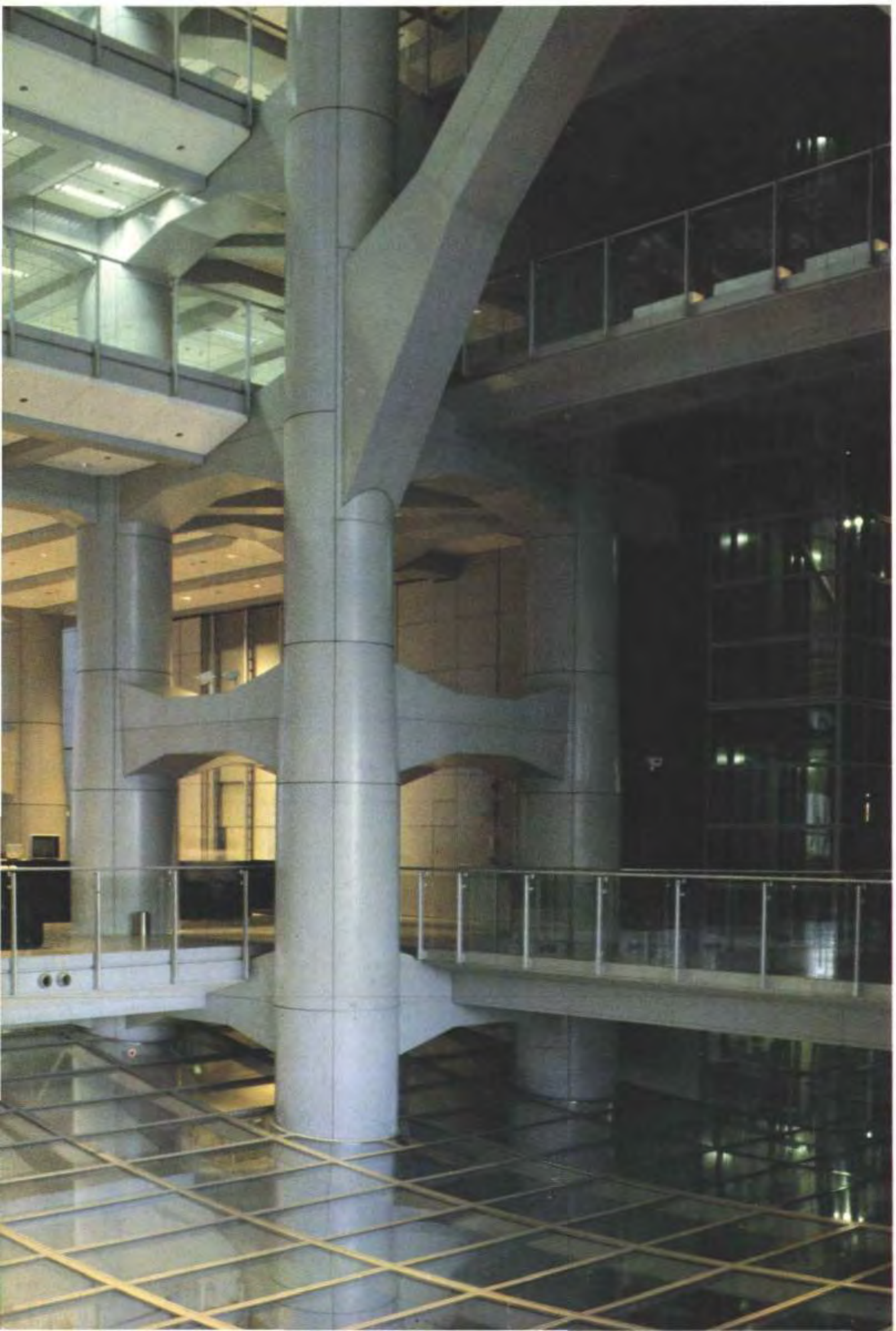
Live loads for the assessment of movements after construction were applied in three different patterns to predict extreme values of movement:

(i) 'Chequer-board' pattern live load, as shown in Fig. 50

(ii) The reverse 'chequer-board' pattern of (i)

(iii) Full live load throughout the building.

Joint effects
The geometry of the suspension truss zones is such that weld shrinkage at joints in both horizontal and vertical elements could significantly affect the level of the structure after jackdown. In addition to the welds between mast and node elements, there were welded site splices in both the top and bottom inner booms (Fig. 18). Measurements of weld shrinkage were made on site which generally showed values of between 2mm and 4mm.







Photos: Colin Wade


Weld shrinkage in the masts was compensated for in the individual joint setup, while weld shrinkage in the haunched beams and truss bottom booms was judged to have little or no effect on the vertical deflection of, or forces in, the frame. However, shrinkage during welding of the splice in the top boom induced significant forces in the suspension structure and hangers, which at that stage were propped from below. These forces were dissipated by movement during the jackdown of each building zone.

The prototype tests on the threaded couplers in the hangers and on the bearing connections showed little bedding-in movement on loading. Monitoring surveys during erection bore out these observations.

Sway

As discussed previously, the eccentric mass of the building causes it to sway progressively to the west during construction. Preliminary step-by-step construction analyses showed that, if mast columns were positioned concentrically with columns below during erection, the resulting swayed profile would be unacceptable for the installation of external cladding, lifts, and other fitout elements. A setting-out procedure was therefore developed to improve the final profile.

The following two procedures were evaluated by analysis prior to being adopted on site.

(1) Each mast element was erected relative to a base grid rather than concentrically with the column beneath. By this means, the horizontal movements that occurred between installation of successive sections were countered. This became known as setting out to the 'indian rope'.

(2) At selected levels the new mast elements were displaced to the east relative to base grid at erection. The 'indian rope' was thereby moved progressively further east between levels 22 and 41.

Fig. 51 compares the measured and predicted sway at completion of fitout. The horizontal axis in the figure shows movement subsequent to erection, and it can be seen that the measured and predicted movements agree within 9mm anywhere on the structure. The predicted position of the building relative to base grid is read off from the stepped vertical axis, which reflects the displacement of mast elements east of the base grid at erection. The maximum predicted final displacement is 18mm west of base grid at level 43. This compares with over 70mm if neither of the corrective measures had been taken.

Presets

Presets were introduced into the floors and suspension trusses at erection to ensure a level structure after concreting of the floors and installation of the modules.

Presets varied with suspension truss location, being significantly higher on the top trusses of a mast because of the absence of top booms. Typical preset values for these trusses were of the order of 40mm, while those for 'internal' trusses were approximately 20mm.

Each hanger was fabricated short to allow for extension under load. Fine adjustment was also possible at the threaded couplers on erection.

Movement effects: curtain walling

In concept the cladding elements are suspended panels fixed to the floor structure with adjustment in the fixings to allow the panels to be set level at the time of installation. Allowances for the relative vertical movements of the floors at the head and base of each panel are provided in the base fixing detail, with the movement allowance varying according to the panel's location within the building.

Since cladding installation followed closely behind steelwork erection in each zone, it was particularly important to assess the structure movements which would occur between installation of the cladding and completion of construction.

Movement allowances had been incorporated into the panel design at an early stage, and these could now be checked from the current analyses which used detailed information on the weights of fabricated and manufactured items, together with the final erection programmes.

The analyses provided the minimum movement capacities that would be required on erection, determined from:

(a) Floor deflections and main frame support movements occurring after installation of the cladding because of progressive construction and fitout of the building

(b) Floor deflections due to the superimposed live loading.

Generally, the cladding panels are single-storey-height elements between successive floors in one zone of the building. Movement joints in this situation are determined by relative movements between two floors only, plus temperature effects. The double-storey-height panels located within the suspension truss levels, however, require considerably larger movement allowances. Here the allowances are determined by the relative movements of two building zones rather than of two floors, since the upper stability level floor structure is linked via the hangers to the trusses above, while the lower stability level floor structure is linked to floors in the zone below (Fig. 18).

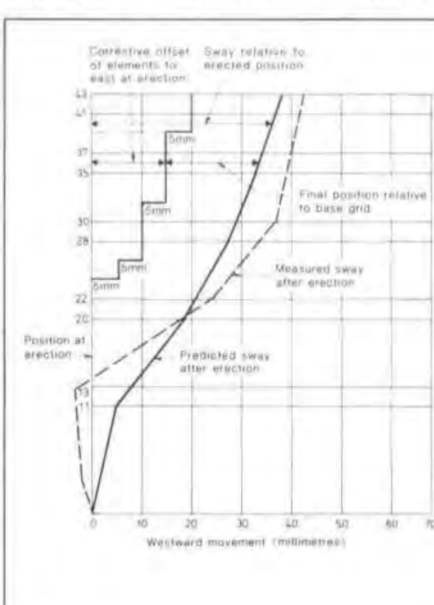


Fig. 51
Measured and predicted sway

Typical perimeter curtain walling has a movement joint capacity at each floor of ±30mm, including temperature effects, while at the edge of the building in a suspension truss zone the movement joint capacity at installation was set at 10mm opening/50mm closing.

Movement effects: fitout

The predicted movements of the building were used in the design and detailing of the fitout elements such as ceilings, raised floors, and partitions, to assess the adjustment required at their installation and the size of any movement joints required post-installation. The effects considered were:

(i) The level tolerance on the structural floor at handover to fitout subcontractors

(ii) Main frame movements and local floor deflections after handover of the structure, but prior to installation of the fitout elements, due to continuing construction above and addition of dead load to completed floors

(iii) Floor deflections due to superimposed live loading after installation of the fitout items.

The adjustment requirements to allow the raised floor and ceiling to be set level at installation were determined from (i) and (ii), and a nominal fitout envelope was determined as illustrated in Fig. 52.

The fully demountable partitions are a hung system, fixing directly onto the ceiling. The dimension between the raised floor and ceiling defined the nominal height of the partitions, and a vertical movement joint is provided at the base. This was set at ±50mm on typical levels to absorb any out-of-tolerance of the ceiling and raised floor installations as well as to allow for extremes of differential loading on the floors above and below. Installation adjustment for the partitions is then not required, since they can be fixed at any time to the ceiling line, and the differential live load deflections that may have occurred between the floors will be taken up in the base joint.

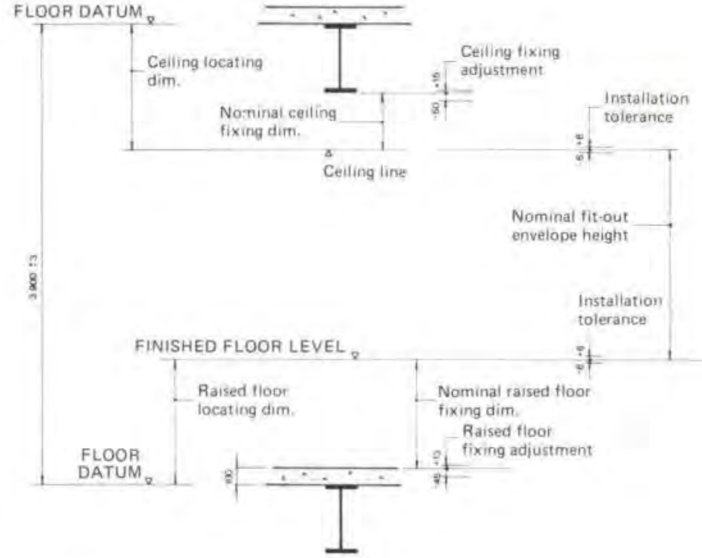


Fig. 52
Fitout envelope

Quality assurance and quality control

Strategy

It was recognized at the outset that the use solely of traditional methods of building supervision would be inappropriate for a large project assembled, to a great extent, from components prefabricated remote from Hong Kong and incorporating materials and techniques not extensively used in buildings.

Experience in the defence, nuclear power, and offshore sectors has shown that the establishment of a formal quality assurance system provides a workable basis for minimizing the risk of materials or workmanship failing to meet the specified standards. This approach was therefore adopted to ensure the acceptability of fabricated items before they arrived on site; site supervision would then focus on the assembly of components of assured quality.

Each contractor was required to establish quality assurance and quality control procedures to the satisfaction of the management contractor, with whom lay the ultimate responsibility for the quality of the constructed works. The implementation of these procedures was then monitored by independent surveillance teams specifically appointed in each country, with, in addition, regular visits by the management contractor's own QA/QC personnel and representatives of the design team.

This general approach was applied to all off-site fabrication contracts for the project, from structural steelwork through to fitout elements such as the ceilings and partitions, with variations in the degree of external inspection to suit both the importance of the elements and the required quality of product. The procedures described below for the main frame structural steelwork fabrication illustrate those used on all contracts.

Structural steelwork fabrication

The sections of the structural steelwork contract documents dealing with quality assurance and quality control were prepared jointly by the management contractor and the structural engineer and included requirements for personnel and procedures for the controlling and checking of the work. These formed a basis from which the steelwork contractor could develop the detailed quality plan for the works to reflect his own particular fabrication methods and controls.

Although critical items were subjected to full inspection and testing, it was recognized that many items could be checked on a random basis. In the event that a defective item was discovered it was vital to be able to identify the batch from which it came and isolate other potentially non-conforming items. Comprehensive documentation of material sources, fabrication activities, and quality control checks, provided a basis for such investigation and subsequent remedial action. It was fundamental to the success of the quality programme that, whenever defective items were discovered, not only were they rectified or replaced but that the reasons for non-conformance were established and any necessary changes instigated to prevent a recurrence.

In order to ensure that quality systems were operative and effective it was necessary to review systematically all activities and procedures. By necessity this included not only a review of documentation but also a degree of random retesting or inspection to substantiate the accuracy of documentary records.

Throughout the fabrication programme, therefore, the management contractor maintained a team of quality assurance surveyors whose task was to monitor the subcontractor's quality systems and carry out random inspections to ensure that the quality standards were consistently achieved at all fabrication centres, of which a total of 22 were used. Particular attention was paid to preshipment inspection of both fabricated components and supporting quality documentation.

In addition to the steelwork contractor's and the management contractor's quality assurance staff, inspections were also carried out by the consulting engineer to check that fabrication conformed to the design concept and that materials and workmanship generally complied with the specification.

A close working relationship was established early in the contract between the management contractor's QA surveyors, the contractor's QA team, and the consulting engineer. This ensured that all were aware of the technical requirements of the contract and the standards expected, and provided the QA team with support in implementing the quality programme. This group also provided an effective interface for considering the acceptability of non-conforming components, since it was inevitable that some would fall below the target quality level. Every effort was made to evaluate whether such non-conforming items could be accepted on an individual 'fitness-for-purpose' basis and so prevent disruption of the overall fabrication or construction programme.

On site

Procedures in the more confined location of the site were a combination of traditional inspection roles and QA/QC processes, particularly on the substructure and foundation works. The management contractor's site engineers controlled and checked all of the contractors' work, working with the contractors' own staff and a team of resident engineers from the consultant.

'Front-line' ultrasonic testing of welds on site was carried out by the steelwork contractor's site QA/QC organisation. A team of inspectors was specially qualified for the project to ASNT level II and worked under the supervision of a CWSIP 3.6 operator. Independent monitoring of their work was by the management contractor's CSWIP 3.6 operator, working together with the resident engineer.

Project programme

The following is a brief summary of the project milestones:

October 1979: competition result announced
May 1980: preliminary concept design
December 1980: final concept design
June 1981: demolition start
November 1981: diaphragm wall start
June 1982: excavation start
January 1983: steelwork start
January 1984: cladding start
November 1983: modules start
October 1984: steelwork complete
September 1984: modules complete
May 1985: cladding complete
May 1984: substructure complete
July 1985: Banking Hall handover
November 1985: building handover.

Acknowledgements

We were privileged to have a number of dedicated colleagues, and the work described in this paper is a tribute to their talents and hard work. However, we wish particularly to acknowledge the contribution made to the structural design of this project by the late Ken Anthony. Some of the original structural concepts came from his fertile mind, and he was the driving force behind the wind tunnel investigation. His untimely death was a loss to all associated with the design of the Hongkong Bank.

We would also like to thank the Hongkong Bank for permission to publish this paper.

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The Hongkong & Shanghai Bank. *The Arup Journal*, 17(4), pp.2-10, 1982.

(8) HUMPHESON, C., FITZPATRICK, A.J. and ANDERSON, J.M.D.
The basements and substructure for the new headquarters of the Hongkong & Shanghai Banking Corporation, Hong Kong (to be published)*

Credits

Architects:
Foster Associates

Consulting civil & structural engineers:
Ove Arup & Partners

Consulting services engineers:
J. Roger Preston

Quantity surveyors:
Levett & Bailey in association with Northcroft, Neighbour & Nicholson

Project coordinator:
R.J. Mead & Co.

Management contractor:
John Lok/Wimpey joint venture

Subcontractors:
Diaphragm wall:
Bachy Soletanche Group

Substructure works:
Dragages et Travaux Publics

Structural steelwork:
British Steel Corporation/Redpath Dorman Long joint venture

Corrosion protection:
Contech/Whitmore joint venture

Fire protection:
Morceau (Fire Protection) Hong Kong Ltd.

Glazed soffit to level 3:
GIG/Listo joint venture

Modules and risers:
HMT Consort (HK) Ltd.

Cladding and curtain walling:
Cupples Products.

* **Editor's note.**
An edited version of this paper appears later in this issue of *The Arup Journal*.

The basements and substructure

Chris Humpheson
Tony Fitzpatrick
John Anderson

This is an edited version of a paper under consideration for publication in the Proceedings of the Institution of Civil Engineers and is published here with the agreement of the Institution.

Fig. 1
Site plan

The site and adjacent buildings

The site overall is 68m north-south by 75m east-west, with the south-east corner slightly truncated. There are major roads on its north and south sides, a minor road on the east and buildings to the west (Fig. 1). Ground level slopes down from south to north, being approximately +6m PD (Principal Datum) along Queen's Road Central and +4m PD along Des Voeux Road. To the south of the site the foothills of Victoria Peak rise steeply from the south side of Queen's Road Central.

The Bank of China, located 10m beyond the site boundary, was built between 1949 and 1951. This building has a two-level basement extending down to about −2.5m PD. It is founded on driven cast in situ piles with expanded bases in decomposed granite at about −7m PD. The Chartered Bank, built in the mid-1950s, is about 10m to the west of the site and has a semi-basement (lower ground floor) which is at about street level on Des Voeux Road. This building is also supported on groups of driven cast in situ piles with expanded bases, with large pile caps generally linked by tie beams. The depth of the piles is unknown.

Prince's Building has a single basement down to about +0.5m PD with a stiff piled raft foundation. The piles are 0.5m diameter straight-shafted, driven cast in situ, founded at an average level of −16m PD in the decomposed granite. This building is about 35m to the north-west of the site. About 15m to the south of the site boundary is Beaconsfield House. This building has no basement and is founded on a grillage of spread strip foundation beams in decomposed granite at a level of about +5m PD. Both Prince's Building and Beaconsfield House were built in the early 1960s.

The oldest building in the vicinity of the site is the Courts of Justice, built *circa* 1900. This has no basement except for a small plant room close to the south-west corner. The building is founded on isolated strip footings which are believed to be supported on timber piles up to 15m long. At its closest point it is about 30m from the north-east corner of the site.

Site investigation

Before commencing the design of the substructure a preliminary site investigation was carried out during the period September 1979 to December 1980. The purpose of this investigation was to determine the geology and soil conditions at the site and to establish details of the existing basement construction and founding levels.

The investigation consisted of 13 boreholes up to 40m deep, three trial pits to inspect the basement raft construction and 17 small diameter cored probe holes to establish existing founding levels. Standard Penetration Tests were carried out and undisturbed Mazier samples taken in the soils, whilst double and triple tube core barrel samples were taken in the rock. A standpipe or piezometer was installed in each borehole to monitor ground water levels.

Following preliminary scheme design a more detailed second stage site investigation was undertaken between March and July 1981. The objectives of this investigation were to provide additional information on the geology and soil properties of the ground surrounding the site and adjacent buildings, to prove the depth and quality of rock at proposed foundation positions and to install an instrumentation system to monitor ground and water movements during the construction of the proposed basement.

The second stage site investigation consisted of a further 36 boreholes up to 48m deep, with Standard Penetration Tests at about 3m centres in all boreholes. Mazier samples were taken in six boreholes, and in situ permeability tests under constant, rising and falling head were made in six boreholes. In addition to the 36 boreholes, three deep wells were installed for pumping test trials.

The instrumentation which was installed consisted of seven standpipes, 22 standpipe piezometers and 11 inclinometers, four of which also had magnetic ring extensometers around the inclinometer tubing.

Ground conditions

These are discussed on pp. 7-9 of the paper by Jack Zunz, Mike Glover and Tony Fitzpatrick earlier in this issue.

Construction

Demolition

Construction began on the site with the refurbishment of the existing Annexe building on the west side. The original raft foundation was underpinned with 160 micropiles extending down to the granite to allow for an extra floor which was to be added to the building and the generally increased floor loadings. Some of the micropiles were installed as rakers to provide lateral stability during subsequent adjacent excavation.

Following completion of the Annexe fitting-out, the massive bronze doors of the main building, which had housed the Bank's headquarters since its official opening on 10 October 1935, closed after business for the last time on 5 June 1981. Demolition began immediately with the stripping-out of some fittings of sentimental value to the Bank, including parts of the 30m long mosaic on the old banking hall vaulted ceiling.

By late November 1981 only the 8m deep reinforced concrete basements of the old building remained, covering the northern half of the site, and preparatory works for diaphragm walling could begin. After the installation of temporary steel strutting, 1.5m wide slots were cut in 20m lengths through the slabs and the raft foundation where the new wall was located inside the existing structure. The slots were then back-filled with a lean-mix concrete to which bentonite had been added to maintain the propping action and yet allow subsequent excavation by diaphragm walling equipment.

This exercise proved both difficult and time-consuming since the concrete sections were massive and heavily reinforced and the base of the raft was below the water table. Nevertheless, the process was completed in some seven weeks.

Diaphragm wall

Exploratory prebore holes had meanwhile been drilled at approximately 4m centres along the complete line of the diaphragm wall. These gave the probable founding levels for all panels, and also indicated the variations in the ground through which they would be excavated.

The 51 diaphragm wall panels, with a total perimeter of 245m, were excavated under bentonite using a clam-shell grab, with three rigs in operation on the site. Each panel was toed-in 300mm into moderately decomposed granite by chiselling. Panel lengths varied from 3.7m to 6.0m, with primary and secondary panels being cut in three passes and successive panels in two.

Experience on previous diaphragm wall construction in the area had shown that ground movements were very sensitive to the excess piezometric head between the slurry inside the trench and the ground water outside. Since raised guide walls were not practical on the site, this excess head could only be generated by lowering the external ground water level. This in itself, however, generates ground settlement, and hence a fine balance had to be struck between ground movements caused by too low an excess head and those due to dewatering to increase the head (see section on ground movements for a fuller description of this problem).

A general drawdown of the surrounding groundwater of 2m was finally specified, with absolute minimum piezometric levels of −2m PD in the decomposed granite and 0m PD in the overlying marine deposits and fill. To carry out the dewatering and subsequently to maintain control of the water levels, 37 wells, each 150mm diameter, were drilled to rock at 6m centres around the outside of the site, plus a further 15 wells inside the old basement. The wells were equipped with eductor well-points which allowed them to be used as either a drawdown or a recharge system, thus ensuring that close control could be exercised.

Piezometers and standpipes which had been installed around the site were used to monitor drawdown levels. Those adjacent to the panels under excavation were read at

Photo Ian Lambot

six-hour intervals, others close to the site every 24 hours and those remote from the site twice each week.

The original specification for the bentonite slurry for use in the diaphragm wall trenches was based on the API Standard Procedure for Testing Drilling Fluids, with the values based on Wyoming bentonite. However the bentonite actually used was of French origin, and it proved impossible to achieve results from the daily tests which complied simultaneously with the specifications for density and shear strength. The slurry with the specified density had too low a shear value, whilst changing the mix to give the correct shear value produced too dense a slurry which would not allow sufficient settlement of fines and resulted in too high a suspended sand content. This would have resulted in poor concreting and a poor rock-concrete interface.

Following a series of tests the specification was modified to suit the actual material being used, although this was accompanied by stricter controls on slurry level in the trenches and ground-water piezometric levels.

On completion of excavation of a panel the profile was measured for width and plumb using a Koden Drilling Monitor. This produced a graphical read-out of the profile of the sides, and enabled trimming to be carried out to ensure that the specification was met (1:80 vertically, width not less than 1000mm). Panel excavation in total took 318 rig-days, finishing in July 1982, equivalent to some $21m^2$ of panel per rig per day. The deepest panels went to 36m below ground, with an average depth of 27.5m.

At the start of diaphragm wall installation in the north-west corner of the site, obstructions in the form of the sheet pile temporary works retaining wall to the old building basement were encountered. These had eventually to be taken out piecemeal using hand excavation, when it was also found that the original sea-wall ran along the line of the diaphragm wall.

The critical design cases for the reinforcement to the diaphragm walls were those arising during basement excavation and construction. Maximum cage weight was 28 tonnes, installed in three sections using couplers to join the lengths to avoid congestion. A total of over 1,200 tonnes of reinforcement was used in the diaphragm walls. Panel concrete was a retarded, self-compacting tremie mix with a characteristic strength of 32 N/mm^2, supplied from a nearby ready-mix plant. Concreting time averaged approximately six hours per panel.

Following completion of the diaphragm wall, the base of the wall was grouted to ensure as watertight a 'box' as possible for subsequent basement excavation. Two-stage grouting was employed, contact grouting of the concrete-rock interface being followed by fissure grouting to 5m below the base of the wall.

Four ducts, each 100mm diameter, had been placed to the bottom of each panel with the reinforcement cages. Each duct was drilled 500mm into rock and grouted at 10 bars to refusal. Following this contact grouting, alternate holes were drilled 5m into the rock and grouted at 15 bars to refusal.

An internal dewatering system was installed within the site with two objectives. Firstly, it allowed a reduction of ground-water inside the site below the excavation level so as to increase the available passive soil resistance, and secondly it allowed a check on the effectiveness of the grouting before commencement of excavation.

A total of 15 wells was installed, each with a submersible pump, allowing a stable drawdown to − 15m PD to be achieved. This was maintained for some two years throughout the construction period, the last wells being switched off only after completion of the rock anchors to the lowest basement slab in June 1984. Piezometers left in the central wells showed that piezometric levels recovered to their original values by September 1984.

Caisson foundations

Three types of caisson excavation were used to form the foundations (Fig. 2). The most numerous were the 58 'secondary' caissons for the basement columns. These were 2.1m internal diameter shafts, hand-dug in the traditional Hong Kong manner.

Each caisson is dug by one or two men using hand-tools at the base of the excavation, with spoil-removal by electric hoist generally operated by female members of the diggers' family. At the end of each day's excavation, generally a depth of 0.8m to 1.0m, a 100mm thick concrete lining is poured to stabilize the sides, using a wedge-shaped shutter to allow both a gap for pouring and easy removal of the shutter the following day.

All secondary caissons were excavated to a minimum of 300mm into massive granite, giving an average base level of − 25m PD, with the deepest extending to − 33m PD. As a final check that there were no areas of weak rock below the base, 50mm diameter probe holes were drilled 4.5m below each caisson and the rate of penetration measured. In four of the caissons extra excavation was required as a result of these probes. Excavation of secondary caissons took just two months, finishing in November 1982, and at the peak employed 48 excavation gangs.

On satisfactory completion of the probe hole in each caisson the reinforcement cage was placed, together with the ducts which would subsequently be used to install the permanent rock anchors, and the shafts concreted up to the level of the lowest basement slab. Although it had originally been intended to tremie the concrete into the caissons, the combination of the grouted base to the diaphragm wall and the internal dewatering system so reduced the flow of water into the caissons that concreting was in fact carried out effectively in the dry.

In order to be assured that there was good contact between the concrete and the rock at the base, ten caissons were selected at random and cored through the concrete to 500mm into rock. All cores showed a void-free and gravel/sand-free contact.

Holding down bolts and baseplates were set into the top of the concreted shafts and the 356 × 406 UC sections of the basement columns located onto them, with the open bore around the column then being filled with a no-fines concrete up to ground level to provide lateral stability during construction.

To gain programme time, general excavation to B1 level had been carried out in parallel with the caisson excavation for the basement column. However, since the main temporary propping system at ground floor level was to be supported on these columns, a preliminary propping system was needed to allow this first phase of excavation to proceed. This was provided by 2m × 1m deep reinforced concrete beams supported on the old building's foundations, fortuitously occurring at reasonably useful levels.

After installation of the basement columns, the main temporary propping system, consisting of twin 900 × 300mm I-beams, was installed and the preliminary system broken out. This propping system also formed the working platform for the site, and allowed superstructure construction work to proceed simultaneously with the basement works. The system was designed by the management contractor for a live loading of 50 kN/m^2 to cater for the construction plant which would use it.

As described in the previous paper the main masts (6.5m × 6.2m overall plan size) extend to the lowest basement level. In order not to have to wait until the general construction had reached this level before commencing mast erection (which was on the overall project critical path), ways were sought to advance these local areas and so mitigate the effects of programme slippage which had occurred in the first phase of the works. The solution finally adopted was to construct 10m internal diameter access caissons in each of the eight mast locations to extend to B3 level.

Excavation was carried out by a combination of hand tools and small back acters and proceeded in steps of 0.5m depth, the 500mm thick reinforced concrete lining being cast in each 0.5m before excavating the next step. The access caissons varied in depth from 8m to 14m depending on location, reaching up to 19m below road level.

From the base of each access caisson were dug the four primary caissons for the mast foundations, one under each column of the masts. Shafts were of 2.5m, 2.7m and 3.5m diameter, with bell-outs in the rock to either 3.5m or 4.1m diameter. Blasting was used for the base excavations into rock, involving close monitoring of adjacent bank buildings to ensure vibration levels did not affect sensitive equipment. As with the secondary caissons the bases were probe-drilled to ensure sound rock beneath, with three holes being drilled in each base. Average penetration of the foundations into rock was 5m,

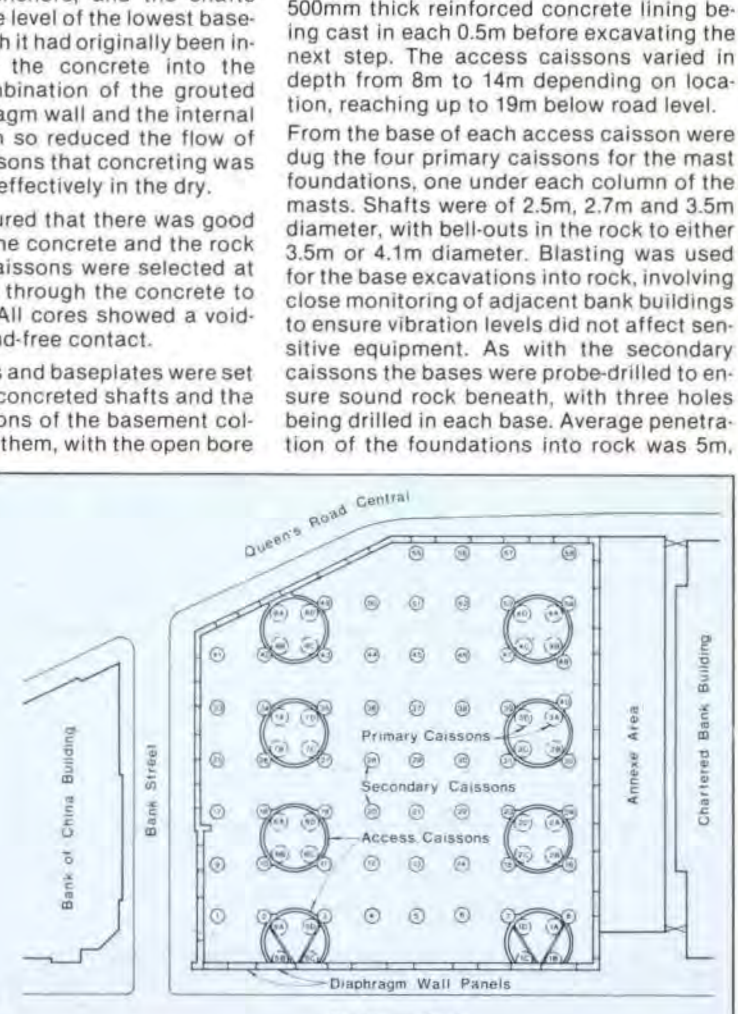

Fig. 2
Diaphragm wall and caissons

Photo Ian Lambot


HILTON

with a maximum of 7m and an average base level of approximately −30m PD.

Test-coring of the first four primary caissons, which had been concreted by tremie under water, showed an unacceptable sand/gravel layer some 100mm thick at the concrete-rock interface. Although several possible remedial methods were discussed, in view of the high loads on the caissons and their importance to the total building, it was decided, albeit with reluctance, that the only sure remedy was to break them out totally and reconcrete. The method of concreting was changed for the remaining 28 caissons, which took place during December 1982, as well as for the four re-built ones. All primary caissons were cored after concreting and all showed sound concrete-rock interfaces.

Excavation and slab construction

Successful completion of the access and primary caissons allowed mast steelwork erection to start on programme in January 1983, at which time the combined work platform/propping system was completed and basement excavation and construction could begin in earnest.

The general site level having been reduced to 1m below the underside of the first basement level, the slab formwork support system could be installed. A high quality, fair-faced concrete finish was required, and hence a system comprising resin-coated plywood forms laid on a pre-cambered grillage of steel beams was employed. All joints were to be recessed and featured, and great care was exercised by the contractor in the design of the forms, which was done in close collaboration with the design team. The support steelwork was set on screw-jacks at the columns to give fine level adjustment.

The plywood forms were imported from Finland, and erection of the formwork for the first time took some six weeks. However, for subsequent levels the formwork was simply lowered intact from the slab above on threaded suspension bars connected to hydraulic jacks above the slab, the formwork having been initially lowered 50m at seven days after casting. Great care was exercised during all activities on the plywood. Carpenters were issued with plimsolls during fixing, and each area was covered with polystyrene before steelfixing commenced, which was not removed until immediately prior to concreting.

The structural requirements for the concrete were quite normal, the design having been based on a characteristic strength of 30 N/mm^2. However the very high quality of finish required, together with the strict colour requirements of a uniform light grey over the whole of the large areas, placed great demands on the mix design.

A series of trials was therefore initiated, involving vertically and horizontally cast trial panels (1m x 1m and 2m x 1m respectively) using different release agents, plasticisers and mix proportions. Three months of testing culminated in April 1983 with the casting on site of a 3.5m square, 300mm thick prototype panel, complete with full reinforcement and features, which then formed the basis for judgement of finish of the actual works.

All basement slab concrete was pumped from the work platform, with pours averaging 133m^3 and taking some seven hours. The final finish on the slabs is generally considered to have repaid the time and effort expended in the design and preparation, it having been described as the best as-struck concrete in Hong Kong.

Excavation below concreted slabs was with five mechanical front shovels loading two skips which were raised up to the work platform through access holes left in the slabs. At this stage also the diaphragm wall was cleaned off as it became exposed and checked for any leakage, and the horizontal reinforcement couplers, which had been included in the wall for connecting to the basement slabs, were exposed and checked for location. In general the wall proved to be exceptionally watertight, and with only a few exceptions the couplers were able to mate up with the slab reinforcement.

Rock anchors

With the commencement of rock anchor installation there began the last phase of the underground structural works. A total of 245 anchors was required in the main basement, of which 36 are located in primary caisson foundations under masts to provide an increased factor of safety against uplift due to wind loads, whilst the remainder are in the secondary caisson foundations to the basement columns to resist hydrostatic uplift. A further 66 anchors are located in the single-level Annexe area basement on the west side, which are similar in principle to those in the main basement.

Anchor loads varied from 650 kN to 2100 kN, with all the anchors being designed with double-corrosion protection systems based on the then-new draft British Standard Code of Practice *DD81*. A 111mm external diameter corrugated plastic sheathing was used as the main barrier, with plastic tubing around individual strands in the free length in addition to the internal grout. Anchors were pre-assembled and wound onto a custom-built drum off-site, and installed directly from the drum.

Prior to anchor installation on site a series of tests was carried out to prove the design, including two full anchor tests, one of which was on site in a redundant duct. Bond-length, transmission of bond through the sheathing, in situ grouting methods and the effects of the tubing around the strands were all investigated before the details of the installation were finalized. Water tests were carried out at the base of each hole after drilling, and pre-grouting carried out as necessary.

The complete anchor installation took some six months from November 1983 to May 1984, with the work being organized (and reorganized) around the other construction, and in effect becoming the Cinderella of the basements.

Because of their importance to the building, all the anchors are located so as to be accessible for re-stressing. In addition, 28 anchors have been selected for permanent monitoring as part of the building management process. Following the successful completion of the end-of-installation lift-off tests in January 1985, routine monitoring will commence in January 1987 and be repeated every five years thereafter.

Plaza floor

Possibly the most public area of the new building is the ground floor, known as the Plaza, which combines its role as the main entrance to the new banking hall with that of a north-south public thoroughfare under the building to link two of the city's major east-west pedestrian routes.

To overcome the 2m level drop across the site without interrupting the continuity of space, the architects selected a series of short slopes connected by level strips. A granite top course overlays waterproofing and thermal and acoustic insulation, and the whole 'sandwich' is supported on precast panels, interlocked by an in situ topping, on a grillage of in situ reinforced concrete beams.

Following the success of the as-struck finish on the basement slabs, an exposed concrete soffit to the Plaza floor was selected as the ceiling to the public area in B1 below. Precast panels had to be used, however, due to the construction restrictions in this congested area, and a further series of concrete finish trials was therefore carried out to ensure that the required finish would be obtained. A total of 859 precast units were manufactured, generally 2.4m x 1.2m in plan and 200mm thick.

On-site organization

The organization of the construction contracts was through an overall management contract whereby sub-contracts for sections of the work, known as packages, were let. This arrangement allowed the tendering of individual contracts on a competitive basis as soon as that portion of the design was complete, thus ensuring minimum delay before execution of the work whilst retaining the cost benefits of competitive tendering. The management contractor had overall responsibility for quality control, programming, temporary works and construction co-ordination, and also provided the common-user services such as hoists, electrics, toilets, etc.

To control and keep record of the many different activities, official procedures on the site required all communications to pass through the management contractor. This could sometimes become quite involved, but as often happens the individuals concerned rose to the occasion and a very close working relationship developed between resident engineers, management contractors and sub-contractors whereby problems were resolved by discussion as they arose, to be followed later by the paperwork.

This proved particularly effective during diaphragm walling and basement concreting, where decisions and actions were required immediately as the work progressed if delays were to be avoided.

Ground movements: General

It is inevitable that construction of a 20m deep basement will cause ground movements. The object of the extensive design and analysis which was carried out was therefore to develop a construction sequence which would be as fast and as cost-effective as possible without causing distress to adjacent buildings and services.

At all stages of the design, considerable effort was made to estimate ground movements and to assess the consequences of those movements, with the database for the preliminary analyses being principally obtained from records of movements associated with the construction of other projects in the area. Back-analyses were subsequently carried out during construction using the as-measured movements to improve the quality of the final predictions.

The three main causes of ground movement which had to be investigated were dewatering, diaphragm walling and the excavation sequence itself.

Dewatering

With ground water levels within 2 to 3m of the surface, dewatering within the site to reduce these levels by at least 18m was required for the basement construction. Past experience had shown that dewatering completely decomposed granite (CDG) can cause significant ground settlement and so it was important that a reasonable assessment be made of likely drawdown outside the site boundary.

Records were available for drawdown due to dewatering of a nearby site during caisson construction. These records showed that dewatering caissons to 20m below ground level caused drawdown at up to 250m from the site. This data was used in a back-analysis to assess the bulk permeability of the CDG and overlying strata and to develop a numerical finite element model which

Photo: Ian Lambot

could then be used to investigate future dewatering proposals.

This study showed that the pattern of drawdown in the CDG is largely dependent on the extent to which the CDG aquifer is confined. To achieve a match between the calculated and measured drawdown it was necessary to postulate an increasing permeability with depth within the CDG, and also no recharge of the CDG aquifer from the overlying fill and marine deposits (i.e. to 'make' the CDG stratum behave as a confined aquifer). This requires a low permeability material at the interface between the marine deposits and CDG strata, and results in a perched watertable in the fill when the CDG is dewatered. Such a phenomenon was observed in practice.

The numerical model was then used to estimate drawdown due to dewatering inside a diaphragm wall box. The results of this analysis clearly demonstrated the importance of achieving a good seal between the diaphragm wall and the intact rock. Even a small gap between the base of the wall and rock was shown to result in significant drawdown outside the site (Fig. 3). The analysis indicated that drawdowns could be limited to about 2m to 3m, which would be acceptable, by ensuring that any gaps were grouted.

During the second stage site investigation, further data on ground settlement due to dewatering was obtained by carrying out a series of pumping tests. 250mm diameter wells were installed on Des Voeux Road and Queen's Road as part of the SI contract.

When the SI had been completed and all piezometers and standpipes installed, pumping tests were performed by reducing the water level in the wells in stages using submersible pumps and monitoring the piezometric levels in adjacent standpipes and piezometers. Temporary settlement points were installed for the duration of the tests to monitor ground settlements.

The drawdown in the CDG and associated settlements were greater for the wells on Des Voeux Road than for the well on Queen's Road Central. Very little drawdown was observed in standpipes in the fill on Des Voeux Road. This indicated the existence of a relatively impermeable layer between the marine deposits and the CDG preventing vertical flow into the CDG from the overlying highly permeable fill, a conclusion consistent with that derived from the earlier back analysis. On Queen's Road Central, however, the marine deposits are absent and the limited radius of drawdown and small settlements indicated that the fill and CDG are hydraulically connected.

The ratio of ground settlement to drawdown measured in the pumping tests was considerably less than the previous records indicated. This was attributed to the fact that previous measurements could be associated with first time dewatering of the CDG, whilst the pumping test results probably reflected the fact that the site had already been dewatered during the construction both of the previous Bank headquarters and of nearby buildings.

From the results of the computer analysis of drawdown outside a diaphragm wall and the measurements taken during the pumping tests, it was concluded that ground settlements due to dewatering to rock level inside the completed diaphragm wall box would be of the order of 10mm.

Ground movements due to diaphragm wall construction

During the construction of Chater Station, significant ground movements occurred as a direct result of diaphragm wall construction. Ground settlements of up to 75mm at a distance of 3m from the wall were reported[1].

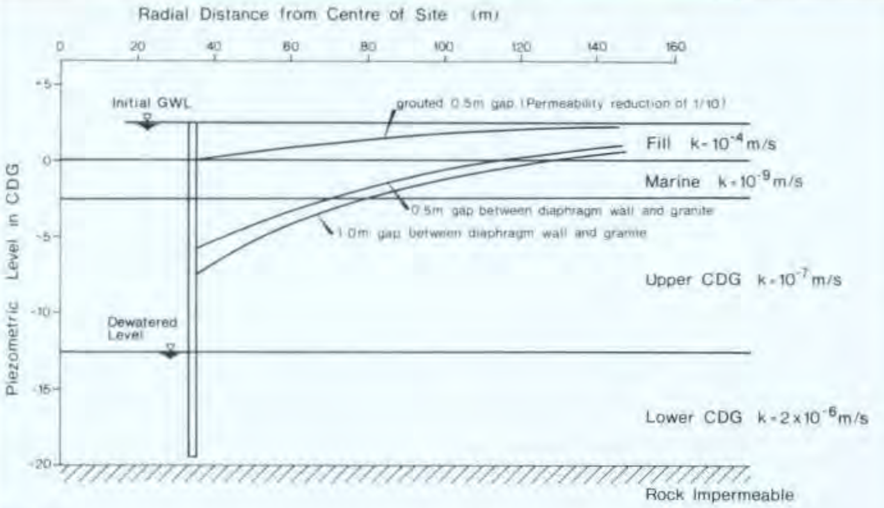

Fig. 3
Effects on dewatering of gaps at base of diaphragm wall

These ground movements were subsequently attributed to swelling and softening of the CDG adjacent to a trench under excavation due to horizontal stress relief. Field tests carried out in Hong Kong showed that the movements are related both to the effective stress in the decomposed granite adjacent to the trench during excavation and to the size of the panel. The stresses are controlled by the difference between the head of bentonite slurry in the trench and the head of water in the ground (i.e. the excess slurry pressure).

The field measurements available were limited but indicated that the magnitude of the ground movement was very sensitive to the excess slurry head, especially at low values. To investigate this problem a series of drained triaxial tests was undertaken to study the swelling of CDG due to reduction in horizontal stress. These tests demonstrated that the rate of increase in radial strain, or swelling, rapidly increases at low values of horizontal stress.

The object of these studies was to obtain the optimum balance between panel size, excess slurry head and dewatering to minimize combined ground settlements whilst at the same time not impeding construction progress. The specification finally adopted was for a maximum panel length of 6m and a minimum excess slurry pressure of 35 kN/m^2 (approximately 3.5m head), to be obtained by dewatering the surrounding ground so as to avoid the complications of using raised guide walls on the site.

It should be noted that the swelling problem is confined to the CDG, and in the fill and marine deposits an excess slurry head is only required to maintain trench stability. This was calculated to be 2m.

Ground movements due to basement excavation

The most difficult assessment to make was that of ground movements due to basement excavation. During the excavation lateral movements of the diaphragm wall would occur as the earth pressures from the surrounding ground are transferred onto the wall and support system. This causes horizontal movement and consequent settlement of the surrounding ground.

Obviously, the magnitude and extent of the movements are controlled by the stiffness of the wall and support system, the properties of the ground and the sequence of construction. The difficulty arises in formulating a suitable method of analysis accurately to assess the importance of each of those factors. The first estimate to be made is of the lateral wall movements during the various phases of construction. It is then necessary to assess the likely ground surface settlements associated with those lateral movements. Both of these stages are fraught with difficulties.

Several methods of estimating lateral wall movements have been proposed in the past. The majority of these methods are based upon subgrade reaction models where the wall is modelled as a vertical beam and the soil is represented as a series of independent horizontal springs. Depending upon the sophistication of the method, the earth pressures, represented by spring forces, may or may not be limited by the active or passive pressures. In these models the user is required to specify spring stiffnesses or subgrade reaction moduli to represent the soil. These are difficult parameters to define since they depend not only on the soil properties themselves but also on the geometry of the excavation.

To overcome some of the problems associated with subgrade reaction models, a method of analysis was developed which assumes that the soil on both sides of the wall behaves as a linear elastic continuum between active and passive limits[2]. In order to use this computer program to assess wall movements it was necessary to define the soil properties in terms of modulus of elasticity and the active, passive and at-rest horizontal pressure coefficients.

To test the validity of the model, use was again made of data obtained during the construction of Chater Station, where inclinometers had been installed within the diaphragm walls. The measurements of lateral movement at various stages throughout the construction period were used to 'calibrate' the current analysis.

Reasonable agreement between measured and calculated wall movements was achieved if the lateral soil stiffness, E', was assumed to be related to the SPT blowcount, N, by the relationship $E' = 0.8 \times N$ MN/m^2, and the horizontal pressure coefficients were related to the angle of shearing resistance, ϕ', determined from laboratory tests using Caquot and Kerisel[3]. The site investigation had demonstrated that ground conditions along Des Voeux Road were quite different from those along Queen's Road Central.

Two models were developed to represent the north and south sides of the site, with the different soil properties derived from the SPT profile for each side as shown in Fig. 4. The calculated wall movements at the end of basement construction are shown in Figs. 5 and 6.

No standard procedure exists for relating ground settlement to lateral wall movement.

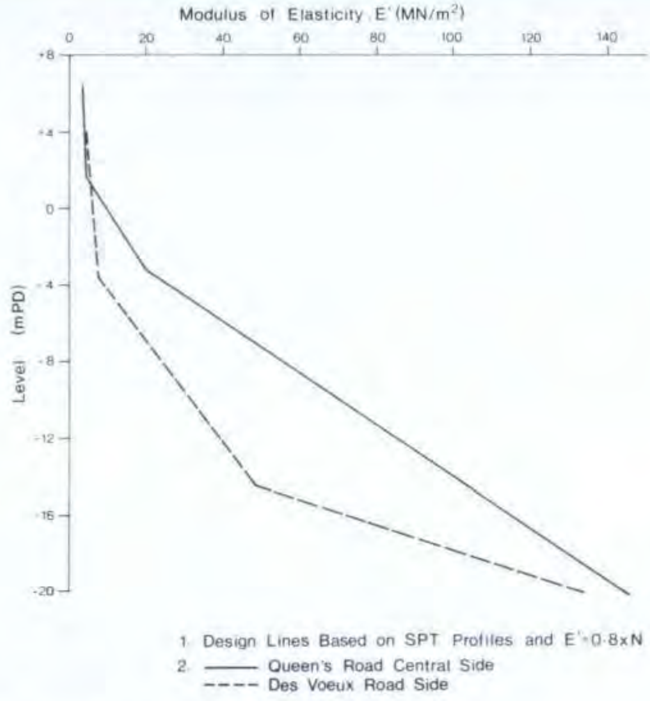

Fig. 4
Variation of soil modulus of elasticity with depth

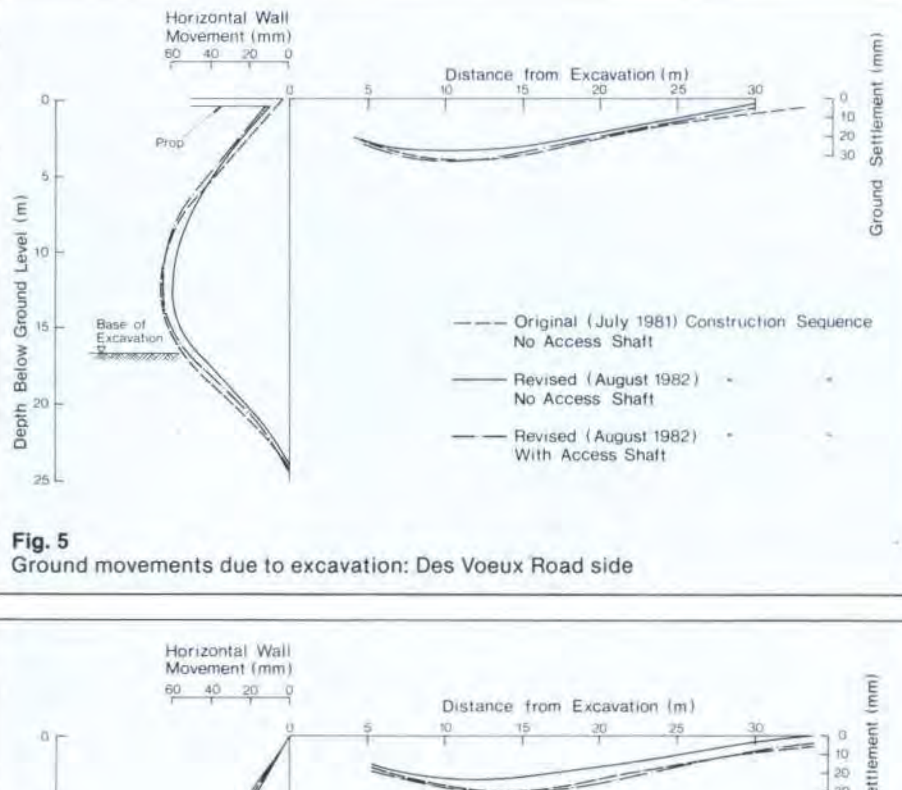

Fig. 5
Ground movements due to excavation: Des Voeux Road side

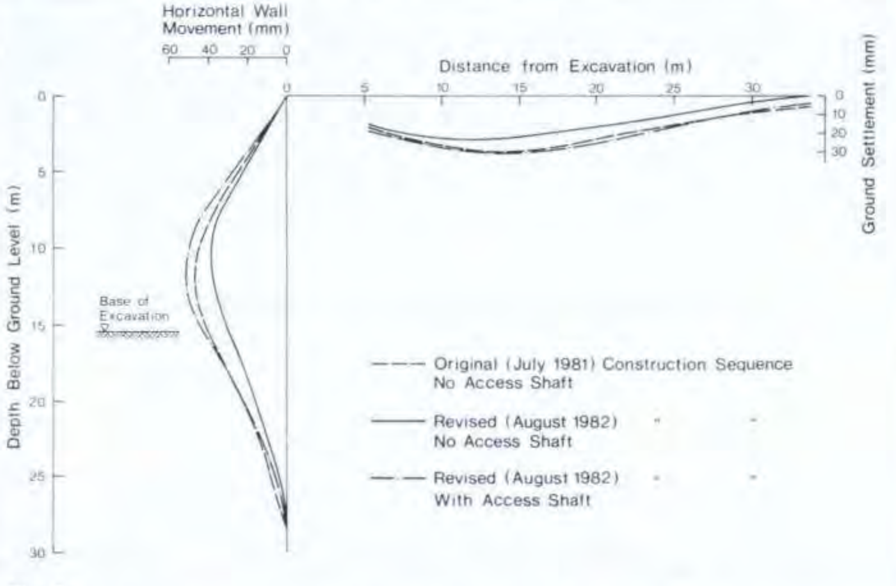

Fig. 6
Ground movements due to excavation: Queen's Road Central side

Finite element analyses can be used, but as with the lateral wall analysis it would be necessary to calibrate these solutions against measured values. This was attempted for the Chater Station data but was both extremely complex and not very successful. Assuming elastic behaviour for the soil led to a great underestimation of ground settlement, and it was necessary to adopt an elasto-plastic model in order to achieve anything approaching the measured values.

A study was made of published data relating lateral movements to ground settlements, and from this the relationship indicated in Figs. 5 and 6 was developed for ground movements due to excavation.

The numerical models were subsequently used to carry out parametric studies on such variables as prop stiffness, slab levels, extent of excavation prior to concreting of slabs, wall stiffnesses and the timing of ground water lowering. In this way an optimum construction sequence could be developed which not only satisfied ground movement criteria but also recognized the cost and programme requirements of the project.

One of the most important conclusions of this study was that in order to keep overall lateral wall movements within reasonable bounds the top of the wall had to be propped at all stages of construction. If the top were allowed to move the resulting wall movement would dominate subsequent movements. Another important finding was that dewatering the site to rock-head level before excavation began, did not result in greater movements than dewatering as the excavation proceeded.

The effect of omitting a slab level during initial excavation was also examined so as to speed up the construction programme. Although initially feasible and included in the construction sequence, the basement scheme was subsequently modified to include the 10m diameter access caissons. These were introduced at a late stage to recover lost programme time and so allow the superstructure steelwork to proceed on schedule. In order to compensate for the extra movements which would be generated by these large caissons, the construction of the 'omitted' slab was therefore reinstated to its sequential place in the top-down method.

Further programme gains were subsequently made when the ground movement survey results at the end of excavation to B1 level showed somewhat less settlement than predicted. This allowed a relaxation to be given on the limit of excavation below a slab prior to the concreting of that slab. The ability to assess rapidly the consequences of such scheme modifications was of significant benefit to the project, since it allowed the incorporation of measures to speed progress by being able quickly to satisfy the strict requirements of the Hong Kong Government Approving Authorities in relation to assessment of effects on adjacent buildings and roads.

Predicted total settlements

The initial predicted total ground settlements for all phases of basement construction are shown in Fig. 7. As can be seen, the largest settlement of about 55mm was predicted on Des Voeux Road whilst the maximum predicted settlement for Queen's Road Central was about 45mm. The largest component of settlement was that due to basement excavation.

A reassessment of the ground movements due to excavation was subsequently carried out, as has been described, in order to check the viability of the modified construction sequence using the 10m diameter access caissons. The movement profiles generated by this study are shown in Figs. 5 and 6. It was found that although the access caissons caused a significant increase in ground movements, the introduction of full propping action at B2M level during excavation (where previously a double-height dig had been proposed) compensated sufficiently to allow the access caissons to be used.

Estimates of the expected settlement of adjacent buildings indicated a maximum of 20mm – 25mm for the Bank of China and the Chartered Bank. Fortunately the Courts of Justice fell outside the expected zone of influence and so no movement of this building was anticipated. The calculated differential settlements were small, well within the limits defined by Burland & Wroth[4]. At worst the predicted movements should result only in minor cracking in plaster finishes, and no structural damage to any of the adjacent buildings was anticipated. It was considered that close to the excavation, settlement and lateral ground movements could cause small cracks to appear in the roads and pavements. Differential settlements however were again small and hence tram lines and underground services were unlikely to suffer damage.

Instrumentation

From the early planning stages it was decided to install instrumentation to monitor ground movements and ground water levels, and also to monitor closely the movement of nearby buildings. This instrumentation would act as an 'early warning' system if effects were not as anticipated, allowing either the construction method to be modified or preventive measures to be implemented before any actual damage occurred.

An example of such measures was during caisson excavation, when several caissons were being constructed immediately adjacent to the diaphragm wall and in some instances extended several metres into rock below the toe of the diaphragm wall. During the final stages of excavation for these caissons the levels in nearby piezometers were observed to fall and approach the maximum allowable drawdown limit. To prevent excessive drawdown the CDG was recharged locally through the existing eductor wells, and the piezometric levels were thereby maintained within acceptable limits.

In total, 41 piezometers and standpipes were used to monitor ground water levels around the site. 11 inclinometers were installed to monitor the variation of lateral ground movement with depth, and a further 12 inclinometers were installed within the diaphragm wall to monitor lateral wall movements.

29 precise ground settlement survey stations were installed at strategic locations around the site. The number of ground settlement stations was subsequently increased by the installation of an additional 27 settlement points by the Hong Kong Highways Office, who were concerned about the effects of the construction on underground services. The concern of the Highways Office was shared by the Building Authority, who required the installation of a further 30 temporary settlement stations before allowing construction to proceed.

Settlement stations were also installed, with the owners' consent, on all neighbouring buildings. On the immediately adjacent buildings three targets were installed at various heights up the building facade so that tilt of the buildings could be monitored.

Having installed this instrumentation the taking of readings became a full-time occupation. Two subcontractors were appointed to carry out the work. The first contract was essentially for surveying work, in-

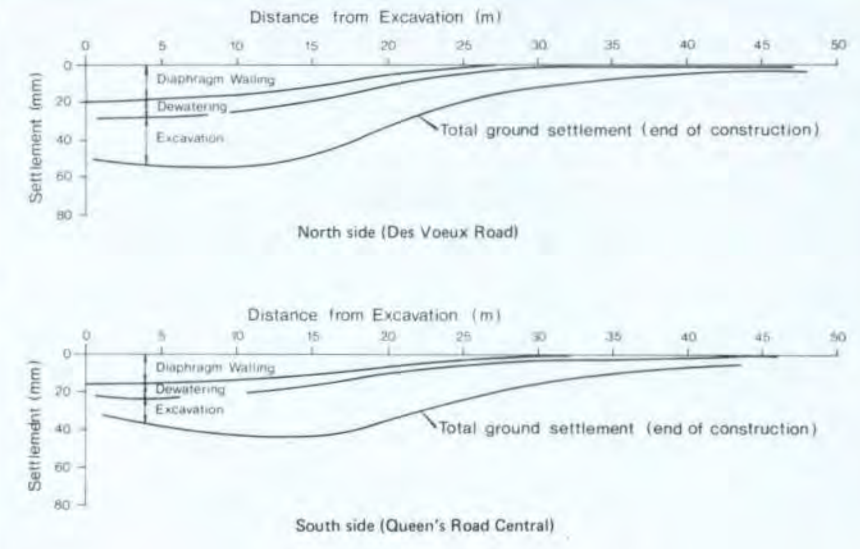

Fig. 7
Initial predictions of total ground settlements

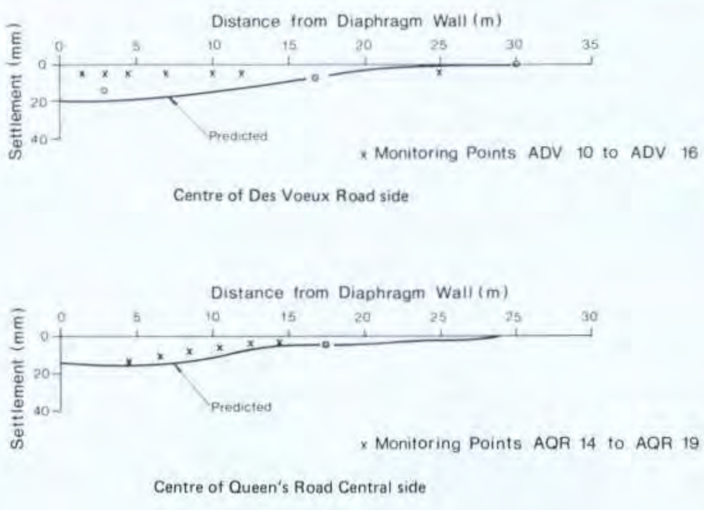

Fig. 8
Measured and predicted ground settlements due to diaphragm walling

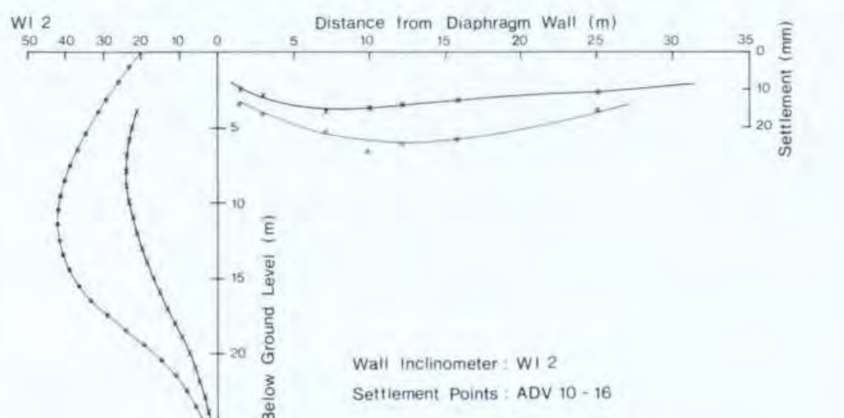

Fig. 9
Wall movement and ground settlement during excavation phase: Des Voeux Road

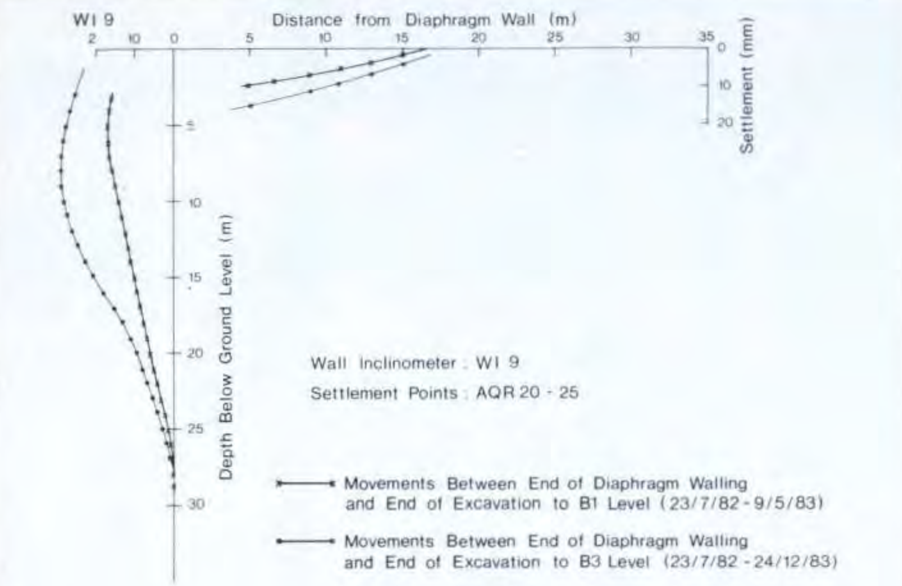

Fig. 10
Wall movement and ground settlement during excavation phase: Queen's Road Central

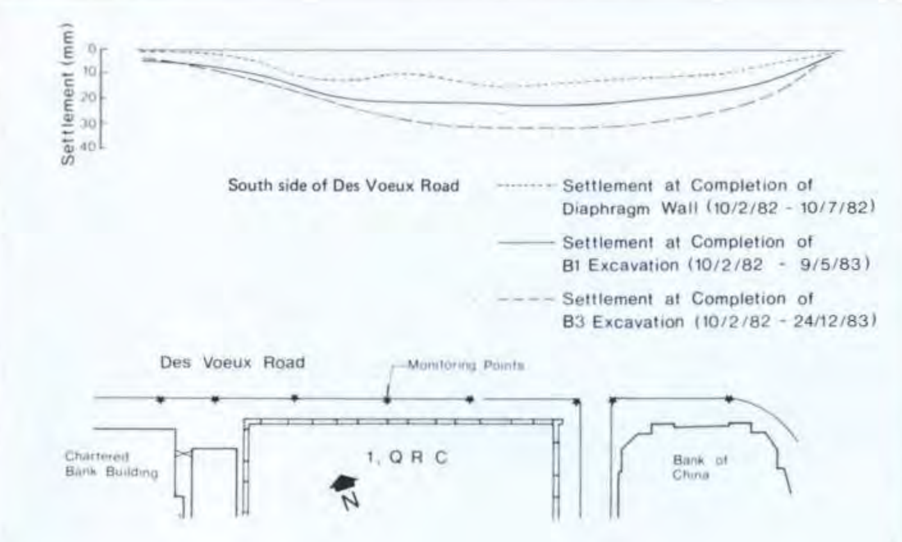

Fig. 11
Total ground settlement profiles: Des Voeux Road

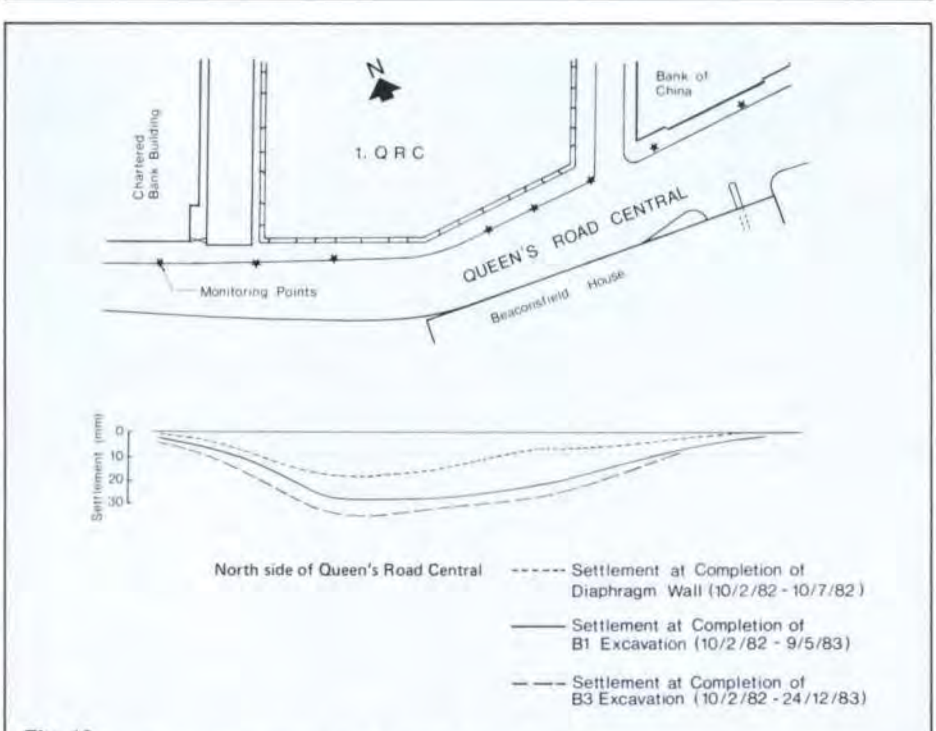

Fig. 12
Total ground settlement profiles: Queen's Road Central

volving precise levelling and the monitoring of tilt targets. The second contract was for the reading of ground water levels and the taking and processing of inclinometer readings.

The frequency at which any particular instrument was read depended on the works in progress at the time.

Measured movements

Fig. 8 gives the comparison between measured and predicted ground settlements due to diaphragm wall construction, and Figs. 9 and 10 show the measured lateral wall movements and ground settlements on north-south sections across Queen's Road Central and Des Voeux Road due to basement excavation. Figs. 11 and 12 show the total settlement profiles along the two roads. The significant reduction in actual diaphragm walling settlements compared to the predictions is principally due to the achieved excess slurry heads being generally greater than the specified 3.5m, particularly along Des Voeux Road where the head was maintained at close to 4.5m.

The eductor well points proved to be well suited to maintaining constant drawdown levels in the CDG, so allowing close control to be exercised on the excess slurry heads. With this system each well point is individually controlled, and by regular small adjustments and frequent reading of piezometers it was possible to limit drawdown variations to within 0.5m for the duration of diaphragm walling.

Comparison of Figs. 6 and 7 with Figs. 9 and 10 shows that maximum ground settlements during the excavation phase (including dewatering) were of the order of 60% – 70% of the predicted values. The lower actual values are partially attributable to the grouted diaphragm wall being a more effective water cut-off than had been allowed for in the analysis, and also the stiffness of the CDG was found to be generally higher than anticipated. It can be seen that the major part of the ground movements due to excavation occurred during the excavation to B1 Level.

The two adjacent buildings principally affected by the construction were the Chartered Bank and the Bank of China, each of which settled by a maximum of 15mm, with maximum tilts towards the 1, QRC site of approximately 1:2000. The total settlement of Beaconsfield House was between 6mm and 10mm. No distress or damage was recorded on any building.

Acknowledgement

The authors wish both to thank the Hongkong Bank for permission to publish this paper and to acknowledge the considerable efforts by their colleagues in carrying through the design of this project, for without teamwork such an undertaking is not possible.

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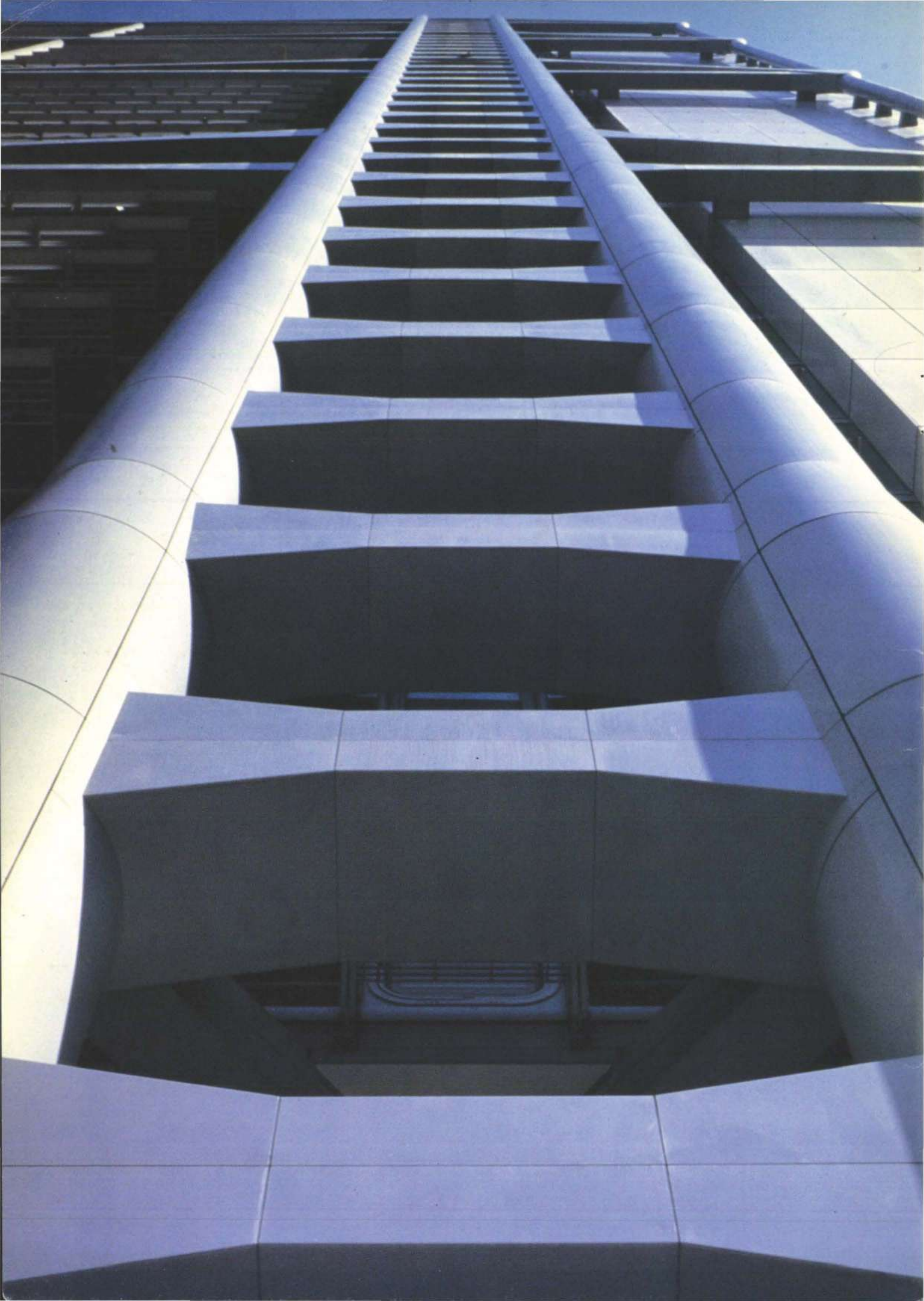

The derivation of overall design loads from the wind tunnel test results

Michael Willford

The structural design and wind tunnel tests were described in detail in the paper by Jack Zunz *et al* which appears earlier in this issue of *The Arup Journal*. This note, taken from reference 1, describes in more detail the way in which overall design loads on the building were derived from the wind tunnel measurements.

Characteristic loads and partial factors

Some of the problems that arise in attempting to make rational use of predictions of load effects derived from wind tunnel tests for structural design are discussed in reference 2.

In particular, decisions must be made on:

(1) How the reference or characteristic load effect should be defined

(2) What partial factor to apply to the chosen characteristic value to achieve an appropriate level of safety and reliability.

The conventional definition of the characteristic wind load (implied by *CP3: Chapter V* when used in conjunction with *CP114*, *BS449* and *CP110*) is that load which occurs with the 50-year return period wind acting in the most adverse direction for the member under consideration. Although convenient for code-based design, this definition has the disadvantage that the probability of the value being exceeded is not, in general, defined. When using the results of wind tunnel tests with a directional wind climate integration, alternative reference values can be chosen which are better defined statistically. Possible reference values are:

(1) The 50-year return period load effect (having 63% probability of being exceeded in 50 years)

(2) The expected maximum value of the load effect in a 50-year period (having, roughly, a 50% probability of being exceeded in 50 years).

The required partial load factor will depend both on how the reference value to which it is applied is defined and also on the level of safety required. Calibration with the level of safety currently embodied in *CP110* has led to the following relationship (Croft's equation 10).

$$R_u > 0.94\bar{S}\ (1 + 4.8\sqrt{(0.01 + 0.77v^2)}) \qquad (1)$$

where R_u is the required ultimate structural resistance (*CP110* definition)

$\bar{S}$ is the expected maximum value of the load effect

v is the coefficient of variation of S.

The greater the uncertainty and variability of S, the greater is the partial factor that should be applied.

Analysis of uncertainty and variability

The coefficient of variation of S can be considered in two primary components:

(1) The variability in the load effect due to the inherent variability of the wind climate and the random nature of wind loading

(2) Cumulative uncertainties in response prediction due to uncertainties in the accuracy of:

wind climate models
wind tunnel test simulations
structural properties
modal analysis.

The inherent variability of the wind process can be quantified by examining the relationship between a load effect and its predicted return period. As described in reference 2, when the relationship between a load effect and the logarithm of its return period can be taken as linear, the expected maximum value in a 50-year period can be calculated from the 50-year and 100-year return period values as

$$\bar{X} = 0.168X_{50} + 0.832X_{100} \qquad (2)$$

and its coefficient of variation from the slope, a, as

$$v = 1.282\ {}_{/a\bar{x}} \qquad (3)$$

where $1/a = (X_{100} - X_{50})/\mathrm{Log}_e 2$

The effect of inaccuracy in the modelling procedures cannot be quantified rigorously. However, one of the dominant unknowns is the total (structural plus aerodynamic) damping, and the effect of its uncertainty can be established by means of a parametric variation study. In this case the total damping was varied between an expected value of 1% critical and a worst credible value of 0.5%. The worst credible value is the worst value that the designer can realistically believe could occur, and here corresponds to the lowest values that have actually been measured for sway modes of buildings. A difference of three standard deviations is taken between the load effects calculated with these two values of damping enabling the coefficient of variation to be calculated.

A total coefficient of variation of 0.10 has been assumed for all the other sources of uncertainty.

Calculation of design load effects

Table 1 shows the expected maximum values (for a 50-year period) and the corresponding coefficients of variation for the overall base shear forces and torque on the building, calculated according to the principles described above.

The *ultimate* design load effects calculated according to Equation 1 are given in Table 2. For comparison, the *working* values obtained using the 1976 Hong Kong wind code are given, and also values X^*_{50} obtained from modal analysis of the wind tunnel test measurements applying the 50-year period wind in the most adverse direction (the traditional characteristic load definition).

In practice the torque is resisted by the masts almost entirely as opposing shear forces in the N – S direction on the two sides of the building. Further analysis, taking account of correlation effects between N – S shear and torque, showed that the effective N – S shear on each half of the building R'_u = 15.5MN, compared with R'_u = 11.75MN implied by Table 2 when torque is not incorporated. It should be noted that the aerodynamic shear forces and torque were found to be virtually uncorrelated in the force balance tests. However, within each mode the resonant shear and torque are clearly fully correlated. The responses from different modes were assumed to be uncorrelated.

The following may be concluded:

(1) The statutory design forces (working loads) are greater than the calculated ultimate loads, indicating there to be a substantial margin of safety for both strength and peak deflection. This is largely due to sheltering, both topographical and by surrounding buildings, the effects of which are ignored in the Hong Kong Wind Code.

(2) The ratio of ultimate load to the traditionally defined characteristic load is seen to be in the range 1.60 to 2.02 which is significantly higher than traditional partial load factors. The traditional approach can thus produce results that are neither consistent nor conservative when applied to the results of wind tunnel tests.

The following factors will have contributed to the higher partial load factors obtained in this case:

(1) Responses with a significant resonant component increase with wind velocity with an exponent greater than 2.

(2) The uncertainties in predicting dynamic response are greater than for static loads.

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A logical approach to the determination of cladding design pressures. Proceedings of the 3rd International Conference on Tall Buildings, Hong Kong and Guangzhou, December 1984.

Table 1

Load effect		$\bar{X}$	Coefficient of variation			
			wind	damping	modelling	total
E – W shear	(MN)	6.24	0.19	0.092	0.10	0.23
N – S shear	(MN)	12.7	0.17	0.031	0.10	0.20
Torque	(MNm)	142	0.23	0.092	0.10	0.27

Table 2

		Ultimate load	Working loads	
Load effect*		Equation (1)	HK Code	X^*_{50}
E – W shear	(MN)	12.3	23.3	6.1
N – S shear	(MN)	23.5	37.9	14.7
Torque	(MNm)	297	–	152

Photo: Ian Lambot

Development and use of a cementitious barrier coating

Turlogh O'Brien
Allan Marsden

Introduction

The corrosion protection problems presented by the design of the Hongkong Bank headquarters building have been described in the paper by Zunz *et al* elsewhere in this issue.

The design constraints resulted in a situation in which many of the external members could potentially remain damp or wet indefinitely. Moisture could be trapped in the ceramic fibre blanket fire protection during construction or collect from condensation occurring later. Maintenance will not be practicable during the design life of the structure.

Previous experience suggested that the most appropriate form of corrosion protection would be cementitious, since the alkaline nature of the protection provided by cementitious materials is not hindered by the presence of moisture. Paints and other barrier coatings are noticeably less effective in these circumstances. No known paint scheme could be relied upon to give a 50-year life in these conditions, particularly in those areas where the corrosion protection would, of necessity, have to be applied on site after welding.

Development

It was concluded at an early stage that a conventional 50mm of concrete was not practicable for this structure. A decision was therefore taken to consider a cementitious barrier coating consisting of (nominally) 12mm of modified cementitious mortar. Preliminary trials were carried out at Wimpey Laboratories in 1981 to evaluate a number of alternative means of achieving this. The tests carried out in this trial are shown in Table 1.

The results of these trials were sufficiently encouraging for it to be accepted that the concept was practicable. The most suitable material appeared to be a gun-applied, SBR polymer-modified mortar, incorporating melt-extract stainless steel fibre reinforcement. Application by gun was essential for large-scale use on complex surfaces. It also gave a dense, well-compacted material, but introduced the problem of overspray.

Table 1:
Testing of Wimpey Laboratories trial mixes

(1) Spray up trials with the selected materials onto specially fabricated test samples
(2) Determination of coating thickness
(3) Density of applied material
(4) Total water absorption
(5) Impact testing
(6) Bend testing
(7) Water vapour permeability
(8) Air permeability
(9) Drilling test
(10) Effects of heating the material
(11) Macroscopic examination
(12) Spray up onto mock-ups of complex structural elements.

Stainless steel fibres were added only to the lower 6mm of the overall thickness, in order to strengthen the coating both for the transit conditions and in-service stresses. The use of the SBR polymer significantly reduced the permeability of the product, allowing it to be used much thinner than would be necessary with concrete. Thus the material used brought together a number of established concepts in a system which is more obviously a variant of a concrete repair method.

A further set of trials were carried out in 1982 at the Teesside Laboratories of the British Steel Corporation, enabling a clearer delineation of the properties that might be expected from the production material. These tests also confirmed that the material could not satisfactorily be applied to a painted steel surface, as adhesion was inadequate. Since a holding primer was to be used to prevent corrosion of the steel in transit, it had to be accepted that re-blast-cleaning would be required in Hong Kong.

Following these tests a specification was prepared for tender purposes which defined the properties required of the final material (Table 2) and the raw materials to be used (Table 3) but with only a limited set of requirements for the application of the material. The aim of this was to allow a specialist subcontractor to propose the application techniques that he, with his specialized knowledge, considered would be most suited to the task of achieving the properties required.

It was envisaged that a three-month period would be set aside between award of subcontract and commencement of application in which such techniques could be refined and perfected and, if necessary, the specification altered.

In the event, the three month system trial period had to be drastically curtailed and was eventually merged with trials carried out to demonstrate that certain local sands, which did not comply with the specification, nonetheless produced a satisfactory end product at lower cost. These trials took place in Hong Kong in late 1982. They were generally successful, except that it became clear that the specified adhesion value could not be achieved consistently. It was felt, however, that the risk involved in reducing the allowable adhesion to 1.0 MPa was acceptably low. This change was made, and the local sands were accepted.

Application

The application of CBC took place at two locations, the off-site yard at Junk Bay and on site after erection. The material was applied to (virtually) all areas of primary structure externally, and to those areas most readily coatable internally. The decision to use the material internally was an economic (and programme) choice, since the material is a considerable overspecification in these areas in terms of corrosion protection. The total area of steel which was coated was of the order of 48,000m². It was originally envisaged that 80% would be coated off-site, and transported to site in the coated condition. The handleability of the material proved excellent, and very little transit damage occurred. Where small areas of damage did occur, or where small areas of unacceptable material were found and cut out, repair was carried out using a hand-applied epoxy mortar.

On site, the areas left clear of material for site welded joints, for temporary attachments, for crane supports and other reasons were prepared by blast cleaning prior to the CBC application. Any large defects in the off-site material were cut out and treated in a similar way.

Table 2:
Property requirements at tender

Property	*Requirement*	
Thickness	≮12mm	≯20mm
Adhesion	≥2.5 MPa when measured by pull-off of cored disc (using Elcometer adhesion tester)	
Vapour permeability	≯15 g/m^2/day at 75% RH and 25°C (using modified version of test in *BS 3177*)	
Chlorides	≯0.05% Cl^- by weight of cement	
Strength	Capable of withstanding 8kg on a 50mm Ø ball dropped over 2m without visible damage or delamination	
Cracks	No cracks >0.1mm width	

Table 3:
Materials to be used in cementitious barrier coating, as at tender

Material	*Description*
Cement	OPC to *BS12*
Sand	Zone 3 washed natural sand to *BS882*, 100% passing 5mm sieve
Fibre	Melt extract type 430 stainless steel fibres, 12mm nominal length
Dry materials to be supplied pre-batched and mixed in proportion 3:1 sand:cement, with 5% by weight fibres added to mix for first layer.	
SBR	*Ronafix* from Ronacrete Ltd. or equivalent approved
Water	Mains water, tested to *BS3148:1980*
SBR and water to be mixed 50:50 by volume. No other additions allowed.	

Table 4:
Typical Figg test results

Sample	*Results*	
Junk Bay CBC	1000-1600 sec	
Site CBC	approximately 1000 sec	
	Time(s)	Type of material
Standard interpretations	less than 30	porous mortar
	30-100	20 MPa concrete
	100-300	30-50 MPa concrete
	300-1000	Dense, well compacted concrete
	1000 +	Dense polymer modified concrete

Table 5:
Typical water vapour permeability results
(Full thickness specimens)

Sample	*Result*
CBC	0.2-5 g/m^2/day
Reference concrete (50mm thickness)	15-30 g/m^2/day
Specification	≯15 g/m^2/day

QA-QC

Due to the relative thinness of the coating compared with conventional concrete, it was clear that a high level of quality assurance would be required, especially since the programme and access arrangements precluded intensive quality control on the final product. Inspection and testing were carried out at all stages from the barge carrying the sand, through the batching plant on to the actual spraying of the material. The most difficult area was testing the as-sprayed material, which, of course, had to be carried out either non-destructively, or on separate test pieces. In situ testing consisted of visual inspection for defects, thickness check using an Elcometer coating thickness guage, and a hammer-tap inspection for loss of bond.

Tests on sprayed test panels had necessarily to be carried out quickly (before the relevant elements were covered with fire protection, etc.), and in order to gain a rapid impression of the permeability of the material, a modified version of the Figg air permeability test was developed and used[1], together with a number of more conventional tests, such as crushing strength and pull-off adhesion. Sample permeability results are given in Tables 4 and 5.

Conclusion

The design constraints on the project created an extremely challenging problem for the corrosion engineer, with no established protection system wholly satisfying the requirements. By modifying and extending a technology previously applied in the repair of reinforced concrete, however, a system has been developed which has given the required assurance of protection for the life of the structure in an aggressive environment.

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Improvements to the Figg method for determining the air permeability of concrete, by R. Cather, J.W. Figg, A.F. Marsden and T.P. O'Brien. *Magazine of Concrete Research, 36* (129), pp.241-245, 1984.

Photo: Ian Lambot

Project planning

Peter Bolingbroke

Our Project Planning Group first became involved in the project in November 1979, when they were asked to estimate the construction times for various development options being considered by Foster Associates. This information was later incorporated in a report which outlined 10 strategies for developing the site and indicated the timescales and costs applicable to each. By the end of January 1980, the client had chosen a development option which provided a public banking facility in an adjoining Annexe before the 1935 building was demolished and retained this facility until the new building was open.

Vacation and conversion of the Annexe

When the basic development strategy had been decided, the Group was asked to collaborate with representatives of the Bank's property and operational departments in preparing a programme for vacating the Annexe, so that a date could be established for the conversion work to start. As this involved the relocation of a number of departments and a complex series of moves, a critical path flow diagram was used to analyze the situation and this identified 30 November 1980 as the earliest date by which the Annexe could be finally cleared.

By mid-February, Fosters had prepared five outline schemes for converting the premises to a small clearing bank and PPG had drafted indicative construction programmes for each. These programmes showed that the time required for carrying out the site works would range from seven months to one year, depending on which conversion scheme was chosen. From discussions which followed in the next two months it was apparent that the client favoured the scheme with the shortest construction time, so a detailed study was made of the overall programme, including design and pre-ordering times, to test its feasibility with regard to conversion work starting immediately the Annexe was vacated. These studies confirmed that the selected scheme could be accomplished between 1 December 1980 and 30 June 1981, provided that certain long-delivery items were placed on order in June 1980 and that a considerable amount of overtime and double-shift working was permitted during the construction phase.

Vacation and demolition of the 1935 building

On the assumption that the converted Annexe would be ready for occupation by July 1981, the client then commissioned a programme study to see whether all departments housed at 1 Queen's Road Central could be relocated by that date, thus clearing the way for demolition of the 1935 building. Working in close collaboration with the departments concerned, a plan was formulated for transferring all personnel to other offices in Hong Kong prior to July 1981, except for the banking hall staff who would move into the Annexe as soon as the conversion works were complete. Finally, the six months period which had been assumed for demolition of the 1935 building was confirmed by a local contractor, so it now seemed possible that construction of the new building could begin in January 1982.

Construction studies

From February to May 1980, various construction methods and sequences were investigated, with the object of establishing a programme for the new building being operational in 39 months, the timescale first identified for this development option. The initial studies were based on a preliminary design which included a substructure 10m deep and a superstructure which rose some 30 storeys above ground level. It was also assumed that the structural steel frame would be featured externally and that high-quality cladding would be applied to all elevations. Finally, it was apparent that sophisticated services systems would be installed throughout the building and that specialized finishes would be required in several areas.

To enable a building of this nature to be constructed in the shortest possible time, a structural design concept had already been adopted whereby erection of the superstructure could proceed in parallel with basement construction. Furthermore, it was an agreed strategy that many of the building services and internal finishes would be designed as prefabricated units for speedy installation on site. Based on these criteria, an outline programme was drafted for the major building elements which identified that erection of steelwork should start some 2½ years before the opening date, to allow sufficient time for the cladding, fit-out and commissioning operations which followed. It was evident, therefore, that the total construction time for the new building would depend on two factors; namely, the date when the first steel sections would arrive on site and the time required to complete substructure elements essential for the start of steel erection.

With regard to delivery of steelwork, it was estimated that the first consignment would reach Hong Kong by January 1983, about one year after site work had started. This estimate was based on a steelwork contractor being appointed by the end of 1981. When reconsidering the programme for substructure works in April, it was assumed that the lowest basement level would be 15m below Queen's Road Central. This was 5m deeper than the original scheme, but was now thought necessary to accommodate all facilities planned in the basements. The programme showed that a period of 13 months would be required to complete the works which preceded the start of steel erection, thus indicating that the minimum construction time for the new building would be 3½ years.

Target master plan

In mid-May 1980, a preliminary design concept for the new building was formally presented to the board and general managers of the Bank with the aid of slides and a model. At the same time, a target master plan was displayed to illustrate the proposed sequence and timing of the main development stages prior to opening the new building and removing the Annexe (See Target master plan flow chart).

This plan incorporated all programme findings to date and it highlighted the strictly sequential nature of the preliminary works which preceded construction of the new building. Hence, it was stressed that delay in completing any of these works would have a 'knock-on' effect on all subsequent activities. It was also pointed out that the main building programme had been extended to 42 months to accommodate the increased depth of substructure required for the recommended basement scheme. Finally, it was emphasized that occupation of the new building in July 1985 was largely dependent on steel erection starting by February 1983; an important milestone in the master plan.

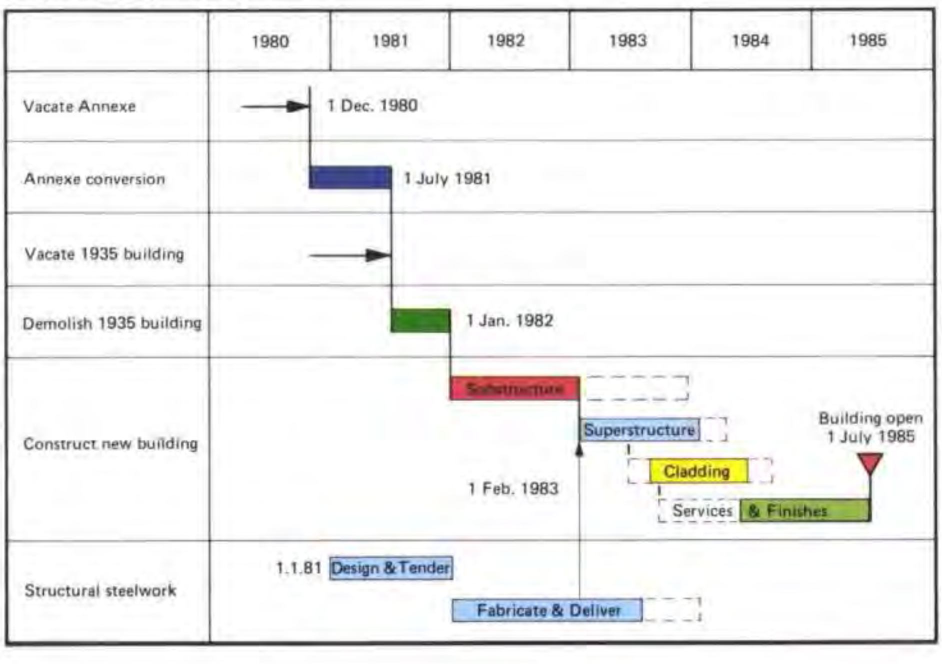
Contractors' programmes

When the six joint-venture contractors were briefed in September 1980, prior to tendering for management of the construction operations, the target master plan was again displayed. On this occasion, the construction schedule for the main building works was shown as 1 January 1982 to 30 June 1985, with only the key date for first delivery of steelwork being identified. This left the contractors free to formulate their own methods and programmes for executing the works prior to and following the start of steel erection, but it was pointed out that the structural design would permit the superstructure and basements to be built in parallel, if required.

It was interesting to compare the various construction methods and programmes proposed by the contractors when their formal submissions were received. For example, two contractors indicated that the whole basement structure would be complete prior to the start of steel erection, one contractor considered that completion of the main building works by July 1985 was over-optimistic, while another felt that the new building could be opened 6 months earlier than scheduled!

Having analyzed the contractors' submissions and interviewed their proposed management teams, the John Lok/Wimpey joint venture was selected as management contractor for the project in October 1980. From that time, all construction planning

Target master plan May 1980



and programming was carried out by the management contractor's staff, both for the new building and for the Annexe conversion works.

Final design concept

In January 1981, formal presentations of the final design concept for the new building were made to the development team, general managers and all members of the Bank board. The presentations covered architectural, structural and services aspects of the building, together with programme and cost matters.

With regard to the overall project plan, the key dates for conversion of the Annexe, demolition of the 1935 building and construction of the new building remained the same as those identified in May 1980. On this occasion, however, the construction methods, sequence and programme proposed by the management contractor for the main building works were described in some detail by a series of slides. The presentation also included an estimate of the time involved in removing the Annexe after the new building opened in July 1985 and completing the remaining portion of the main building on the western boundary of the site.

As the infill works would take place in close proximity to users of the new banking hall and would occupy a period of some 15 months, the client requested a study on earlier removal of the Annexe, so that construction of the new building could be achieved in a single phase. In the last week of January, a report was prepared which compared four development options for the Annexe area (in terms of cost, additional floor area and project completion date) related to various periods for retaining temporary banking facilities on the site. On the basis of this report, the client decided that continuity of construction for the new building should be maintained to July 1985, by removing the Annexe not later than July 1983.

Pre-tender programmes and decision schedules

As the management contractor developed detailed construction programmes for the new building and identified dates for tendering the various sub-contract packages, a pre-tender programme was formulated for each package by PPG in consultation with the relevant members of the design team. These programmes were used for regular progress monitoring of the design and billing procedures, and monthly trend analysis reports were issued for many packages, relating to services and finishing elements, which were subject to a complicated series of approvals by the client and building users.

In parallel with the preparation of pre-tender programmes, a schedule of client decisions and approvals was issued and updated at regular intervals, so that the whole project team was aware of the actions necessary to achieve the out-to-tender date for each subcontract package.

Between April 1981 and September 1984, some 200 pre-tender programmes were formulated and more than 40 issues were made of a schedule of client decisions. Until August 1983 this work was done by the UK staff, but for the remainder of the time was undertaken by members of the Hong Kong office, as the design team was then established in the colony.

TOP Programmes

In September 1984 the Hong Kong staff were asked to assist in the preparation of programmes for obtaining Temporary Occupation Permits for sections of the new building, so that the Bank's departments could move in progressively during the period July to November 1985. The programmes were developed in network form as they had to combine numerous activities and complex interrelationships involving all public utilities, the Fire Service Department, the Building Ordinance Office, the design team, the management contractor and those responsible for commissioning the building services.

Once a week the programmes were reviewed in detail and prompt corrective action taken whenever an activity fell behind schedule. By October 1985 this tight monitoring system had resulted in TOP's for zones 1-5 being granted on schedule and occupation by the Bank personnel proceeding as planned.

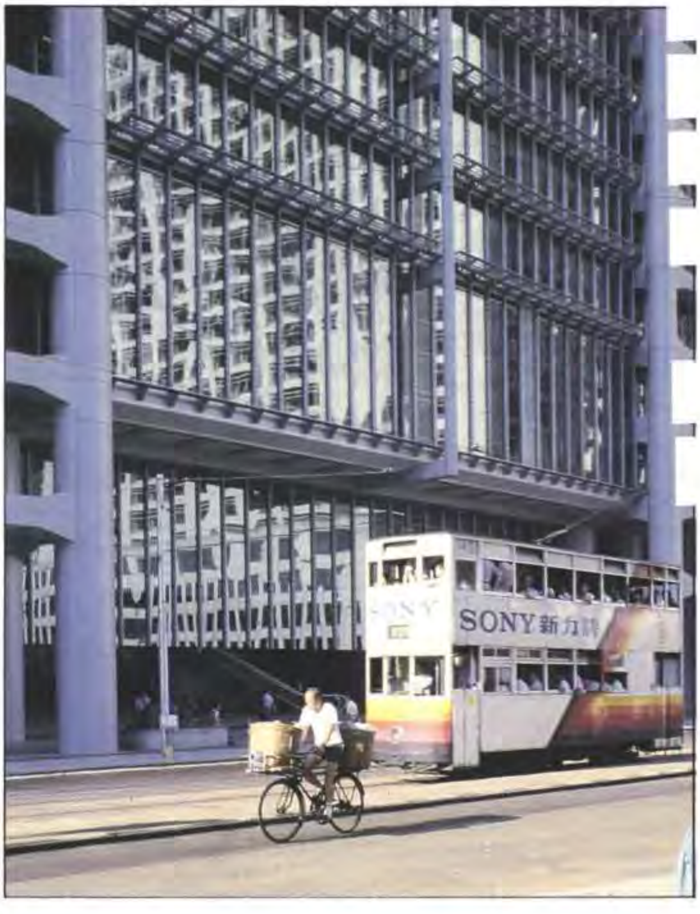
Achievement of plan

It is always interesting to compare plan with achievement at the conclusion of a large and complex construction project. In the case of the new headquarters for the Hongkong Bank, a comparison shows that the progress position at each stage of the development was virtually in line with the target dates set in May 1980 (See table).

Even by Hong Kong standards, this was a notable achievement by all concerned.

Progress positions: comparison of target dates with actual dates

Stage	*Target date*	*Actual date*
Annexe vacated	1.12.80	1.12.80
Conversion of Annexe complete and 1935 building vacated	1.7.81	26.6.81
1935 building demolished	1.1.82	December '81
Start construction of new building	1.1.82	13.1.82
Start erection of steelwork	1.2.83	27.1.83
New building open	1.7.85	1.7.85 (Zones 1-3)



Photos: Colin Wade

Seawater intake system

1 Excavation of Shaft no. 2 at Star Ferry
2 Upper half of Shaft no. 2
3 Drilling probe holes ahead of the face to detect water
4 Tunnel excavation
5 Mobile shutter for concrete tunnel lining
6 Lined tunnel with pipes and cables installed
7 Seawater pipes at base of Shaft no. 2
8 Shuttering and reinforcement at the tunnel's junction with Shaft no. 2
9 Flanged joint and spacer for valve installation on seawater main
10 Base of Shaft no. 2
11 Seawater intake chamber with twin cup screens

1△

2△

5△

6▽

3△

4▽

Arups' project team for the seawater system

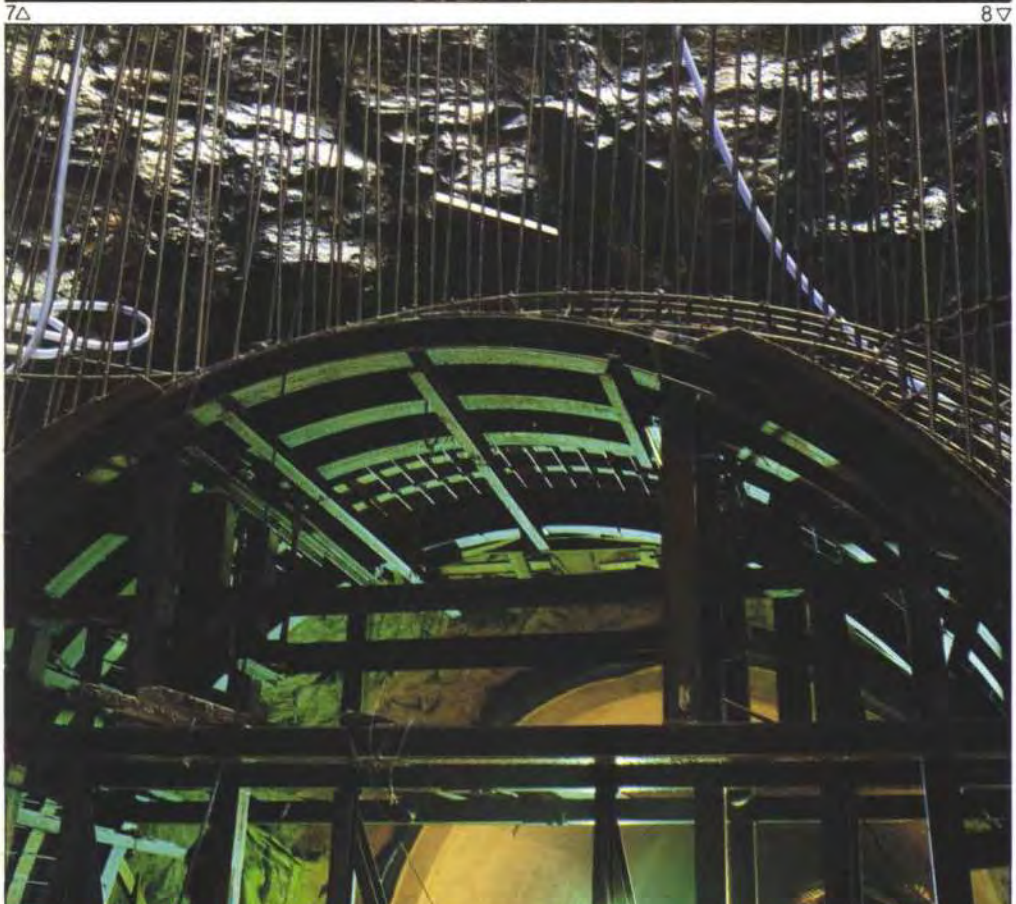
Warren Beynon
Mark Bidgood
David Butler
Peter Chan
Paul Fowler
Naeem Hussain
Alan Kemp
O.Y. Kwan
Lindsay Murray
Simon Murray
Chris Nunns
Douglas Parkes
Brian Pinkerton
Viv Troughton
Colin Wade
Can Wong
W.P. Yeung

Photos:

1, 2, 3 & 9: Simon Murray
4, 5, 7, 8 & 10: Sam Liu
6: Wimpey Construction
11: Colin Wade.

7△

8▽



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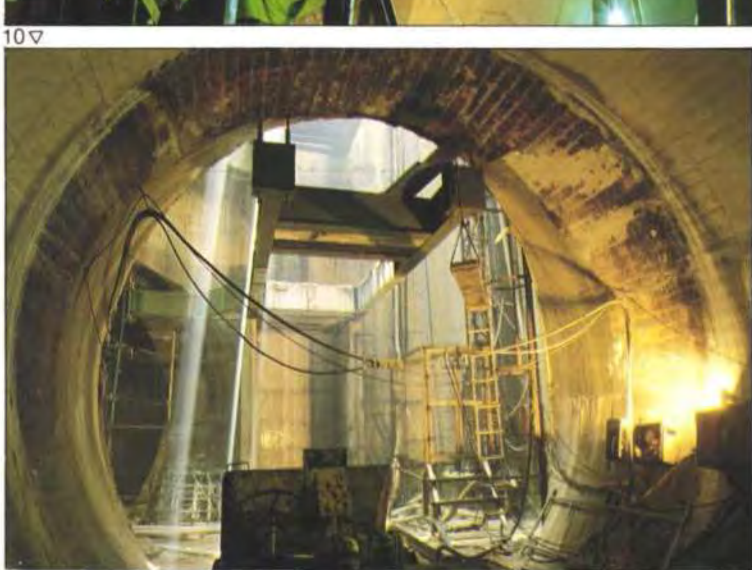

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