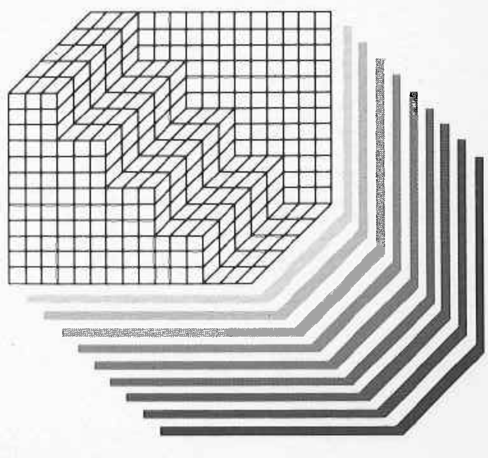
Patterns 9



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Contents

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Tsim Sha Tsui Cultural Centre – The Design Concept

Project data

Client	Building Development Department, Hong Kong Government (BDD)
Architect	Architectural Services Department, Hong Kong Government (ASD)
Civil/Structural Engineers	
(Phase 2)	Ho-Happold
(Phase 1)	Engineering Services Department, Hong Kong Government (ESD)
Services Engineers	BDD Building Services Section
Main Contractors	
(Foundations)	Soletanche-Bachy (HK) Ltd
(Superstructure)	Kumagai Gumi (HK) Ltd
(Site Manager)	David Westwood
Quantity Surveyors	BDD Quantity Surveyors Section
Total Cost	£35m
Completion Date	Summer 1988

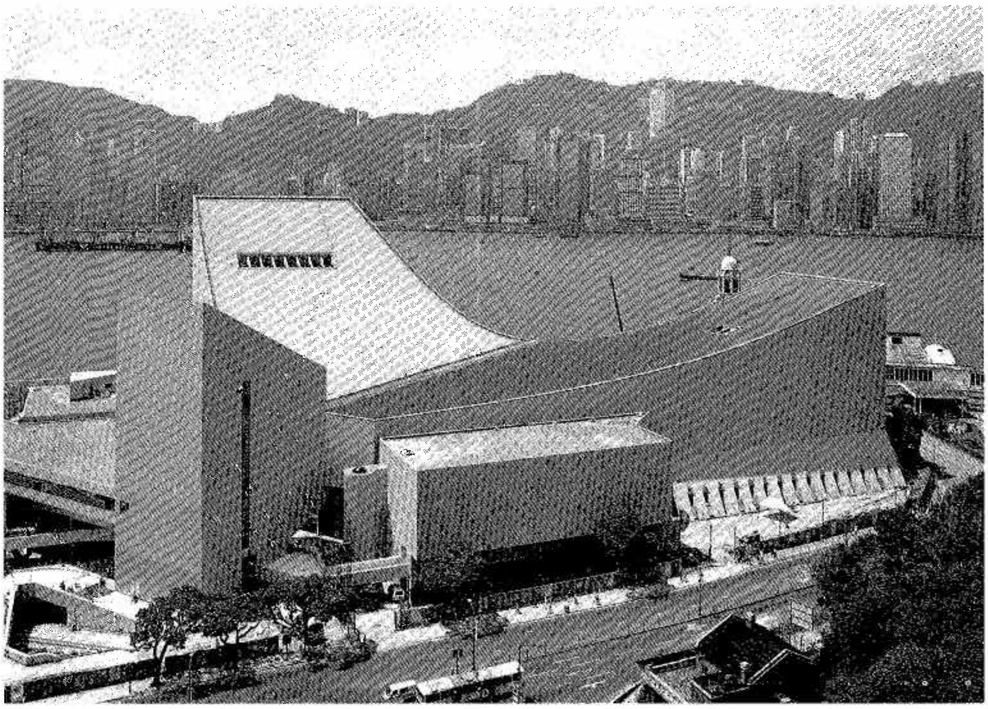


Fig 1.1 Tsim Sha Tsui Cultural Centre at tip of Kowloon peninsula, Hong Kong

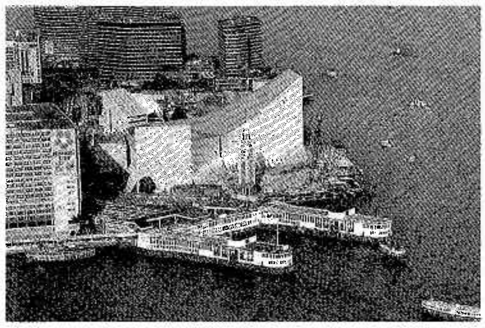


Fig 1.2 Cultural Centre on mainland peninsula separated from Hong Kong Island by one mile wide deep water harbour

This issue of Patterns is devoted in its entirety to describing the inception, design and construction of the winning entry in the structural competition for the Tsim Sha Tsui Cultural Centre, Hong Kong. Completed in 1988 and officially opened by the Prince of Wales in October 1989, the Cultural Centre lies at the southern tip of the Kowloon peninsula, within Hong Kong Harbour, on the site of the former terminus of the Kowloon-Canton Railway (Fig 1.1). Occupying a floor area of approximately 30,000m^2, it comprises a 2100 seat concert hall and 1750 seat lyric theatre, separated by a multilevel central foyer, with linkage to a 450 seat studio theatre. The building structure has a number of unusual and novel features: barrette foundations were used for the first time in Hong Kong; the structure has no columns; and large open spaces are achieved by the use of deep beam walls, long span floor and roof construction, and large cantilevers. The roof, possibly for the first time, is formed from a series of three hung concrete rigidised lattice cable nets, so as to be able to resist the extremely high wind suction forces associated with the typhoon winds of the South China Seas. The articles of this journal, originally published in the Proceedings of the Institution of Civil Engineers (Ref 1.1), separately consider the overall design concept; the design of foundations and superstructure; and the design and construction of the lattice shell roof.

In the early 19th century Britain wished to acquire a trading post in China, and finally to establish peace in the region agreed to take the small hilly island known as Hong Kong. Hong Kong Island is 26 square miles in area and separated from the mainland at Kowloon Peninsula by a one mile wide channel providing a huge deep water harbour (Fig 1.2). In January 1841 a British naval party took possession of the Island, land lease auctions were held and settlement began. Some 20 years later, after a further conflict with China, the British secured control of the harbour by taking the southern tip of the Kowloon Peninsula up to Boundary Road and Stonecutters Island. Forty years further on they secured a lease (for 99 years from 1898 to 1997) on the rest of the Kowloon Peninsula, some further mainland territory, and all the islands immediately surrounding Hong Kong. Acquisition of these new territories increased the size of the colony to 390 square miles, but Hong Kong Island remained the main business and commercial centre.

Hong Kong was primarily a trading town, never as successful as Shanghai, but steadily growing and prospering. Wharves fringed the edges of the island, with warehouses and offices facing the sea, while Government and public buildings, defence quarters, the Hong Kong Club and the racecourse formed the town proper, and summer residences developed on the Peak. Kowloon, where the first settlers had played cricket, housed service industries such as the dockyards, and the majority of the Chinese working force. However, trade with China was the purpose of the colony and the centre of trade was the harbour, the edges of which were reclaimed land. Hong Kong Island was linked to Kowloon by a ferry service (taken over by the Star Ferry Company in 1898), which in turn was linked to the Kowloon Canton railway station in 1911.

Although there was some conflict between the traders and the government, public works started almost as soon as the colony was founded. The first Surveyor General was Alexander Thomas Gordon – there were six between 1843 and 1889, when the Public Works Department (PWD) was established. This department had control of planning, leasing and approvals for an entire city, state-offices and warehouses along the seashore, a town hall, two cathedrals, court buildings, a commercial centre, barracks, hospitals, and villas and bungalows rising up the hills. By 1939 it had 77 subdepartments, with 134 European and 513 non-European officers, and administered land control and sale, all civil engineering, and government and defence buildings. Interest was more in sport than culture, and there was a dearth of theatres and art galleries which would have been built by that stage in a similarly developing British city.

Hong Kong was occupied by the Japanese during the war, and the immediate post war period was largely spent by the PWD in rehabilitating damaged buildings. The Chinese Communist revolution of 1949 changed the colony, leading to an influx of refugee industrialists into Hong Kong followed by a western boycott of Chinese goods. A seemingly endless flow of refugees from the Chinese mainland provided cheap labour, and helped to transform the colony from an economic appendage of China into an immense manufacturing and financial centre. Its population, estimated at 2.4 million in 1955, reached 5.6 million by 1988. Leasing of land sales yielded considerable finance, but the pressure on public works such as water supply and treatment and roads was enormous. Housing became a major problem, and the Housing Department and Housing Authority was set up as a separate body in 1973. Its achievement in providing an enormous number of units on an extremely difficult terrain is itself well worthy of study.

A cultural centre for Hong Kong

The subdepartments of town planning and architecture were combined until 1982, and it was probably this close relationship which led to the development of the Tsim Sha Tsui Cultural Centre. The main sea frontage of the Kowloon peninsula opposite the island had been filled to provide a terminal for both the Star Ferry and the Kowloon-Canton railway station, allowing direct transfer. The decline in traffic with China led to the station being moved further up the peninsula, thus freeing a part of the foreshore for possible extension and development. It was felt that at least the section along the foreshore opposite the island and between the Star Ferry terminal and Kowloon itself should become public space.

In 1974 a plan was developed for a cultural centre, incorporating auditoria for concerts and theatre, a science/technology centre and a museum of art with library and other facilities. The whole was envisaged as an urban park (Fig 1.3). People coming from the Hong Kong side on the ferry would be able to walk directly into Kowloon past the old clock tower of the railway station and through the centre of the auditoria complex, or more slowly along the waterside walks and gardens (Fig 1.4a, b).

The first stage of this development was the space museum, conceived in 1974 and completed on its site in 1980. There are basically two elements in this building, to the east an egg-shaped dome covering an exhibition hall and workshops, and to the west a 200 seat lecture hall, a solar hall with a snack bar and shop. It had been intended to follow the design and construction of the space museum immediately with that of the auditoria building. Considerable architectural and technical design work had been carried out, but the financial climate was such that this was postponed. The architects for the complex were the Architectural Services Department (ASD) of the Hong Kong Government. Under the direction of Jose Lei, the administration building, restaurant block, underground car park and plant rooms (Phase 1) were designed by the Engineering Services Department (ESD). However, when the project was reactivated in 1981, it was decided to put the civil/structural engineering consultancy for the lyric theatre, auditorium and studio theatre (Phase 2) out to international competition. This was widely publicised and a large number of consultants from all over the world submitted experience profiles in the hope of being chosen to compete. The list was then reduced to seven competitors all of worldwide reputation, and the architects' previous plans and sections were issued, with the form, but not the structure, roughly defined. All submitted their structural solutions to the problems; two practices were then selected and interviewed in depth before the final selection. Ho-Happold, a partnership of Hong Kong born structural engineer Tom Ho and Buro Happold, were the winners and were commissioned to act as civil/structural engineering consultants.

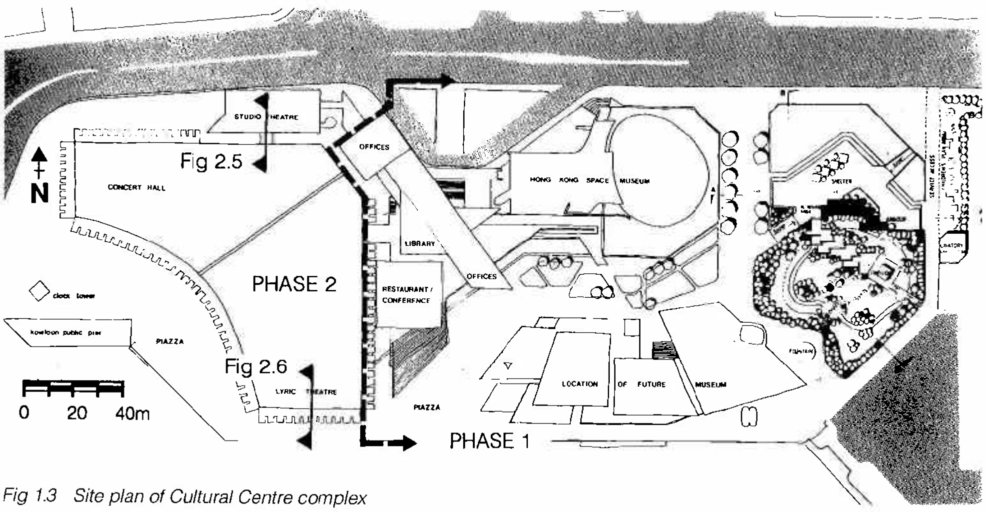


Fig 1.3 Site plan of Cultural Centre complex

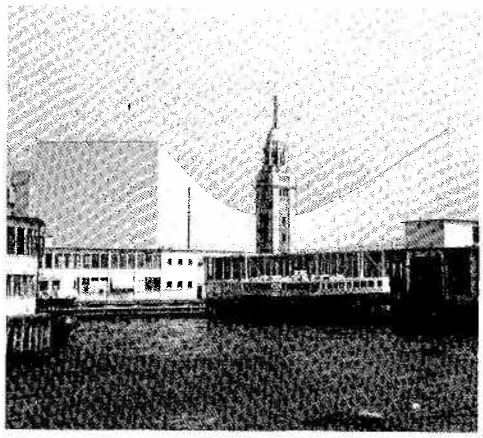


Fig 1.4 (a) Hong Kong ferry terminal in front of complex

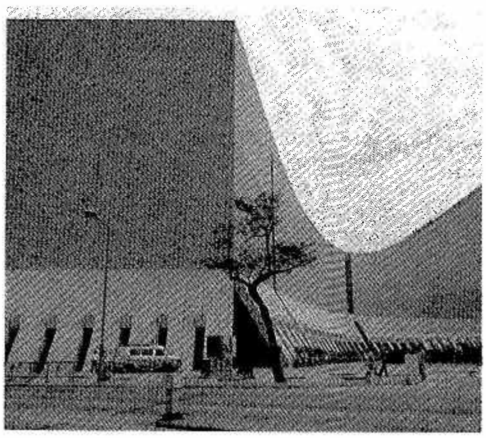
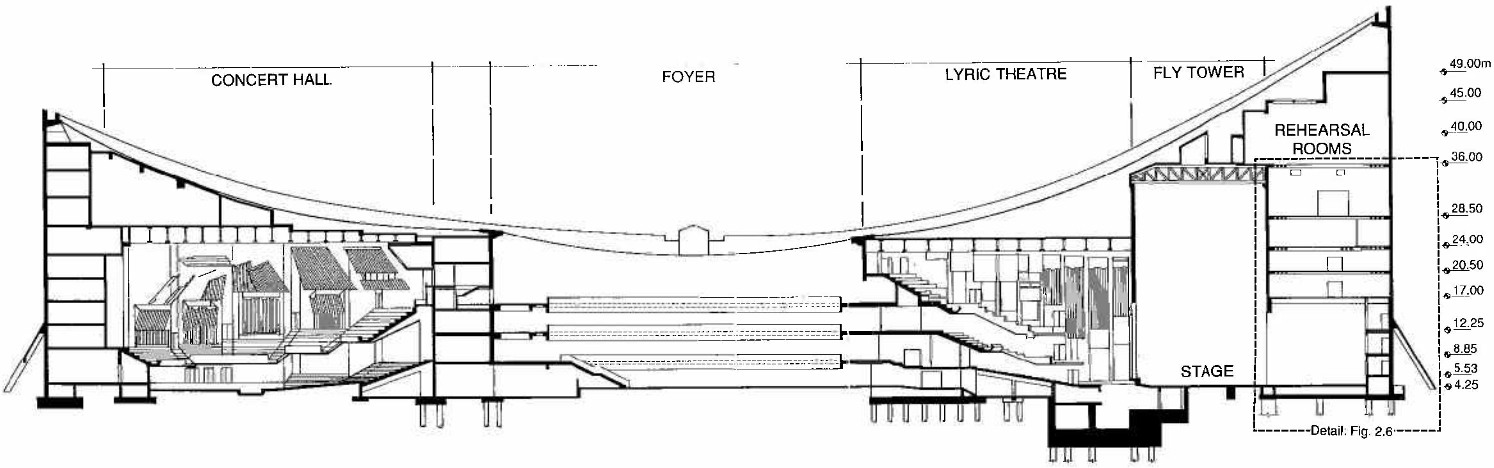


Fig 1.4 (b) Old clock tower of railway station on Kowloon peninsula, adjacent to complex

The design concept

Two approaches are possible to the working relationship between architects and structural engineers. One is the evolution of a design by a partnership between architects and engineers in which, it is hoped, through striving for a breadth of values and developing a joint holistic view of 'the building problem', a balance of symbolism and function will be negotiated and achieved. The alternative is the provision of structural design possibilities by engineers to fit the design determined by the architects' definition of 'the building problem' a kind of sympathetic 'tailoring' to fit a body rather than the evolution of the body itself. For this building it was the latter approach which was appropriate. The ASD's design for the Cultural Centre was treated as entirely fixed – the requirement was for its safe and economic achievement.

The ASD's plans feature a 2100 seat concert hall and a 1750 seat lyric theatre as the main elements on either side of a large foyer providing a passageway from the ferry terminal to the centre of Kowloon (Fig 1.3). A vast curved roof encompasses



the whole and provides cover for the stage towers (Fig 1.5). The whole complex is some 30,000m^2 in area, and includes not only the two main halls and foyer, but also a 450 seat studio theatre, rehearsal rooms, stages, exhibition rooms, conference rooms, press rooms and ancillary and public spaces. The Ho-Happold entry aimed to achieve a column-free construction using only reinforced concrete walls, beams and slabs for the main superstructure, and because the profile of the roof as drawn was almost parabolic, a tensile cable-net roof could be provided. This aspect of the entry was not intended to show a final solution, but rather to express how the design could be evolved further to achieve a complex, tailored roof shape which might reflect the uses of the spaces within.

This approach of tailoring the structure to fit is appropriate for this building because what the engineer is striving to achieve is economy – seen here as a minimum weight of material for the roof, achieved by carrying the load in tension. As H J Cox (Ref 1.2) proved, it is not only more efficient in the use of materials to carry a force in tension rather than in compression or bending, it is also most efficient to divide this load into as many tendons as possible. Thus, providing a doubly curved cable-net roof over the entire building is extremely effective and economic, especially when the box of the building is able by itself to provide an economic anchorage to the 'tension' forces.

Structural solution

The stiff support structure, essential to resist the tensions at the edges of the roof, is readily provided by the superstructure of the concert hall and theatre zones which already contained many internal stiffening walls and floor plates to buttress a large proportion of the external walls. The problem was that while the overall principle might have been correct, the approximate parabolic profile of the roof as conceived by the architects was not possible to follow with cable. As shown in Fig 1.6, the roof surface is divided into two distinct parts by a linear skylight above the foyer. The building interior however, is divided into the three distinct zones of structure. The theatre and concert hall areas comprise floors supported by stiff walls and deep beams, whereas the foyer area is much more open-plan and relatively free of vertical support. These

Fig 1.5 Section through Cultural Centre showing concert hall, foyer and lyric theatre

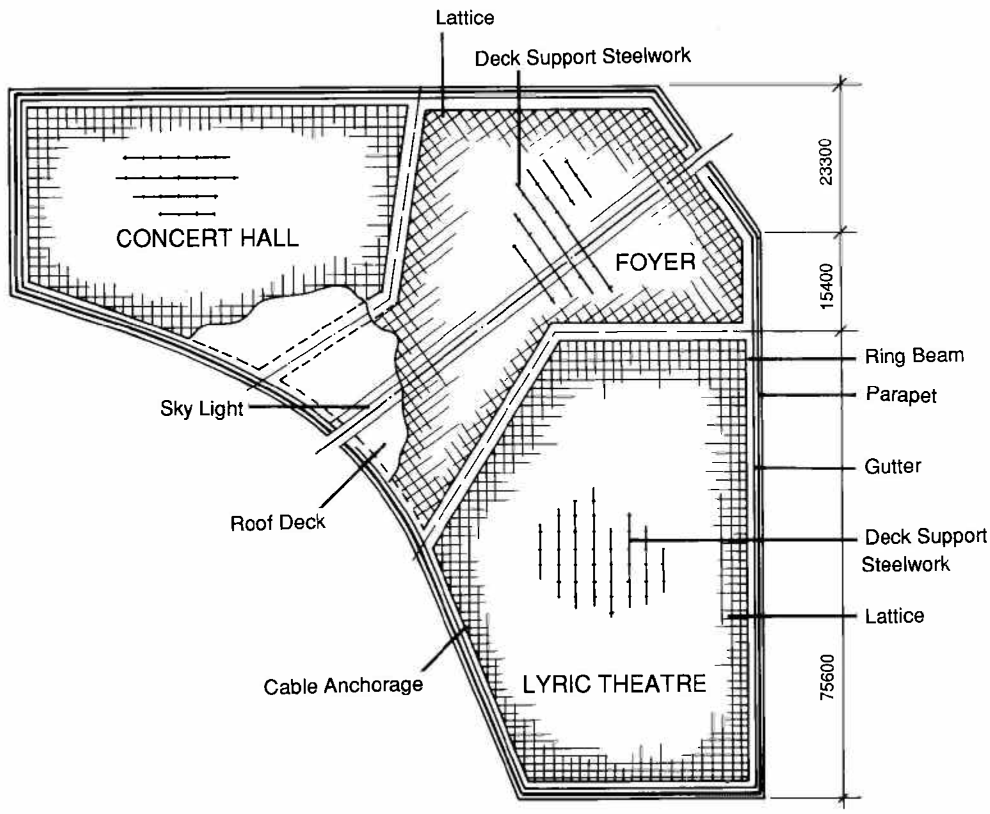


Fig 1.6 Roof plan with surface divided in two by linear skylight, overlying three distinct structural areas of complex

three zones provide natural boundaries for three separate roof structures, with the roof being conceived as a two layer construction. Each of the three zones has a bottom structural layer comprised of a separate hanging rigidised cable net consisting of 30mm diameter cables on a 1.5m grid encased in concrete to form a lattice of beams 160mm wide × 290mm deep. Such a choice was determined by both practical and structural considerations. Individual members had to have sufficient depth to give out-of-plane bending stiffness, and sufficiently large cross-section to enclose the cables adequately. A structural grid of 1.5m with concrete member size of 160mm × 290mm deep was chosen as satisfying both requirements. This cable net supports a framework of secondary steelwork, which provides the support for the external cladding. The completed lattice has the equivalent concrete thickness of 45mm, with a combined reinforcement and cable weight of 25 kg/m^2. The total roof area of 10,000m^2 is divided into three fields – the concert hall, foyer and lyric theatre, all lying on an L-shaped plan with the foyer on the intermediate axis. Rise in roof level from the foyer to the top of the lyric theatre is 34m.

Cable net forces and lattice shell solution

As the cable net in the bottom roof layer rarely lies further than 2.5m beneath the outer roof surface, the 'equivalent structural depth' is small. Although the roof surface is not funicular it is quite close to being so. The nets have double curvature, but because of the given geometry of the boundary walls, for each net there is a significant primary direction of curvature and therefore a primary direction of forces. The primary cables are consequently of larger cross-section than the secondary ones.

Cable nets resist uniform downward loads very adequately and economically by tensile forces alone. However, loading from wind and other live loads will not be uniform, and the cable net will resist these forces without large deformations only if the uniform deadweight loads are significantly larger than the non-uniform loads, or if the net is made stiffer and can then resist these forces through a combination of bending and axial compression. Since Hong Kong is in the monsoon belt of the South China Seas, it is regularly subjected to very high winds. It was essential therefore that careful assessment be made of the likely patterns and magnitudes of wind loading on the Cultural Centre, and to this end an accurate scale model was commissioned and tested in a wind tunnel. The results of the wind tunnel test detailed later in this journal demonstrate that the wind suctions are very much higher near the roof perimeter than within the central zone. This is due to the fact that there is no parapet or other feature to 'trip' the wind flow and therefore the wind can 'attach' itself to the sharp edge. Such features as parapets were regarded as incompatible with the architectural intent, which required the least possible visible separation between roof and walls.

In order to resist these resulting high non-uniform wind and live loads on the cable nets without excessive deflection a deadweight in the region of 250–350kg/m^2 would be required. This is considerably greater than that considered economical, and would carry the added penalty of higher cable loads and consequently larger cables and larger forces to be resisted at roof edges. It was therefore considered more appropriate to use stiffness to resist non-uniform loading, as this also provides a useful control of the dynamic response of the structure to wind-exciting loading. A braced cable truss was investigated, but was considered to be too uneconomic in form and proportions to offer a realistic option.

By itself, a cable net has almost no stiffness; non-uniform loads simply cause it to change shape, and uplifts cannot be resisted at all. But if the cables are surrounded in concrete the lattice so created becomes much stiffer; in fact it becomes an inverted shell or dome, and resists uniform uplift forces by compression in just the way that a normal dome resists downward loads. When non-uniform loads, either up or down, are applied, the 'lattice shell' now has the stiffness to act as a grillage of individual members spanning between stiff joints, each member having bending stiffness in both the horizontal plane (in-plane bending) and in the vertical plane (out-of-plane bending); in-plane bending moments in each member are passed to its right-angled neighbours by in-plane shear, out-of-plane bending by torsion (Fig 1.7). Therefore an inverted lattice shell, whose shape is defined by a cable net, resists uniform down-loads by pure tensions in the members, and when uniform up-loads build up so as to exceed uniform down-loads the forces in the members become purely compressive.

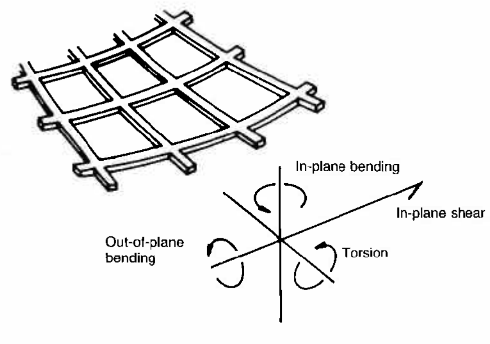


Fig 1.7 Component member forces in lattice shell under non uniform loading

It was this simple concept of surrounding tension cables in concrete to create a rigidised net that lay at the heart of the key feature sought by the architects for the Cultural Centre – that of a pure, sculptured roof. There is logic to the structure which is reflected in the architecture and which is underpinned by cost – the roof structure, at £173/m^2, is extremely economic. Design and construction of the complex took eight years; the following articles consider this process in detail.

Ted Happold, John Morrison and Terry Ealey

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Tsim Sha Tsui Cultural Centre – Foundations and Superstructure

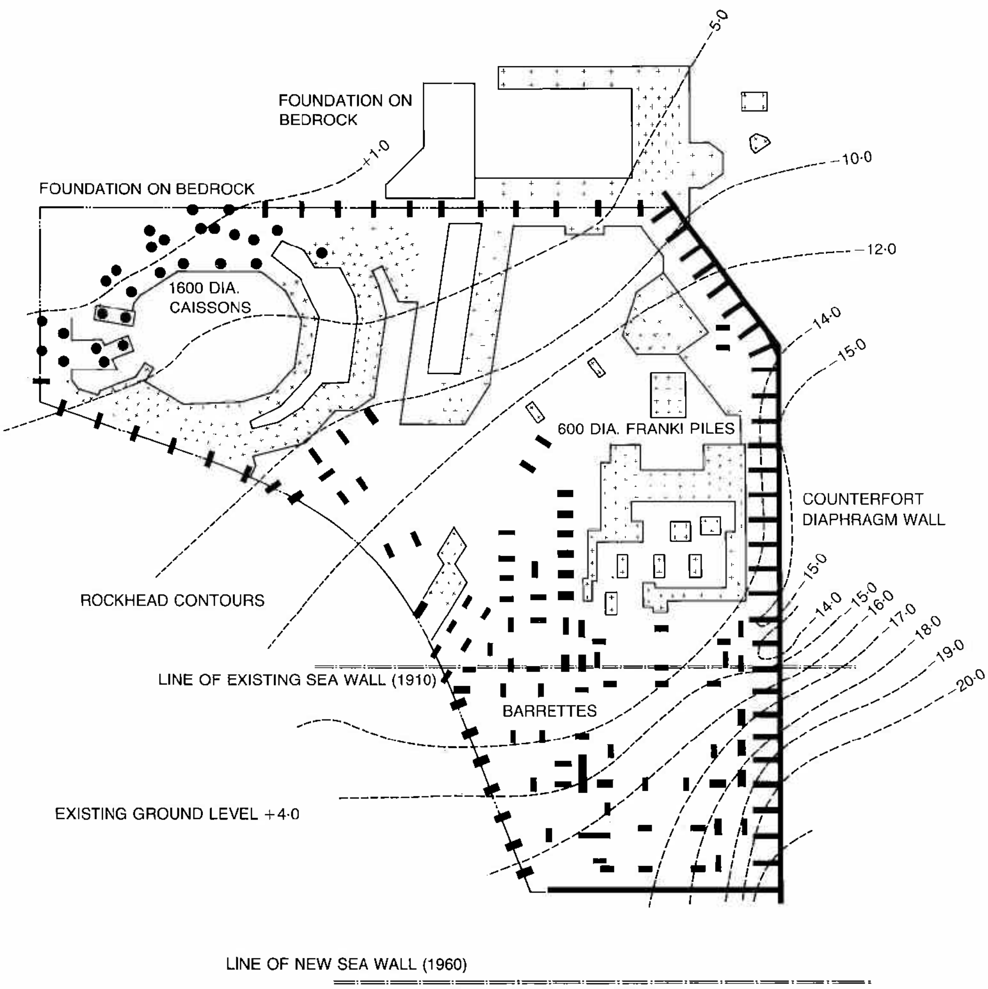

Fig 2.1 Lines of sea walls and depth to bed rock revealed in site investigation, with zones of selected foundations

Design commenced under some pressure in July 1981. It was essential that the lyric theatre foundations should be complete as soon as possible to provide a basement retaining wall for Phase 1 (Fig 1.3), the administration and restaurant complex that had already been designed by the ESD and for which a construction contract could then start.

Foundation investigations and design

The available geotechnical data consisted of borehole logs for Phase 1 and a few from the mass transit railway which had been built to the east of the site. These, and the subsequent investigation revealed a site consisting of approximately 7m of fill over marine deposits and some alluvium, all on decomposed granite and bedrock, with a notable wide variation in both bedrock level and the overlying depth of fill.

The construction of sea walls and reclamation to provide development land is virtually a continuous process in Hong Kong. The site in question had two sea walls – the present one which was built in the early 1960s, and a buried wall which was believed to be contemporary with the 1910 railway terminus. Behind each the site had been filled, mainly with decomposed granite. The exact location of the older wall was unknown and had to be traced in a later site investigation as not only could it present a physical obstruction for any new foundations, but it also represented a boundary to the types of fill, of great importance in the design of piled foundations. Normally piles can derive some of their carrying capacity from the friction generated between the pile sides and the surrounding soil. However, on fill sites effectively 'negative' skin friction can occur with land settlement and therefore a proportion of the end-bearing capacity must be set aside to counter this force, obviously leaving less available to carry the load of the structure above. The older material between the earlier seawall and the original shoreline could then be considered as fully-consolidated. Site investigations determined the location of the old sea wall and revealed the variation in bed rock depth (Fig 2.1). The north-west corner of the site shows bedrock virtually at ground level, while in the south-east corner it is at a depth of 25m. This wide variation in level is typical in Hong Kong where decomposition of granite has occurred in tropical conditions.

The usual method for tendering piled foundations in Hong Kong is to provide the contractor with a schedule of intended loads and moments, the soil report and any restrictions on pile types. The contractor then designs the pile system, accepting full responsibility for it. He is only occasionally required to construct the pile caps, this usually being undertaken by the main contractor. However, to ensure that the client is not provided with cheap piles but expensive pile caps, the contractor also designs and prices these so that the complete cost is known. The system works well for standard situations, but was not appropriate for the Cultural Centre where the designers needed to retain full control so allowing opportunity for change as the design evolved. The main element requiring control was the 9m deep retaining wall along the boundary between Phases 1 and 2. This not only served as the basement wall for Phase 1, but also provided the foundation for a 60m high wall of the theatre to be built above. In order to maximise wall stiffness and provide support for two 400mm thick walls spaced 4m apart, counterfort diaphragm walls were proposed, with stem and flange 800mm thick, and counterforts at 4.7m centres. The diaphragm wall was excavated to bedrock, with beams cast over the stems as anchorage points for temporary vertical ground anchors designed to stabilise the wall during Phase 1 excavation. In the final condition the temporary anchors would be released, and the ground slab of Phase 2 would then act as a horizontal tie. Inclined anchors back into the Phase 2 area might have been more efficient, but would have required access and installation during the Phase 1 contract. There was also a risk that inclined anchors could have been damaged during the subsequent installation of piles and barrettes in Phase 2.

Some movement of the top of the wall would be inevitable during the construction work and would be dictated by the sequence of excavation between the contract phases. Although the excavation sequence could not be effectively controlled or predicted, it was important to study the range of

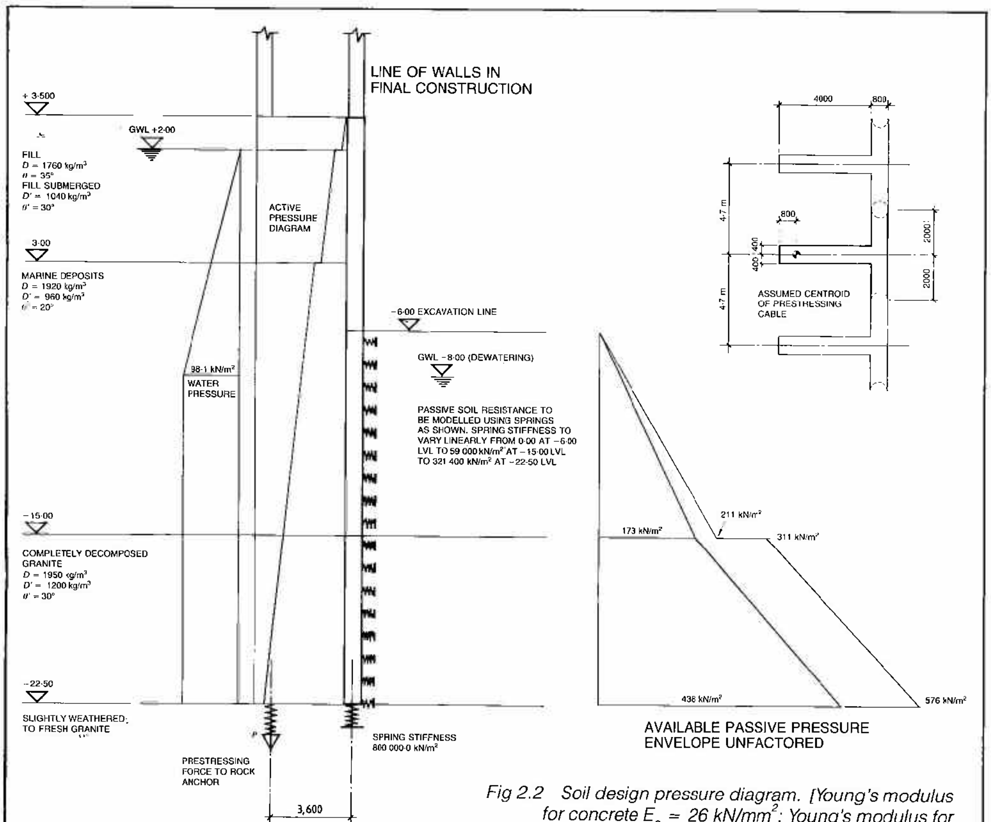

Fig 2.2 Soil design pressure diagram. [Young's modulus for concrete E_c = 26 kN/mm^2; Young's modulus for prestressing strand = 175 kN/mm^2.] A typical set of design criteria for a specific depth to bedrock.

Fig 2.3 1600mm diameter hand dug caissons adopted where depth to bedrock was less than 5–6m

movement and to establish acceptable criteria so ensuring that the superstructure to be built above would remain vertical. To elucidate the likely movements of the top of the wall, an analysis was carried out under a variety of load conditions, water levels and depths to bedrock. When an acceptable solution was satisfactorily established, the variables that could not specifically be defined were presented to the tenderers as three standard sets of design criteria for various depths to bedrock, of which Fig 2.2 is typical. It was specified that the horizontal movement at ground level should not exceed 10mm inward or 25mm outward during any stage of the Phase 1 or 2 excavations and construction work.

The extensive use of diaphragm wall techniques made it logical to continue with this form of construction to provide barrette foundations; 'barrette' being the French term for a length of diaphragm wall used as a pile. The authors had previously used the technique for the foundation design at Centre Pompidou, Paris, however this was the first time that such foundations were used in Hong Kong. They were ideal for the zone between the old and new sea walls, where negative skin friction accounted for a large proportion of the pile load. A conventional 600m diameter pile has an end-bearing capacity of 140 tonnes, but using the standard Engineering Services Department's negative friction formulae, 100 tonnes of this is required solely to provide for the negative skin friction effect. Although barrettes attracted a similar degree of skin friction, their greater cross-sectional area, variable design capacity and the fact that they could be founded on the bedrock rather than on decomposed granite, ensured their success.

Where foundations were behind the original sea wall, 600mm diameter Franki piles became a more practical solution, as both negative skin friction and depth to decomposed granite were reduced. However, these piles became impractical once the depth to rock was less than 5–6m, and 1600mm diameter hand dug caissons were adopted as an alternative (Fig 2.3). A few foundations in the north-west corner of the site could be founded directly on bedrock.

The barrettes and diaphragm walls were tested for satisfactory founding using sonic coring techniques, while the 600mm diameter piles were vibration tested. Both techniques were relatively new in Hong Kong, and for some barrettes it was necessary to correlate results with the more traditional coring methods. These cores showed the joint between concrete and bedrock to be perfect in all cases. The contract called for all Franki piles to be vibration tested, backed up by three pile tests to working load. If an individual pile test failed the settlement criteria of 7mm, then a further three piles would be tested, and so on without additional cost. These severe criteria, requested by the ESD, certainly ensured that workmanship on all piles was of the highest quality.

The foundation contract was let to Soletanche-Bachy for HK$ 29 million. Work started on site in October 1981, and was successfully completed in the allotted nine months.

Design of ring beams and perimeter walls

At the first design team meeting, the architect stated that the building was to be regarded as a combination of intersecting surfaces – there could be no columns, and dramatic effects were to be created by the use of large spans and cantilevers. Superimposed over the three functional zones of lyric theatre, auditorium and foyer lies the structural envelope of perimeter walls, ring beams and roof (Fig 1.6), the form of the latter having already been decided by the time Ho-Happold was appointed as described earlier. Basic data defined the crest levels for the two wings and the low point on the curved surface joining them. A cubic spline curve was used to find a best fit for these points, the tops of the walls on either side of the building then being related to this curve by radial generators. Thus, the original, freely drawn architectural sketches were redefined in a mathematical form – an essential precursor for the development of the roof perimeter and the ring beams and walls supporting it. The perimeter walls vary in height from 21m in the foyer

Fig 2.4 (a) Anchorage blocks of cable net above the surface of the ring beam

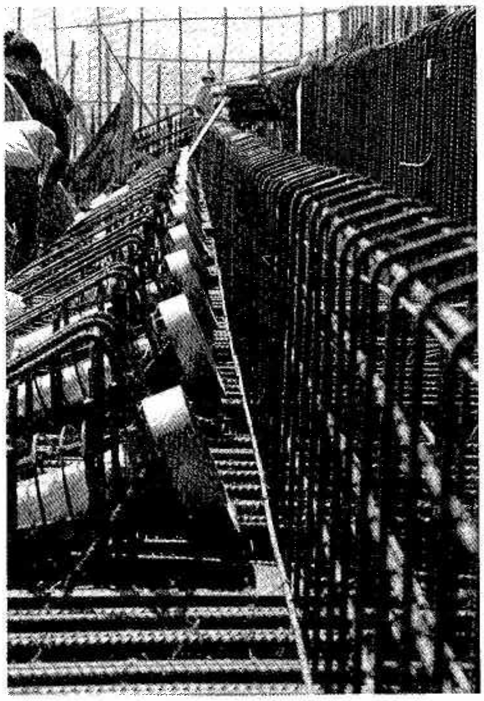

Fig 2.4 (b) Anchor pocket formed where cable net and ring beam have low angle of incidence

to 56m behind the fly tower to the lyric theatre and 40m at the rear of the concert auditorium. In general, a 400mm wall thickness was adopted for the lyric theatre and auditorium, where service floors provided horizontal stiffening and stability. In the foyer, this means of lateral restraint was not available and a 600mm wall had to be used.

The roof cable net produces varying horizontal and vertical forces, depending on curvature and span, and under extreme wind loads these forces could be reversed, making the rigidised cable net act as an inverted shell. The roof cables intersected the 2650mm wide ring beam face at mid-depth. Depending on the angle of incidence between the cable net and the ring beam, the anchorage blocks were generally above the surface of the beam. In a few cases, where the angle of incidence was low, pockets had to be formed for the anchors (Fig 2.4 a, b). The ring beam itself was designed as a continuous slab supported on three sides by perimeter walls and counterforts and subject to both in-plane and out-of-plane forces. Spacing of the counterforts at 8–12m was determined by the planning and counterfort restraint was provided by a push-pull through a minimum of three floor slabs. In a few instances where the height to the first restraint slab was considerable, the system of counterforts, ring beam and wall had to be treated as a vertical grillage to obtain adequate strength and stiffness.

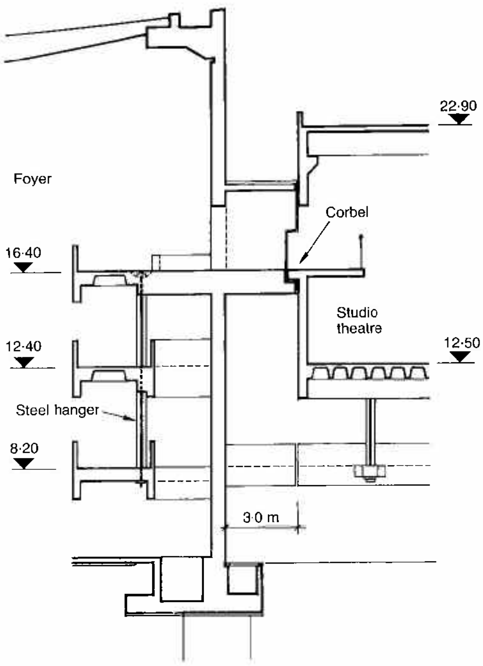

Fig 2.5 Section through north wall of foyer adjacent to studio theatre

The foyer provided the most interesting part of the ring beam design, as the number of counterfort walls available to provide lateral restraint was limited. A new range of problems was presented, the most difficult being the 25m high north wall adjacent to the studio theatre (Fig 2.5), with intermediate 6m cantilever floors at the 8.2m, 12.4m and 16.4m levels. An added complication was the requirement for a 10.7m × 2m escalator slot at the root of the two lower cantilevers. The solution lay in using two 150mm diameter solid steel hangars to help suspend the slab from the cantilever above it. The maximum strain that the hangar might experience was limited to ensure that the deflection did not exceed the operating tolerance of the escalator. The combination of ring beam and cantilever slabs was such that, unsupported, the north wall could not provide adequate resistance to the moments and lateral deflections that would act upon it. The solution entailed part of the weight of the studio theatre being mobilised to act as a counterbalance via a short corbel bridge link. The contract documentation required that the wall and slab at 16.4m level and the corbel link to the studio theatre be propped and cast as one. The remaining wall and ring beam, together with the studio theatre, would then be completed. Only then could the 16.4m slab be depropped and the suspended slabs at levels 12.4m and 8.2m constructed. Thus only by a combination of counterbalance effect and the use of both ring beam and slab at 16.4m as horizontal beams, was it possible to achieve a stable wall.

The use of braced counterfort walls necessitated a close check on the horizontal forces transferred to each part of the structure as there was no guarantee that equal and opposite forces would occur. In a few instances it was necessary to use internal walls to transmit out-of-balance forces to lower levels, greatly complicating the provision of movement joints. It had been the intention that such joints should be at 30–40m centres. However a combination of the need for continuity at cantilever balconies, resolution of horizontal forces, and maintenance of acoustic barriers forced acceptance that joints could be provided only at points of major structural weakness. This occurred at three locations, each due to an abrupt change in wall stiffness – one occurred at the narrow window through the foyer wall forming the continuation of the line of the central roof light; the second at a vertical glazing slot in the rear of the auditorium located 18m from the rear wall; and the third at the vertical glazing slot to the rear of the stage for the lyric theatre. As a result, the longest external wall was 85m. The idea of leaving a 1m wide construction joint to be cast at a later stage was considered, but while this would have eased the shrinkage problem during construction it would not have solved the long term problem of thermal movement.

A series of finite element analyses were carried out to study the differential shrinkage stresses arising from the waffle slabs, which with a high surface area to volume ratio, shrink faster than the 600mm solid walls. Because shrinkage is a time dependent effect, it was important to consider various speeds of construction, and for the thermal condition, the orientation of the walls and therefore the length of time they were exposed to the sun also had to be taken into account. In each case, the base of the wall was treated as partially restrained by the ground beams, although in actual fact the detailing tended to provide nearer full restraint. Results of this analysis proved that the mass of the 600mm thick wall was sufficient to prevent large thermal fluctuations under the low diurnal temperature range of 5°C. The worst temperature effects in fact occurred in winter when the building was heated.

As considerable effort had been put into these studies, it was disappointing to find that most of the work could have been avoided. The movement of the top of the wall coincided almost exactly with that of a free unrestrained member. Re-examining the analysis, it was easy to see with hindsight that the height of the walls and flexibility of the roof were such that little or no restraint to the longitudinal forces existed at ring beam level, and doors at ground level were only minor stress generators, as they were located in zones of low strain.

In the event, a very low level of shrinkage cracking occurred at the Cultural Centre, helped in part by the high humidity and low diurnal temperature range, but achieved for the most part by the excellent quality control maintained for the concrete and its curing. A 30N/mm^2 concrete was specified, containing 30% PFA cement replacement. Lateral reinforcement in the external walls was provided to ensure crack widths did not exceed 0.2mm, in line with standards required for watertight concrete, as at the time the final form of waterproofing treatment had not been agreed.

When the problem of movement joints was first discussed, an inspection of several existing buildings was carried out. Although there are not many long low rise buildings in Hong Kong, a variety of solutions had been adopted and it was noticeable that the few buildings constructed without joints showed no signs of distress. This absence of significant cracking was almost certainly due to the flexibility of construction and to the attentiveness of the original designers, although they may also have been assisted by the rather peculiar creep properties of Hong Kong concrete, acting beneficially in this instance.

Deep beam walls

The lyric theatre has three stages – the principal being 23m × 19m with a 26m high fly tower, the rear stage 18m × 14m, and the side stage 16m × 16m, both with 14m headroom. Above the rear and side stages accommodation is provided for scenery painting, changing rooms, rehearsal and administration areas, many of which require large column free spaces. To achieve these spaces and span the stages below, the fly tower walls and the walls supported by them were designed as deep beams, the height of wall available for deep beam action being 17m with a maximum span of 19m (Fig 2.6). Following CIRIA recommendations (Ref 2.1), a minimum practical dimension of 400mm was

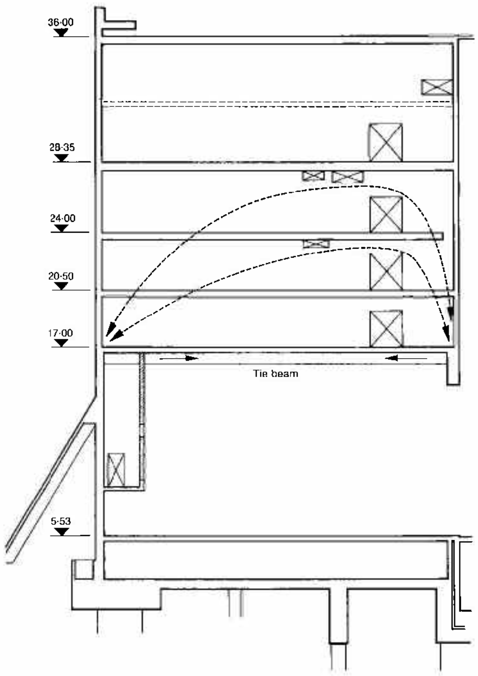

Fig 2.6 Section through deep beam walls of fly tower and lyric theatre showing action of forces

adopted to accommodate the reinforcement. As the walls had to remain propped only until sufficient depth had been cast, the main task was to minimise what this depth need be. Initially a finite element analysis was used to establish the tension zones by treating the walls as homogeneous unreinforced concrete. An assessment was then made of the amount of reinforcement required and expressed as equivalent areas of concrete by making further computer runs. The problem with this type of analysis is that the plane strain model assumes that the wall is perfectly elastic; even if the concrete is cracked it is assumed that it can still transfer stress. Stiffness is therefore overestimated, and there is a risk that the iterative computations can run away, due either to crushing at the supports or to the spread of the tensile zone. Finally, to obtain a better understanding, those elements whose tensile stress exceeded 0.5N/mm^2 were removed, leaving only the reinforcement to act as a tie. This gives the other extreme, as the shear interlock between the concrete is omitted. In practice it was possible to show that the deflection and resultant stresses set up by these three analyses did not vary significantly.

While these analyses proceeded, it was considered prudent to make provision for a 800mm × 1000mm deep tie beam for each wall which could then be either reinforced or post-tensioned, according to the outcome of the analysis. In the end a passive reinforced system was preferred, as post-tensioning would have required either stressing of the beams before casting the floor, or devising a method to avoid loss of prestress through the floor slabs. The effect these complications would have had on construction time, and consequent difficulties in anchorage location, finally weighed in favour of the passive system.

Allowing for doorways and service holes, it was eventually found that a minimum depth of 12m must be completed before walls could be depropped. Clearly, the temporary propping system itself had to be rigid if settlement stresses were to be avoided. This was achieved by specifying a uniform distributed load and a deflection limit of 10mm for the temporary supports under each wall. Propping loads were of the order of 400 to 500kN/m run, but where recent fill occurred it was appreciated that this deflection criteria would not be achieved unless piled foundations were provided. Although this could strictly be considered as a temporary works requirement, piles for the support of temporary works were provided as part of the foundation contract. Pile caps were designed as hollow box girders to reduce self-weight and to ensure a platform sufficiently wide to give lateral stability for the temporary works (Fig 2.7). The temporary works scheme was prepared to ensure that the specified criteria could be achieved economically using practical section sizes. At the time of tender, the contractor was required to submit temporary works proposals and demonstrate by calculation that the deflection limits would not be exceeded.

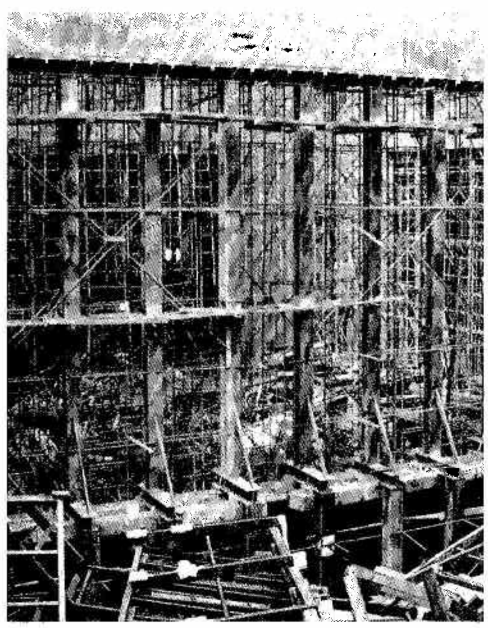

Fig 2.7 Temporary propping system of lyric theatre

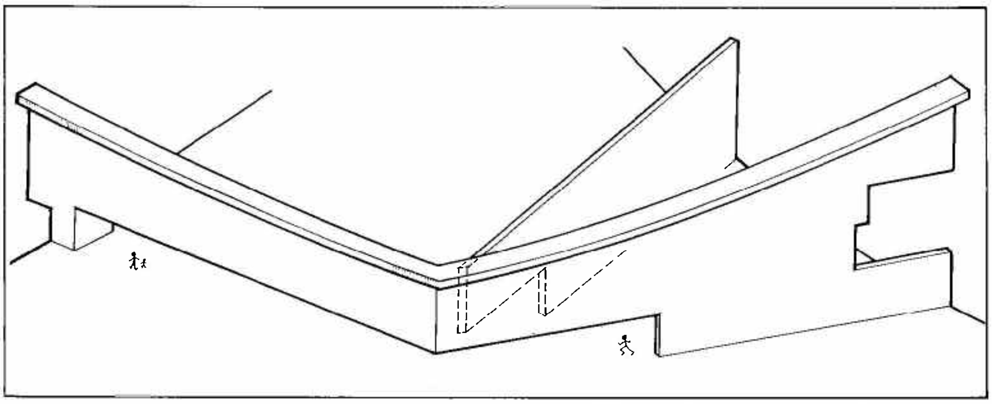

Fig 2.8 (a) Cantilever support to the upper circle of the lyric theatre

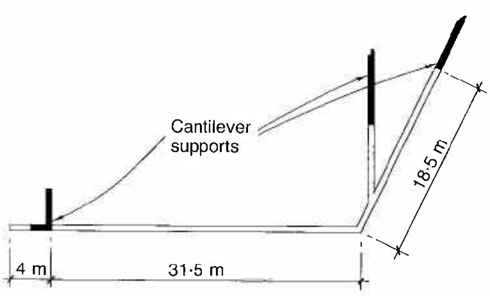

Fig 2.8 (b) Plan of lyric theatre cantilevers

A major feature of the lyric theatre was the design of the support to the upper circle (Fig 2.8 a, b), a wall acting as a 6m deep T-beam with a span of 31.5m. The wall supports the cable net roof from both foyer and lyric theatre, together with part of the upper circle seating and the theatre's inner roof. One end was supported by a short length of wall before the beam continued as a 4m end span to the external wall. Such an arrangement of 4m and 31.5m spans would have resulted in high shears and moments, and so a movement joint was formed on the face of the external wall, leaving the 4m span to act as a cantilever. At the other end the beam was supported by two cantilever walls. The shorter cantilever, providing support at right angles, was 8m long, 4m deep and 650mm thick, and also supported the inner roof of the theatre. The second cantilever providing support at approximately 60°, was 17m long, 400mm thick, but in this case, due to the roof curvature, 9m deep. Relative stiffness of these intersecting cantilevers was of considerable importance in determining the distribution of the forces and the stress concentration at the supports, the latter being treated as prestressed anchorages to ensure adequate linking of the vertical reinforcement at the leading edge of the walls.

Foyer galleries

The foyer was envisaged as a vast space 70m long, varying in width from 40m to 20m and surrounded on all sides by three floors of cantilever galleries, linked with bridges at the narrowest point. The first gallery is accessed by five 4m wide stairs to allow visitors a graceful climb to the theatre level (Fig 2.9). This combination of stair and gallery acts as a 20m span tied arch, the remainder of the first gallery being supported by cantilever construction. The second and third galleries, up to 8m in span, are supported mainly by cantilever action. With a structural depth of 1m for foyer cantilevers, ribbed and waffle construction was adopted to keep self-weight to a minimum. A spiral stair linking first and second levels posed a problem acting as either a

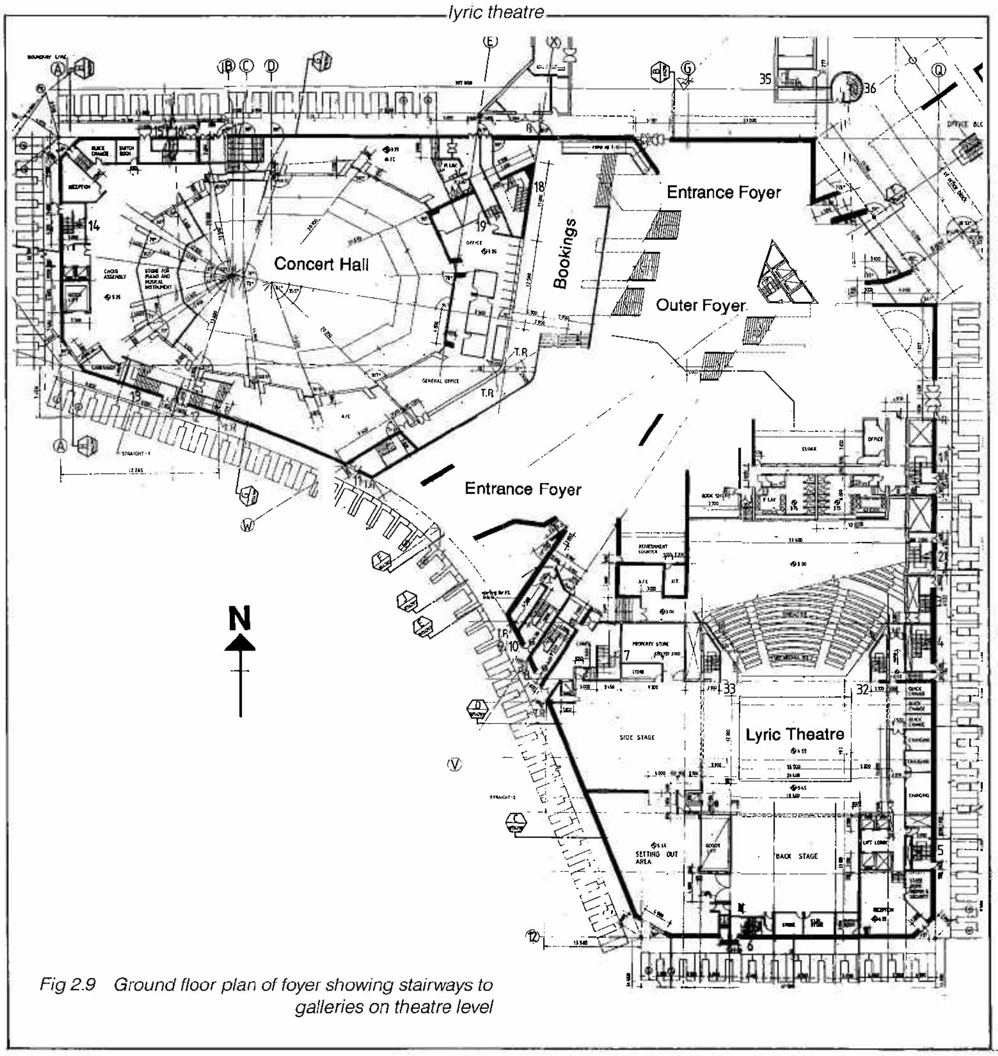

Fig 2.9 Ground floor plan of foyer showing stairways to galleries on theatre level

Fig 2.10 Main stairways, spiral link between first and second levels and cantilevered galleries of foyer

spring in compression or tension, depending on relative loading conditions (Fig 2.10).

The unusually low E value of Hong Kong concrete required that a greater consideration be given to the effect of creep on design. An interesting article entitled 'Is it concrete?' was one of the first studies which had revealed the modulus of elasticity of Hong Kong concrete to be considerably lower than figures recommended in CP 110. The reasons for this phenomenon were not clearly understood at that time and only a few structural engineers were aware of the effect. Bridge engineers were better informed following the unexpected deflection and loss of prestress of the Ysing Yi island bridge, after which the Hong Kong Highways Department had issued design guidance notes. For the Cultural Centre project grade 30 concrete with an E value of 21 N/m^2 was adopted, as opposed to the CP 110 value of 28 N/mm^2. Cantilevers were checked for various age-strengths and for combinations of redundant compression and tension steel before depropping. Detailed instructions regarding the camber and duration of propping were given to the contractor, and it was stipulated that the balcony parapets should not be cast until 56 days after the cantilevers had been depropped.

Concert hall and inner roof construction

The banks of auditorium seats comprised a system of cantilever beams framing into the 22m high rear walls, stabilised by access floors, the size and shape of these walls being determined by the geometrical faceting of the hall required to achieve the desired reverberation time. In a few cases cantilever moments had to be resisted by propping forces at the roof level before they could be resolved. The setting out of the auditorium was particularly complex – the most efficient solution lying in the use of a CAD system. Having successfully established this it was important to avoid inconsistency between the architect's and engineers' drawings, and so the unusual step was taken of explicitly writing into the contract documents that the engineers' drawings be used for all setting out procedures.

The auditorium and lyric and studio theatres had inner roofs acting as acoustic barriers and providing support for the ceilings, suspended sound diffuser panels and lighting grids. Composite construction of plate girders supporting in situ 150mm × 175mm slab was adopted for the 31m span lyric theatre, 19m span studio theatre and for the auditorium where the spans varied from 21m to 35m. The advantage of such construction lay in its lightness during erection, and in avoiding the need for falsework which in most cases would have been more than 20m high.

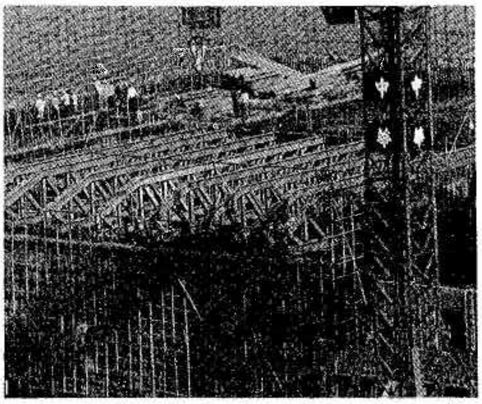

Fig. 2.11 Roof trusses of fly tower in position

Deep plate girders, 1.5m deep and at 3.1m centres, provided composite beams for the lyric theatre. However in the auditorium, design was complicated by 50% of the roof area being occupied by rehearsal and plant rooms (Fig 1.5). This together with the varying spans of the beams, required care in ensuring deflection compatibility. Beams ranged from 2m to 1m in depth, and were also designed to act compositely where walls occurred along beam lines. Slabs were required to be cast in transverse strips, starting at midspan to provide a small improvement in section stiffness of each successive pour, thus helping to minimise the self-weight deflection. As the self-weight of each of the longest beams was 16t, the designers proposed a method whereby the beams could be lifted from the foyer to a track on either side of the auditorium. The two longest beams could then be used to form a trolley to transport the shorter beams sideways before lowering into position.

The fly tower roof trusses were designed to support 13.5kN/m^2, representing the live load of scenery and counterweights. Working platforms, lighting bridges, side galleries and safety curtain were also supported by these trusses (Fig 2.11), as were part of a rehearsal room, an access corridor and a smoke ventilation structure. In order to minimise both member sizes and overall depth, trusses were designed with partial composite action, resulting in an overall depth of 2.4m for a 19.2m span and consisting of back to back channels (432mm × 102mm) with 203mm universal columns for struts and diagonals. In view of the weight and inaccessibility of the fly tower, each truss was designed to be broken down into three more easily managed sections. However, during construction the size of cranes used by the contractor rendered this unnecessary (Fig 2.12).

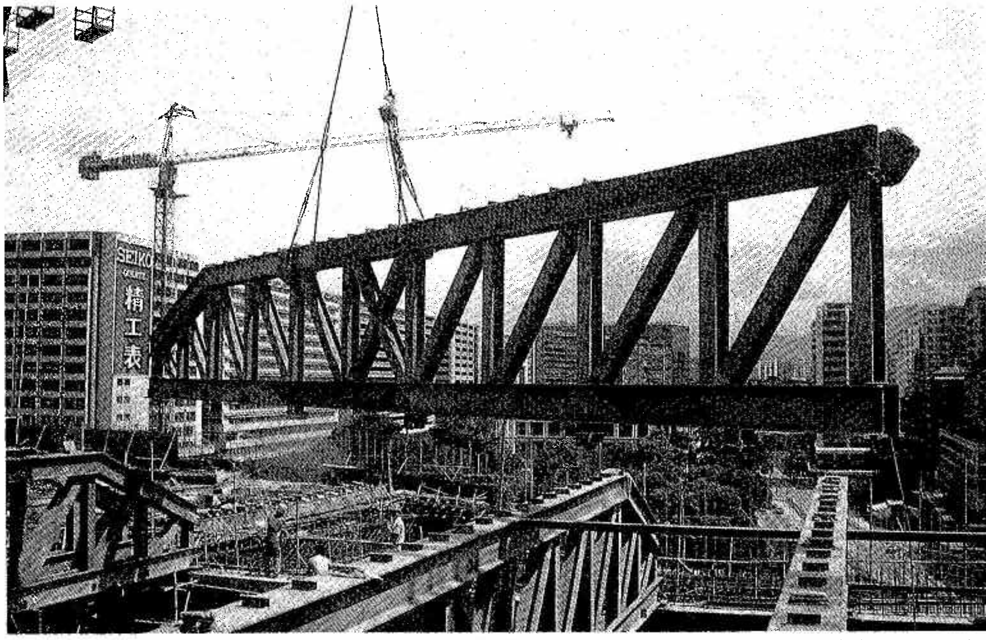

Fig. 2.12 Manoeuvring of roof trusses of fly tower during construction

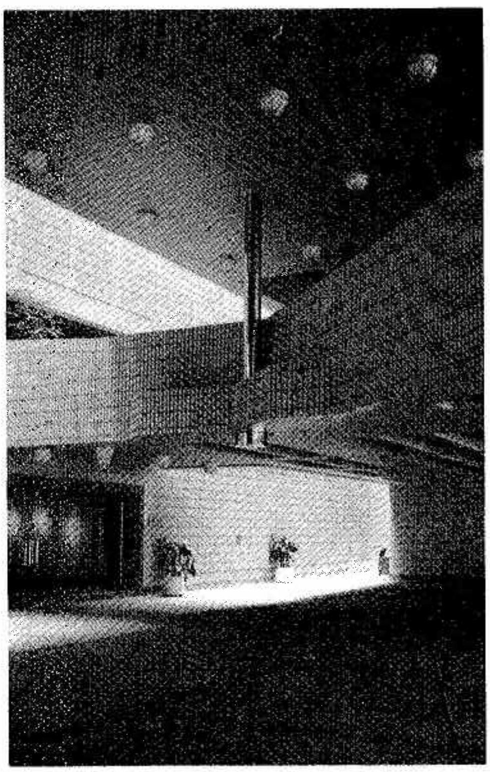

Fig. 2.13 Solid billet hangers supporting access deck within studio theatre

Studio theatre

The studio theatre also contains many of the features characteristic of the rest of the Cultural Centre – deep beam walls, cantilever construction, hanging slabs and the use of solid billet hangers (Fig 2.13). The theatre 41m × 19m, is supported at each end on service towers. Walls cantilever from these service towers by 2.5m on the north and south faces. The theatre floor on the second level consists of a central 1.4m deep five cell box girder slab. This box girder is 7.5m wide and spans the width of the theatre, supporting 11m side spans of 600mm deep ribbed slabs between it and the towers. It also provides anchorage for two 150mm diameter solid steel hangers which support a 600mm deep ribbed slab access deck. This complete system of theatre floor and suspended slabs transfers its load to the lower flange of 11m deep beam walls spanning 41m, which provide support for both a cantilever seating gallery and the roof construction. However, the supports for the deep beams are themselves 2.5m cantilevers.

Construction of the superstructure was carried out over a three year period, with successful completion of works in 1986.

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2.1 CIRIA: 'The design of deep beams in reinforced concrete' CIRIA Guide 2 1977

Tsim Sha Tsui Cultural Centre – Analysis of the Roof System

As described in the first article, the cable nets comprising the basic structure of the Cultural Centre roof system are anchored into a concrete ring beam which runs at the top of the wall around the perimeter of the building and at the top of the walls dividing the foyer from the concert hall and lyric theatre zones (Fig 1.6). The roof deck had to be lightweight to avoid loading the lattice shell and ring beam anchorages any more than was necessary. Furthermore it had to be watertight and sufficiently insulated to control heat gains and losses through the roof, and the chosen system of roof decking had to be sufficiently flexible to assume the curvature of the roof surface. In the event, metal sheeted roof deck supported by purlins and rafters and by a light steel transfer system of struts, rafters and purlins was adopted. Circular hollow sections were used for the struts and rafters, because such sections are particularly suitable where members do not meet orthogonally. Purlins are of channel section, to which sheeting is easily fixed (Fig 3.1). Bracing of the framework is required to provide stability and to resist the horizontal component of imposed live and wind loads, and is provided by inclined members.

The skylight at the centre of the roof which provides a strong line of daylight into the building along the axis of the foyer, is constructed of hollow cross-section members in a similar manner to the general roof steelwork system. Rainwater drains directly down the roof slopes to main gutters each side of the skylight, which with a width of 1500mm and depth of 750mm take rainwater run-off from the peak storm intensity (Fig 3.2). A gutter sitting on the concrete ring beam, runs around the entire perimeter of the roof. Water from each of the 1500mm wide main gutters discharges via a hopper into two 800mm diameter rain water down-pipes, which are surrounded with concrete to provide lateral stability and sound isolation.

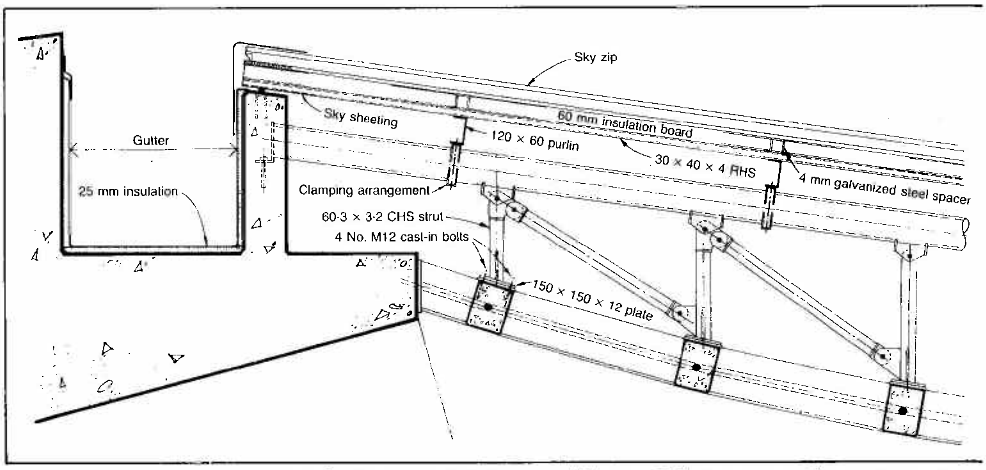

Fig. 3.1 Section through roof construction showing inclined bracing, and side gutters of complex

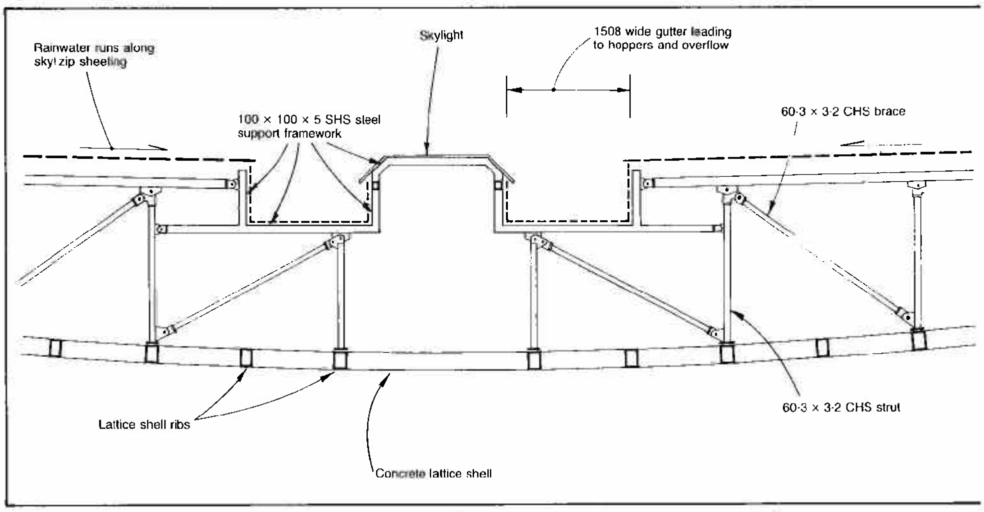

Fig. 3.2 Section through central skylight and main gutters of roof

Roof loadings

Loadings to the roof are made up of dead, live and wind load components. Dead loads result from the self-weight of the cable net, and of the concrete and reinforcement of the lattice shell, the weight of the roof deck and its intermediate support system, and the weight of building services. Live loads arise from either rainfall or building maintenance, for which a uniformly distributed load allowance of 15 kg/m^2 was made. Extremely heavy rainfall can occur during the tropical storms of the South China Seas, and as the roof of the Cultural Centre is large by normal standards, it was necessary to calculate the probable maximum depth of rainfall on the roof surface and in the gutters in order to evaluate the maximum live loading due to rainfall. A return period of 100 years was assumed, for which the expected peak rainfall intensity was 400mm/h for a storm duration of 30s. Calculations indicated that

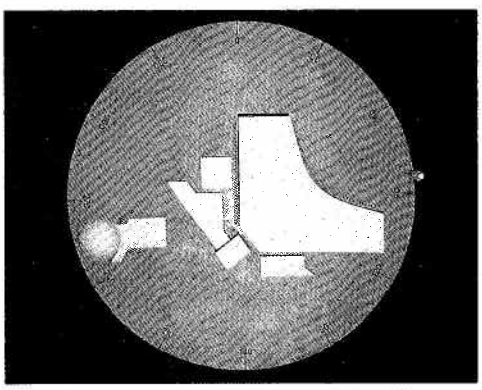

Fig. 3.3 Scale model of complex (1 : 300) in wind tunnel showing ground roughness blocks in foreground

roof surface loads due to rainfall would reach 30 kg/m^2 at the lower levels, and gutter loads would be up to 280 kg/m.

Wind tunnel tests were carried out on a 1 : 300 scale model in October 1981 at the Building Research Establishment using a 1 : 3 scale wind thereby giving a 1 : 100 wind tunnel time scale factor (Ref 3.1) – a 0.01s duration for a model represented 1s for the real structure. The model was in the wind tunnel for a total of eight working days (Fig 3.3). Pressures were measured at each of 86 tap locations for 12 angles of incidence of the wind, taken in steps of 30°. Pressure readings for each tap and for each angle of incidence were output, together with pressure contour plots (Fig 3.4 a, b).

Wind speeds have been recorded continuously in

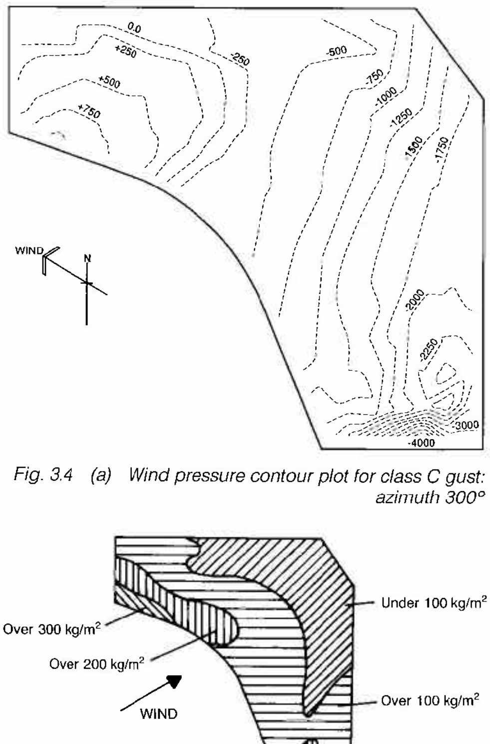

Fig. 3.4 (a) Wind pressure contour plot for class C gust: azimuth 300°

Fig. 3.4 (b) Wind contour plot : azimuth 60°

Hong Kong since the last century, except for the period of the Japanese occupation in World War II. The Meteorological Office in Bracknell carried out a Gumbel analysis (Ref 3.2) of the wind data and predicted a value of 139 knots as the 3s gust wind speed at 10m above ground for a return period of 50 years. These results compare with the maximum gust of 140 knots during Typhoon Wanda in 1962 which is still the highest recorded gust speed since records started in 1884. The analysis also predicted a value of 79 knots (40m/s) for the hourly mean wind speed at 10m above ground for a return period of 50 years. This compares with the maximum hourly mean wind speed of 69 knots recorded at Kowloon in 1896 and of 78 knots recorded at Waglan Island in 1979. The equivalent hourly mean wind speed used in the wind tunnel testing was therefore fixed at 40 m/s.

For most of the roof surface the pressures plotted from the wind tunnel tests were lower than recommended by the Hong Kong code of practice on wind effects. However the pressures recorded were greater in some areas of the roof of significance. For example, peak outward pressure for design of roof structure was 425 kg/m² from the wind tunnel tests (cf 300 kg/m² from the code of practice), and the peak outward pressure for design of cladding was 725 kg/m² (cf 480 kg/m²).

Pressure spectra measurements were made over areas of the roof in order to assess the magnitude of the forcing functions at frequencies close to each modal frequency of the structure. At the time of the tests it was not known how stiff the roof structure would be; a dead-weight cable net was still a possibility. The very high non-uniform wind loads indicated by the tests were consequently a key factor in the choice of a rigid lattice shell structure. Subsequent eigen-value analysis to determine the modal frequencies of the structure gave values significantly higher than those for which a dynamic analysis of the structure would be required.

Building geometry

The fundamental element in the geometric definition of the Cultural Centre is the 'wall'. These geometric walls are planes curved and bounded in space such that they are fully coincident with the outer face of the external structural walls and the foyer face of the two main internal structural walls. The full set of wall definition data establishes the reference geometry of the Cultural Centre. All subsequent geometric information, such as cable end system points, is then determined by means of offsets from the reference data. In this way, the data input is minimised and the possibility of geometric incompatibility eliminated. An outline of the reference wall geometry is shown in Fig 3.5, together with the definition of the global X, Y, Z axis system. The vertical ordinate Z is measured relative to the Hong Kong Ordnance Datum. The level Z_p on the outer roof surface of any point P having plan coordinates X_p, Y_p, is determined by applying the principle of radial generators between parapet system lines coincident with the tops of the relevant reference walls. If the parapet system lines are redefined to have a constant normal offset V* below the wall tops in the vertical plane, then radial generators describe a surface parallel to the top roof but offset normally by V* throughout. Thus, in Fig 3.5, Z^1_Q represents the level on this surface of point Q (X_Q, Y_Q). Such surfaces are used to define the position in space of secondary steelwork connections for strut tops, rafters and purlins.

Cable end system lines are defined from the relevant adjacent wall by normal offsets, v* and h*

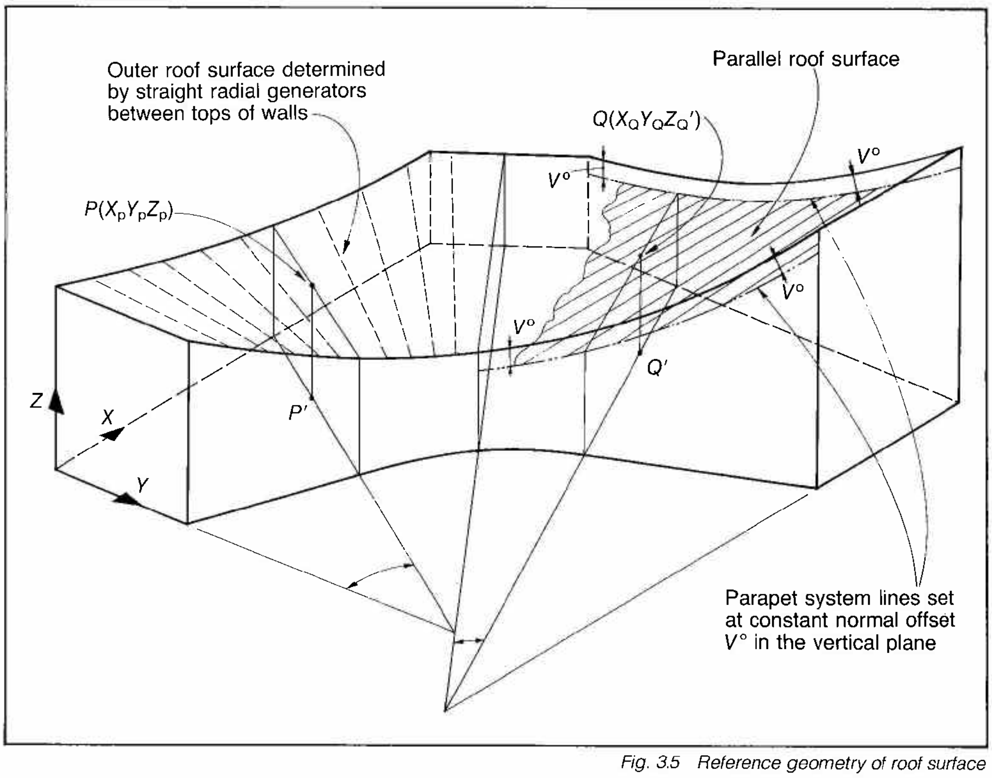

Fig. 3.5 Reference geometry of roof surface

in the vertical and horizontal planes (Fig 3.6). Boundary nodes for the subsequent cable net form finding B are then automatically located on a system line on specification of a single ordinate. On completion of the net form finding by non-linear relaxation, the spatial coordinates of any node N are known. The member lengths for the secondary support steelwork (struts, bracing, rafters and purlins) are then calculated by relating this information back to the geometry of the outer roof and parallel surfaces.

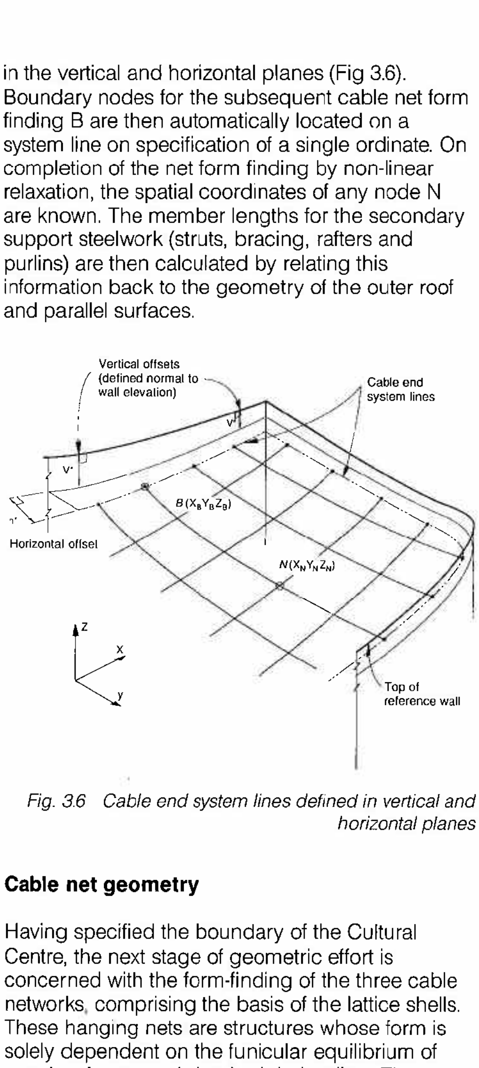

Fig. 3.6 Cable end system lines defined in vertical and horizontal planes

Cable net geometry

Having specified the boundary of the Cultural Centre, the next stage of geometric effort is concerned with the form-finding of the three cable networks, comprising the basis of the lattice shells. These hanging nets are structures whose form is solely dependent on the funicular equilibrium of member forces and deadweight loading. The nets are defined as having uniform link lengths in the unstressed state; consequently the only possible method of controlling the surface is by adjustment of the position of the boundary nodes and the length of edge links adjacent to those nodes. The aim of the form-finding process is to determine a net geometry for the deadload case that has a smooth surface and even distribution of member forces, all of which must be tensile, and which also satisfies the clearance dictates of the structure below and the overall limits on boundary reactions.

Numerical form-finding by dynamic relaxation

The geometrically non-linear analysis for the determination of the net equilibrium forms was carried out by a dynamic relaxation (DR) program suite on a Hewlett Packard desktop computer. This method has proved particularly well suited to the form-finding and analysis of tension systems for several reasons. It can cope with structural mechanisms, gross geometric non-linearity and very large out-of-balance nodal forces while line finite elements can be controlled, either elastically with specified axial stiffness (EA) and slack length, or by direct specification of member tension. On-off non-linearity such as cable slackening is checked and implemented at all stages of the analysis and significant local modification of the structure can be accommodated without violent propagation of disturbance throughout the net, facilitating an interactive approach to the adjustment of form. Further, it is a vector method in which equilibrium and compatibility conditions are separated and dealt with successively until compliance at the converged state. A full description of DR is given in Refs 3.3, 3.4 and 3.5.

Data definition and net adjustment

Integration of the geometric model of the structure into the analysis software permits the automatic location of net boundary nodes on to the cable end system lines described above. Full X, Y, Z spatial coordinates are automatically interpolated on specification of a single X or Y ordinate or of the required length along a particular system line. As this is the only permissible mode of boundary node location, the overall geometric compatibility of net and boundary structures is assured.

The general sequence of net adjustment is therefore quite simply achieved. An initial run with full elastic control of elements immediately adjacent to the boundary provides an overall indication of the distribution of cable tensions, although there may be some cable slackening. Individual cables are controlled in subsequent runs by the specification of slack length of one cable end link and constant tension at the other end. The sag of individual cables, and hence the vertical profile of the net, is adjusted by variation of the specified control tension. Limits to boundary forces can be imposed in the same way.

Having achieved required clearances and stress levels under deadload, the positions of nodes along a cable can be adjusted in an overall sense by altering the specified length while maintaining the control tension at the opposite end. In this way, nodes can be moved along the cable without significantly altering the overall geometry of the surface. On completion of the form-finding, elastic control is re-established in geodesic links, with the slack length calculated from current geometry, member properties and the magnitude of specified tension.

Lattice shell form-finding

For efficient numerical form matching of a desired shape, it is important that the initial specification of boundary data be as accurate as possible. To achieve this, a 1 : 100 scale physical model of the Cultural Centre was built using 6mm plywood walls glued and screwed to a 20mm plywood baseboard, while individual elements were fabricated using 1 : 100 computer plots of wall plans and elevations in order to achieve maximum accuracy. A 60mm mesh of fine chain (equivalent to a 6m grid on the real structure) was suspended from balsa-wood edge beams to model the required net form and, clearances at critical points were checked and adjustments made to achieve a smooth form.

The three cable net fields were then measured individually to locate the boundary nodes (defined by a single dimension as outlined above) and the lengths of edge links adjacent to them, and to define the field topology, thereby providing the initial data for numerical form-finding. Initial positions for internal nodes were located approximately by linear interpolation between boundary nodes, DR dealing with the consequent out-of-balance forces without difficulty.

All initial numerical investigations of cable net forms, and subsequent shell analyses, were based on a square mesh of cables having a 6m slack link length between nodes. Refined smoothing and adjustment of the cable nets was carried out for a 3m grid, with initial data determined by interpolation from the 6m grid values. Final data for the actual 1.5m mesh were established by geometric interpolation from the 3.0m mesh. Slack length correction factors, dependent on local surface curvature, had been automatically included in the 6.0m and 3.0m meshes on the assumption of this final 1.5m spacing. Final net geometry was determined for two self-weight conditions, that of cable only and cable plus lattice shell concrete and reinforcement.

Fabrication geometry

Based on these building and net geometries, the contractors were issued with a full set of geometric data comprising definition of setting-out points for wall parapets and ring beams using both the global coordinate system and a local system measured along the length of each wall; global coordinates of cable node positions under cable self-weight and under the dead load of the concrete lattice beam (ballast loading); cable slack lengths and tensions under ballast load; the geometry of the main struts supporting the roof purlins and cladding; and the final roof surface levels, by global coordinates on a 5m grid.

Lattice shell analysis and design

A computer model had to be derived capable of accurately predicting the behaviour of the lattice shell within the constraints of available computer resources. To have modelled just one of the lattice shells using each individual member on the actual 1.5m grid would have required over 1700 nodes, corresponding to 10,200 degrees of freedom. This was beyond the capacity of a medium sized computer. Fortunately, the lattice shell can be modelled accurately by spatial beam elements on a coarser grid than that of the real structure. However, careful attention must be paid to the assignment of the section properties to the pseudo-members so derived.

Lattice shell member properties

The axial, torsional and out-of-plane bending stiffness of these pseudo-members are simply the direct sums of the individual member values. Because in-plane forces in the diagonal direction of the lattice are resisted by shear forces between the lattice shell members, the coupling of the members must be considered. Williams (Ref 3.6) has given the following derivation of the scale factor to be used.

Attempts to derive a finite element with four corner nodes which accurately models the in-plane behaviour of an n x n lattice have failed due to excessive complexity. An alternative approximate approach to understanding this in-plane response is to consider an orthotropic plate subject to plane stress. Here,

$$\begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} = \begin{bmatrix} A & D_2 & 0 \\ D_1 & B & 0 \\ 0 & 0 & C \end{bmatrix} \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \qquad (1)$$

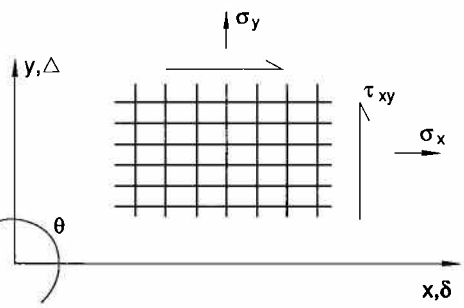

Fig. 3.7 In-plane forces in the lattice shell

where the force terms σ_x, σ_y and τ_{xy} are defined by Fig 3.7, E = Young's modulus and the corresponding strain terms are ε_x, ε_y and γ_{xy}. In the case of a grid shell, $D_1 = D_2 = 0$ in the stress-strain relations. If the members in each direction are the same:

$$A = B = E\left[\frac{\text{(cross-sectional area of members)}}{\text{(spacing between members)}}\right] \qquad (2)$$

Consider then an assumed displacement function for the nodes:

$$\delta = \gamma_{xy} \cdot y \qquad\qquad \Delta = \gamma_{xy} \cdot x \qquad (3)$$

such that

$$\varepsilon_x = \varepsilon_y = 0 \; (\theta = 0) \qquad (4)$$

Individual lattice member in-plane distortions are defined by Fig 3.8.

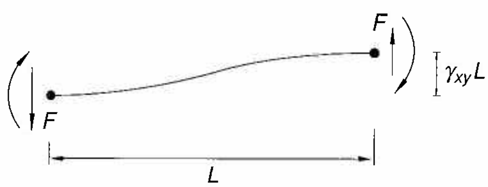

Fig. 3.8 In-plane distortion of individual lattice member

The corresponding forces are given by:

$$F = \frac{12EI_{zz}}{L^3} \cdot \gamma_{xy} \cdot L$$

$$\therefore \tau_{xy} = \frac{F}{L} = \frac{12EI_{zz}}{L^3} \cdot \gamma_{xy}$$

and $C = \left(\frac{12EI_{zz}}{L^3}\right)$ *in the stress-strain matrix.*

Therefore, in-plane bending stiffnesses must be modelled proportionally to L^3 when a coarser analysis mesh is employed, while axial, torsional and out-of-plane bending stiffnesses are proportional to L. A small lattice shell model was set up with fine and coarse grids, using the scale factors above. This produced results that supported the above derivation. The modelling employed for the lattice shell is shown in Fig 3.9.

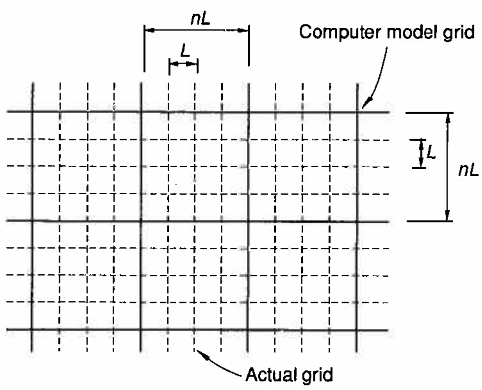

Fig. 3.9 Computer model grid employed for lattice shell

Boundary conditions

In order to reduce the downward deflection and increase the rigidity of the lattice shell, the connection between the latter and the ring beam was generally designed to be fully fixed. There are movement joints in the supporting concrete superstructure which for obvious reasons cannot be extended into the lattice shell. Consequently there will be relative temperature movement between the lattice shell and the ring beams (Fig 3.10 a, b, c). Where the length of the end lattice

Fig. 3.10 (a) Movement joints in superstructure

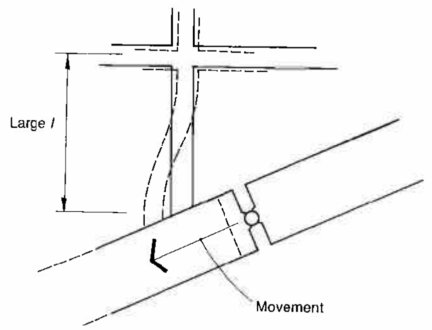

Fig. 3.10 (b) In-plane bending of long end members at boundary

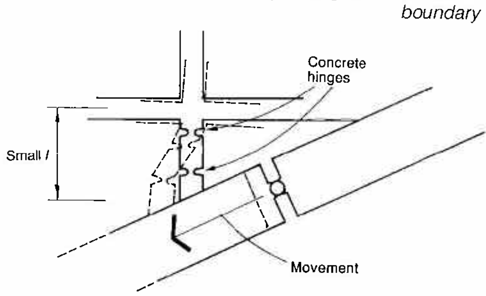

Fig. 3.10 (c) Concrete hinges introduced in short end members at boundary

shell members is sufficiently large, the temperature movement can be accommodated by in-plane bending in the members. However, where the end member length is small, this bending movement cannot be accommodated elastically and concrete hinges were introduced.

Working load analysis and design

The geometry established in the form-finding process was transferred to a linear stiffness analysis program which utilised rigidly jointed straight space frame members. As deflection under working load will be small in relation to the overall spans, a conventional linear analysis was considered satisfactory for ensuring adequate working load member design. Here the lattice shells were analysed for a number of combinations of funicular dead loads and non-funicular imposed and wind loads. Concrete members were designed for the most critical combinations of imposed, wind and temperature loading. Each beam element was checked and suitably reinforced for tensile or compressive axial loads, in-plane bending moments, out-of-plane bending moments, torsional moments and shears.

Evaluation of the collapse load

In addition to the linear analysis for working loads, an acceptable factor of safety against collapse is required, as is an investigation of the dynamic behaviour to ensure that there is no significant dynamic response. Two aspects of non-linearity were considered – geometric non-linearity, which takes account of the deformed geometry, and material non-linearity of reinforced concrete.

Analytical procedures

Two numerical approaches are in common use. A direct eigen-value method generally overestimates the critical load factor, while an iterative application of non-linear Newton-Raphson analysis requires much more computational effort, but produces a lower bound to the solution and provides more information on the structural behaviour up to failure. The eigen-value method will in particular overestimate the critical load factor for this structure, because wind loads are significantly non-uniform and there is no mechanism for updating member stiffness as this changes with large changes in bending moments and forces in the concrete members. Eigen-value analysis is nevertheless useful in elucidating the collapse behaviour of a complex structure, and from an examination of the modes an insight can be gained into means of stiffening the structure.

Eigen-value extraction

Eigen-value extraction involves finding the roots of the polynomial. The more usual transformation methods, such as those of Jacobi and Householder, are best suited to extraction of all the eigen-values where matrices are small and fully populated. However, they are expensive in computer time and inaccurate for large problems. An efficient solution algorithm using the subspace iteration method was applied to extract the lowest 30 eigen-values and their characteristic modes.

Material non-linearity

Cracked concrete members subjected to both axial loads and bending moments have stiffnesses which are considerably different from uncracked ones. The essential computational problem of material non-linearity in materials such as reinforced concrete is that equilibrium equations must be written using material properties that are strain dependent, although strains are not known in advance. Figs 3.11 a, b show the stress-strain relationships for steel and concrete as defined in CP 110 (now BS 8110). The stress-strain relationships for the cables were based on cable manufacturers' material data.

In order to determine the appropriate value of stiffness for a given load increment, axial loads and moments were used to derive the stresses and strains in the concrete, reinforcement and cables (Fig 3.12). The simple theory of bending provides a direct relationship between the radius of curvature and the member bending stiffness.

The curvature l/r is derived from the strain diagram: when applied to a cracked section the relationship is:

$$\frac{1}{r} = \frac{M}{E_c I} \qquad (7)$$

and the apparent second moment of area is

$$I_{cr} = \frac{Mr}{E_c} \qquad (8)$$

Hence the applied loads can be determined from the stress-strain relationship shown in Fig 3.12.

$$N = \int_0^{\bar{x}} (f_{cx}\, dxb) + f_{sc}A'_s + f_sA_s + f_{sca}A_{ca} \qquad (9)$$

$$M = \int_0^{\bar{x}} f_{cx}\, dxb\left(\frac{h}{2} - d_c\right) + f_{sc}A'_s\left(\frac{h}{2} - d_1\right) + f_sA_s\left(\frac{h}{2} - d_3\right) + f_{sca}A_{ca}\left(\frac{h}{2} - d_2\right) \qquad (10)$$

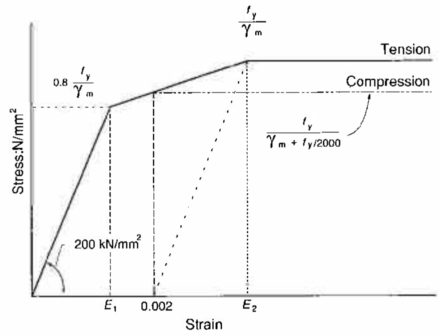

Fig. 3.11 (a) Short term stress-strain curve for steel reinforcement

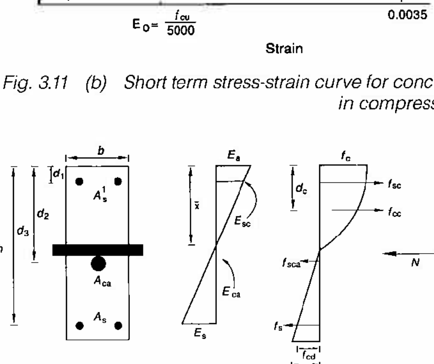

Fig. 3.11 (b) Short term stress-strain curve for concrete in compression

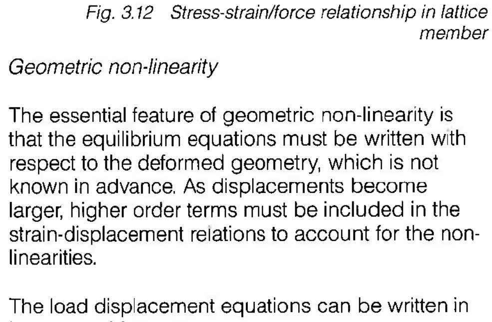

Fig. 3.12 Stress-strain/force relationship in lattice member

Geometric non-linearity

The essential feature of geometric non-linearity is that the equilibrium equations must be written with respect to the deformed geometry, which is not known in advance. As displacements become larger, higher order terms must be included in the strain-displacement relations to account for the non-linearities.

The load displacement equations can be written in incremental form as:

$$\{ [K] + [G] \}\{ \Delta\delta \} = \{ \Delta P \}$$

Here the tangent stiffness $\{ [K] + [G] \}$ is the sum of the elastic stiffness matrix $[K]$ and the geometric stiffness matrix $[G]$. The elastic stiffness matrix is dependent on the elastic member properties and the current geometry; the geometric stiffness matrix depends on the current geometry and current member forces thus depending linearly on displacements. Higher order stiffness terms, those dependent on the square of displacements, could have been included but were considered unnecessary in this case.

Solution to non-linear equations by Newton-Raphson analysis

The choice of solution algorithm is to some extent problem-dependent. The more non-linear the problem, the more frequently the stiffness matrix must be updated. In this case, the modified Newton-Raphson method proved reliable and was independently verified against published non-linear space frame problems. These results had themselves been validated previously by an independent dynamic relaxation based solution (Ref 3.4) which considered geometric non-linearity.

The Newton-Raphson method is an extension of the conventional linear stiffness analysis of a framework, using step-by-step incrementation of the load factor and stiffness matrix until a bounded solution to the buckling problem is obtained, with all non-linear terms updated at each iteration (Fig 3.13). If the applied load is greater than the buckling load, no solution is obtained as the stiffness matrix becomes ill-conditioned. The load corresponding to this situation is an upper bound to the buckling load, while the lower bound is the highest load at which a converged solution is obtained.

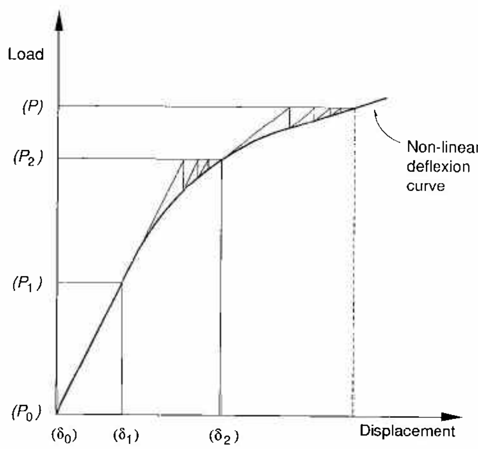

Fig. 3.13 Modified incremental Newton/Raphson iteration

As the iterations proceeded, the program provided graphical plots at selected nodes of the load-displacement curve, permitting immediate visual feedback on the increasing non-linearity as the buckling load was approached.

Interpretation of results

The analysis showed that the minimum factor of safety against buckling for any of the lattice shells was 1.7 for a 3s wind gust of 139 knots (peak upward wind pressure of 425 kg/m^2). In any case, partial as opposed to complete collapse would occur, because the roof must drop to its tensile form when the wind loading has passed. The load to cause total collapse is therefore the downward load required to cause members to fail in tension. As the downward wind load is significantly lower than the upward wind load, a considerably greater factor of safety is present for this load case.

A factor of safety of 1.7 might not normally be considered satisfactory, but was so deemed in this case as the stiffness of the lattice shell only had been taken into account. Further stiffness existed in the steelwork support system to the roof deck. Although this contained expansion joints it did provide local stiffening, and the analysis was re-run adding stiffnesses to some of the members in the primary direction. By this means the minimum factor of safety was increased from 1.7 to 2.5.

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Tsim Sha Tsui Cultural Centre – Construction of the Roof System

The relatively complex concept for the structure of the Cultural Centre roof led to the view that the expertise required for construction might not be available in Hong Kong, and that an international tender should be called. The procedure adopted was a prequalification exercise open to both local and foreign contractors. Each applicant was supplied with a detailed document setting out the engineer's proposed method of construction in the form of written text with a series of drawings showing a step-by-step construction sequence. Each contractor was asked to suggest a method of construction, and to demonstrate the ability, skills and experience they could apply to the project.

The exercise produced a mixed response, ranging from well thought-out schemes through to tenderers who would have clearly found the organisation and management required beyond their capability. Tenderers were graded in accordance with their response, and a minimum tender list of six firms was drawn up. The contract was organised on the basis of tender sum and final response to construction method statement – the successful tender being returned by Kumagai Gumi (HK) Ltd.

For work of this nature, it is important to have a contractor who understands both the accuracy of setting out and fabrication of cable lengths detailed in the specification, and the influence of any errors on design. Kumagai assembled an impressive team assisted by Tokyo Ropes of Japan and throughout the preplanning and erection on site, provided a chartered civil engineer, David Westwood, as site manager, with Mr Higashi as specialist engineer for the roof construction. The design team insisted that the contractor produce an erection procedure manual well in advance of the work on site. This ensured that nothing was left to the last moment and that all procedures including setting out and surveying methods for the roof surface, method of pulling the net into the correct form before concreting, method of shuttering, and attachment of shuttering to the cables, effect and sequence of casting on the roof slope, and the erection of secondary steelwork and installation of the cladding were thoroughly tested before implementation.

Work off site

The specification further called for testing of several features before manufacture, the contractor having to demonstrate that the cable would attain the specified minimum breaking load together with the required axial stiffness (EA value). The contractor fully appreciated that failure to achieve this value would prevent the roof from adopting the desired shape under ballast weight, so requiring extensive adjustment on site. Tokyo Ropes made 3m long samples of Warrington left-hand lay cable, which were tested to demonstrate that the EA could be obtained with a tolerance of 300 kN. Tests were also carried out to determine the minimum breaking load for the swaged-on end fittings. A further series of tests was performed on cable clamps (Fig 4.1), which had been specified to have negligible slip under a clamping load of 1t. Experience has shown that considerable care is required to achieve the correct balance, as over-torquing can cause breakage of clamps or damage to cables. Kumagai also amended the cross-clamp detail to provide support for the concrete shuttering system and the base plate of the secondary steelwork. A locking plate on anchorage castings provided for lateral adjustment of ± 20mm (Fig 4.2) and longitudinal cable adjustment was to be achieved by means of a locking nut on the swaged-on end fitting.

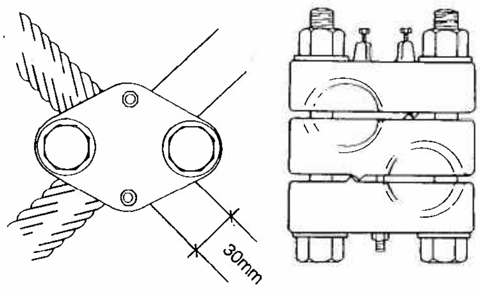


Fig. 4.1 Plan and section through cable clamp

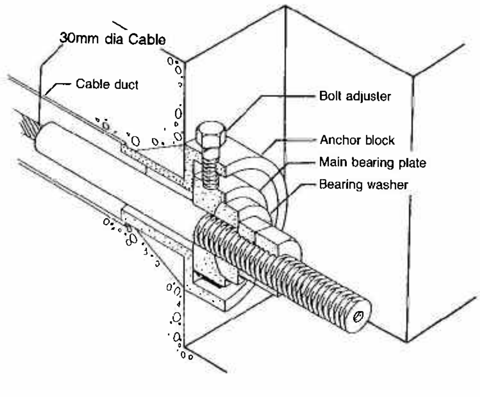


Fig. 4.2 Isometric of cable anchor assembly

If final adjustments are to be minimised, accuracy of fabrication is essential for cable roof work. Each cable, with a maximum length of 90m, had to be laid out in the works, stretched with an initial load and marked at 1.5m intervals to locate the cable clamps. A marking load of 6t, representing approximately 20% of the final working load was sufficient to ensure that the cables were straight, but not too great, to invalidate the form-finding analysis. After marking, the cables were tagged and wound on to correctly sized drums to suit the sequence for erection on site.

Choice of roof cladding

During the last decade, large span roofs in Hong Kong have generally been covered with roofing membranes, some of which have failed during typhoons. The design team were of the opinion that due to the intense ultraviolet light of the local climate, the life expectancy of these materials was limited to 10–12 years, an unacceptably short period for the development. An aluminium cladding system with an expected life of 40 years in the Hong Kong environment was therefore chosen as an alternative. The system offered by the contractor was that of Sky-Zip, which held a licence for manufacture of the Kaiser Aluminium Kal-Zip system. The panels had upstand ribs and were rolled on site in lengths of up to 90m, thus avoiding end laps and ensuring that the side laps were above the roof water level. A sliding clip fixing allowed free expansion at either end. Unfortunately, it was later discovered that Sky-Zip had not been provided with the latest development on the clip designs, so necessitating a series of tests to check the pull-off capacity, and static and dynamic testing of the panel and clip assembly. A test of the complete system under dynamic loads while subject to a rainfall intensity of 400mm/h was also conducted to study the watertightness of the system.

Preliminary studies

Tender documents stated that all setting out should be performed using distomat theodolites, sighting to targets attached to the cable nodes on a 10m grid. Coordinate data had been produced allowing the cable net to be checked under its own dead weight and again after the ballast loads had been applied. The roof form was specified at 20°C, the contractor being required to make survey corrections where necessary to allow for variations in cable lengths due to temperature variations.

It must be appreciated that a cable net roof assumes a form based on applied loads. Thus, under the self-weight conditions due to the dead weight concrete, the roof would deflect by up to 0.6m. It was therefore necessary to pull the cables down into their final form before concreting. This would be achieved by a system of ballast weights attached to cable nodes on a 3m grid, chosen because it maintained a reasonably smooth cable profile and resulted in a manageable ballast load of 1t. However, even using this coarse grid, around 700 nodes would need ballasting.

Various alternatives were considered, including a system of springs which could be attached to the structure below, automatically relaxing as load was applied. However, until the shuttering system was designed, it was not possible to determine accurately the excess ballast required. The ballast attachment system had also to be capable of easy removal as concreting proceeded, so preventing distortion of the roof form under the combined weight of concrete, ballast and shuttering. Eventually, the relatively simple system of concrete blocks ballasting was adopted, although this required the removal of some 300t of blocks at the completion of the project.

Construction of roof

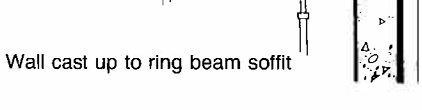
The chosen roof construction sequence is outlined in Fig 4.3 a-g, from the construction of ring beam through to the final installation of cladding.

Ring beams

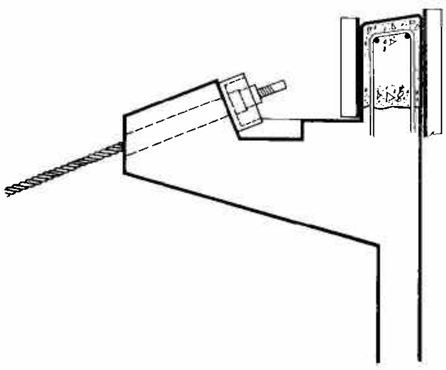
The fixing of ring beam formwork, installation of reinforcement, fixing of anchor blocks and associated cable ducts, and completion of formwork and casting was comparatively straightforward. To ensure that the anchor blocks were located accurately in space before concreting, they were set up over survey points marked on the formwork, surveyed again before casting to allow adjustment, and finally surveyed after concreting. At this final stage adjustment was possible using the sliding bearing plate in the anchor and the threaded terminal at the end of the cable. Fine tuning was carried out using a special indicator set into the main bearing plate, which defined a particular point along the cable axis to be taken as the end point. With a dummy bearing plate set correctly in position, the face of the anchor block was sprayed with paint, leaving a clear indication of where the final bearing plate should lie once the cable net was installed. In the event, axial shortening of the formwork props under the weight of the wet concrete caused some drift of the anchor points, but never beyond that retrievable using the adjustment available at the anchor block face.

With ring beams cast at a maximum inclination of 40º, it was found impractical to place top formwork, as the top surface included the opening for anchor pockets at approximately 1m centres, and would have required some very time-consuming carpentry. In these steep areas the concrete slump was reduced and concrete was poured in three layers to ensure adequate vibration at all levels. It was found that the top 50mm was less well compacted than the remainder, but cores confirmed reasonable compaction below this level. As beams in the steeply sloped areas were generally cast approximately 50mm deeper than required, this top layer was not critical.

Fig. 4.3 (a–g) Ring beam and lattice construction sequence

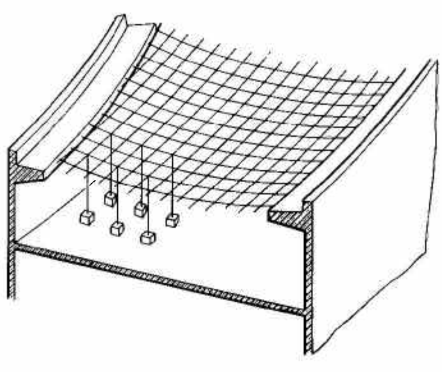


(a) Set up reinforcement, anchor block and duct within ring beam formwork; set out anchor blocks to surveyed marks on shutters

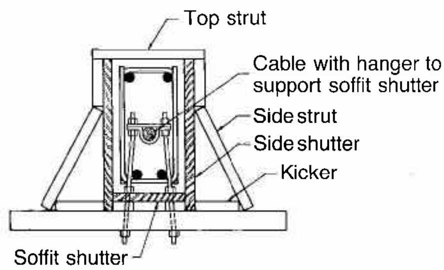


(b) Cast ring beam; adjust cable bearing plates to final position, install cables; cast parapet

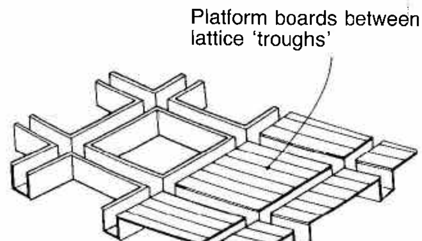
(c) Clamp up cable net nodes, survey self-weight condition, adjust cable lengths at anchor blocks



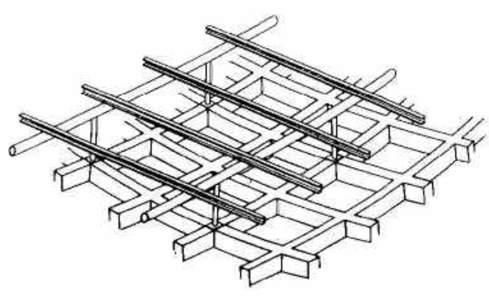
(d) Prestress cable net by hanging ballast at alternate nodes, ballast blocks suspended just above floor slab with wedges placed lightly underneath



(e) Hang soffit shutter beneath cable, fix reinforcement and fit side shutters



(f) Cast concrete forms using platform boards between lattice 'troughs' sitting on formwork



(g) Strip formwork, erect steel struts, rafters and purlins, install cladding

Cable net

The 30mm galvanized steel 34 wire strand cables which had been prestretched to 50% ultimate strength and cycled to remove construction stretch, were all prefabricated to length with swaged-on threaded terminals at both ends, and premarked at cross-clamp centres. Cable marking was carried out using a nominal 6t tension, with a compensatory length adjustment. The specification for cable works stated that a cable E value of 16,000 kg/mm² had been assumed with metallic area 553mm² as a basis for the definition of initial and final roof geometries. Tests carried out in Japan immediately after prestretching showed the cable to be within specification with an ultimate tensile strength of 86t and a maximum working load of 25t.

Before cable installation commenced, three cable samples were tested at Hong Kong University to demonstrate the ultimate strength of the cable and end fitting. It was found during these tests that the E value was non-linear and below that specified. Further tests were carried out in Japan using a representative 70m length of cable. These confirmed that the E value was low at low tensions, but did not show whether or not this non-linear effect had arisen after relaxation, and therefore shortening, of the cable following its first prestretching. If the cables had all become shorter due to relaxation, it would be expected that, once installed on the roof and prestressed, the reduced E value would enable them to reach their correct final lengths. To check this condition, four cables were hung in place on the roof and their length determined by surveying the catenaries. Results show that they had shortened after fabrication. The cables were then loaded with hanging ballast weights, and the resulting catenaries were surveyed. This demonstrated that an initial shortness of 0.1% could be expected and that on reaching a typical working prestress tension of 25t the actual catenary would match the theoretical one (Fig 4.4).

Cables were then installed between ring beams using a simple rig (Fig 4.5). Each cable was loaded on a rotating drum, and one end was pulled out to its anchor point. Primary cables for concert hall and theatre had to be pulled up slopes of up to 40°, using a motorised winch set up on the upper ring beam. A working platform of either bamboo scaffolding or existing concrete floors was available below roof level. Once all primary cables were installed, the bottom and middle sections of the cable cross-clamps were clamped lightly to their marks. The secondary cable could then be laid out on top of the primaries, using the clamps as stops against slipping down the net. Finally, all clamps were completed and tightened (Fig 4.6).

Anchor blocks allowed for adjustment in the location of cable ends. As all anchor positions had been surveyed before cabling, the required lateral adjustment was marked on the face of the anchor and then introduced when cabling. Similarly, the required length of thread protruding through the anchor was adjusted to correct any errors in the direction along the cable.

A survey of each net under self-weight was then carried out. Theoretical computer data gave a geometry at 20°C and assumed a linear E value, but

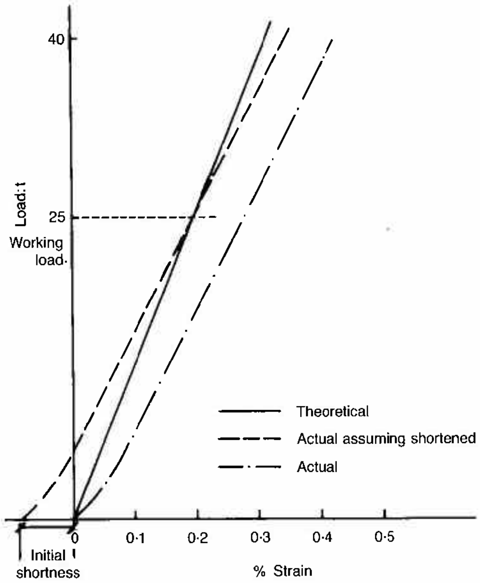


Fig. 4.4 Typical cable stiffness curve

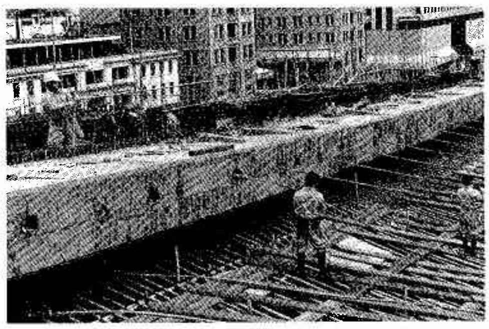


Fig. 4.5 Work on cable erection

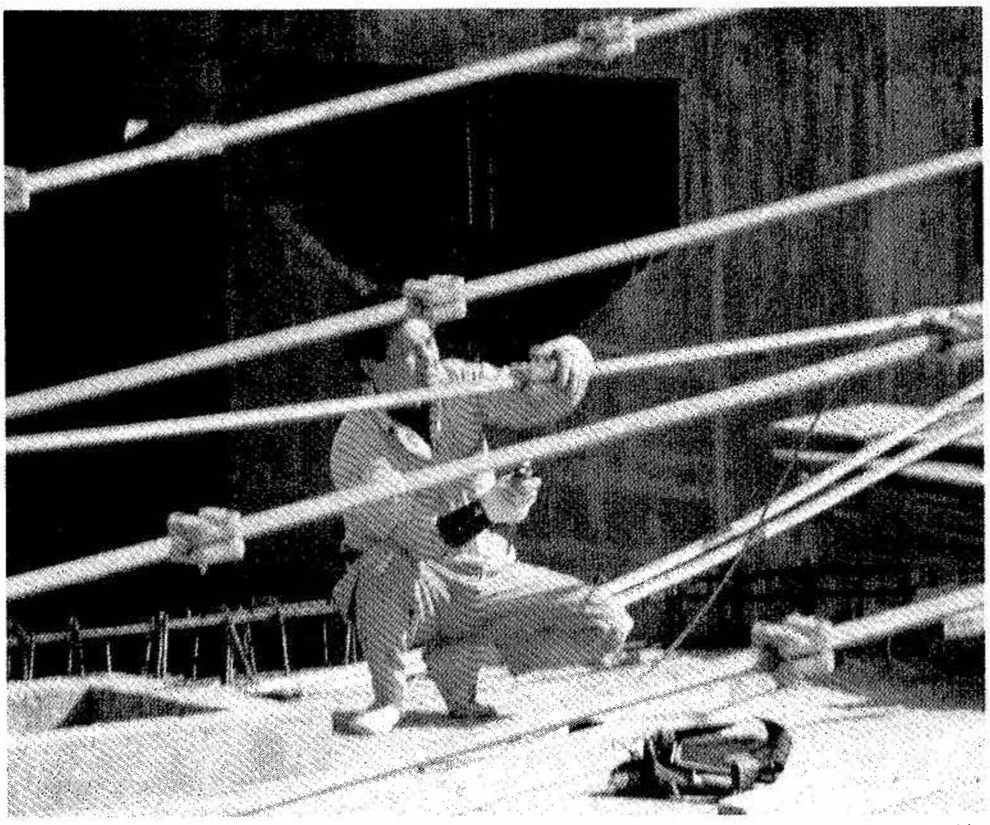


Fig. 4.6 Fixing of clamps on cables

it was found necessary to adjust this to allow for different survey temperatures and for the cable shortness/non-linear E value shown in earlier tests. A second set of geometry at known average cable tensions (final state) was available, and was used as a basis for interpolating other geometries at various strains/temperatures. The hanging net shape could hence be checked to ensure that it conformed sufficiently well with that required to achieve the final specified prestressed shape. Surveying was carried out early and late on a preferably cloudy day, when cables were in the shade and the temperature was stable. Three theodolites were used at fixed locations, and easting and northing readings were taken on a target specially developed by Hong Kong Polytechnic for Kumagai (Fig 4.7) located on selected cable net nodes. From the three sets of readings, a best point of intersection of the 'rays' could be found. As the target included upper and lower sighting pins, the direction cosines of the target (set normal to the cable net) could be calculated, enabling the survey point data to be extrapolated down to the true intersection point between cables.

Once the cables had been surveyed, they were stressed down into the final theoretical profile. This required the application of 316 kg per node, and was achieved by adding 1t at alternate nodes (250 kg per node) plus the continuous weight of soffit formwork and reinforcement which was hung from the cable net. The 1t loads were applied in the form of concrete blocks hung from cross-clamps (Fig 4.8 a,b). Hanger lengths were set such that each block was suspended just above the nearest concrete floor slab below. Once the extra formwork and reinforcement weight had been added, wooden wedges were eased under the blocks so that block and slab were effectively in contact. This adequately ensured that as the weight of poured concrete was added, hanger cable tensions would release automatically. In effect, the ballast load would be exchanged for the concrete load. However, as the roof stiffness out of plane was greater than the direct stiffness of the suspension cables, there would inevitably be a little movement and a little residual tension in the hangers. This was taken out during concreting by the release of a rigging screw in the cable.

Formwork

The original expectation, expressed in the tender drawings, was that the contractor would cast concrete sequentially enabling the re-use of a set of simple shuttering units to complete the whole. With the cable net fully prestressed using the ballast blocks, such a procedure was feasible without causing local distortions. However, in practice the contractor opted to install formwork to one complete cable net before starting concreting. As cable lengths were based on the original loading assumptions, which did not include a formwork

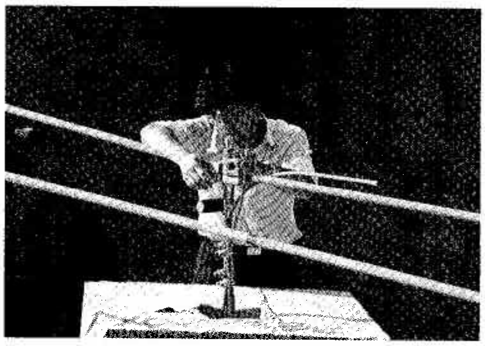


Fig. 4.7 Specially developed survey target located on selected cable net nodes

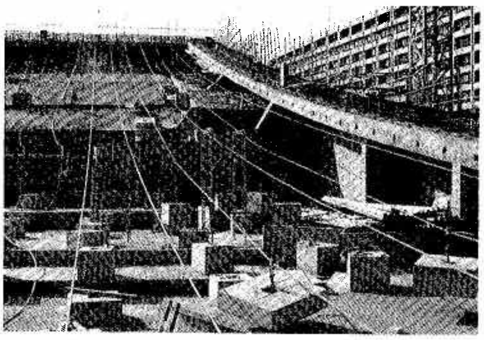


Fig. 4.8 (a) 1t concrete blocks hung from cable cross-clamps

Fig. 4.8 (b) Concrete blocks suspended just above concrete floor slab

component in determining the final geometry, adding the extra weight of formwork to the original cable system was predicted to cause the roof to hang lower by up to 100mm.

Furthermore, because very large areas of concrete were cast at one time, the curing temperature would cause expansion of the cables and an additional sag in the net. To overcome these problems, it was decided to prop the formwork to the concert hall and theatre nets where the roof slabs of the respective

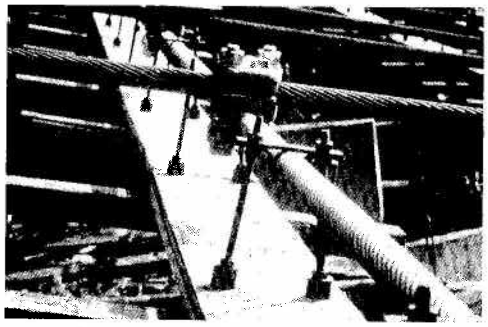


Fig. 4.9 Detail of shutter attachment by clamp and threaded rod for lattice beam formwork

halls were conveniently close to the cable net. The foyer cable net, where the ground slab was approximately 20m below, did not require such propping. As primary cables span comparatively short distances and are more tightly curved than those of the other two nets, the effects of excess weight and raised temperatures were not so significant. Based on experience of casting the concert hall roof lattice and the need to ensure a very smooth final profile for the exposed lattice beams in the foyer, the anticipated errors were considered acceptable. Propping, which was only required to support additional formwork weight and to avoid excessive thermal deflections, was therefore reasonably light and quick to install. Timber and scaffolding were set up to prop the lattice beam soffit at 3m centres.

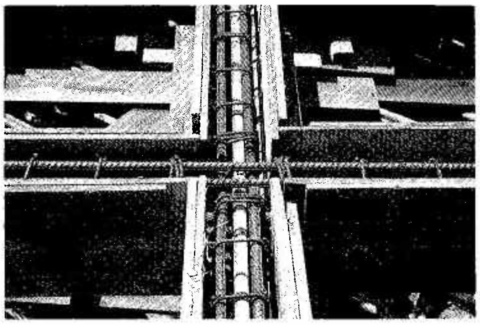
Lattice beam formwork was a simple timber system of side walls and soffit, hung from the cables using special clamps and threaded rods. The soffit along primary cables could continue under nodes, but that under secondary cables was confined to short lengths abutting the primary soffit (Fig 4.9). The reinforcement was placed before the side formwork was installed. An important requirement of the formwork design was to ensure that sufficient articulation was available to allow free movement of the cable net under the weight of the wet concrete. A test rig was set up on site to model the system. The full sequence from formwork placing, through concrete casting, positioning of steelwork to installation of cladding is detailed in Fig 4.10 a–k.

Lattice beam casting

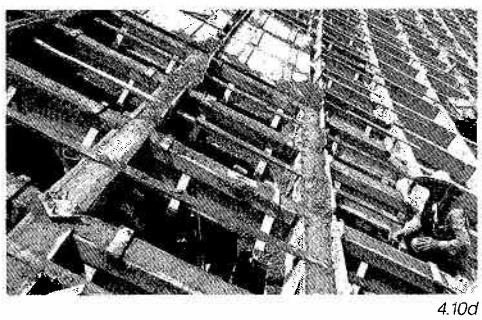
The beams of the lattice are only 160mm wide, and the placing of concrete in this restricted space required some prior planning. To minimise waste and provide easy working access, timber platforms were laid into the open squares between beams with lattice beams becoming troughs below the general level of working platforms. Premix concrete was delivered to the roof in small skips, and could be run alongside each beam line very easily. Work proceeded rapidly with a workforce of about 20

Fig. 4.10 (a–k) Construction sequence from formwork placing to installation of aluminium cladding


4.10a


4.10b


4.10c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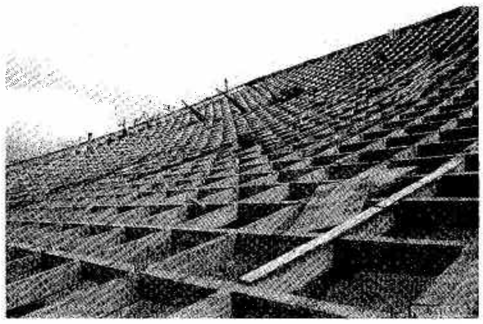
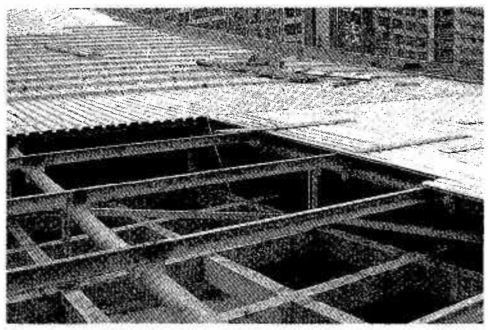
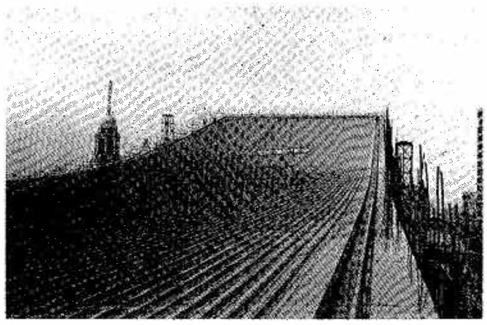
4.10e


4.10f


4.10g


4.10h


4.10j


4.10k

men, and lattices of the concert hall and foyer were each successfully cast in three days, and that of the lyric theatre in six.

The concrete used had a very low water content in order to achieve a high Young's modulus, and slump could be controlled quite closely by variation in the proportion of plasticiser. On steeply inclined parts, it was possible to reduce this slump to assist placing. However, with slopes steeper than about 20°, an extension was made to the side formwork along the lower side of transverse cables, thus facilitating vibration of the concrete to a higher level. After a period of about 45 minutes, the excess concrete could be trowelled away and the top surface made good.

Survey of the three lattice roofs after concreting showed lattices to be on average 15mm below the theoretical net geometry, the level of error depending on the softness (flatness) of each roof. The theatre net proved the most sensitive with props suffering some sway and elastic shortening, as the height between net and supporting slab exceeded 6m in many areas. The accuracy achieved resulted in a deflection/radius of better than 1:1000. Fig 4.11 shows the finished concrete lattice from below, as visible in the foyer.

Secondary steelwork

Cladding was to be installed on a layer of purlins of

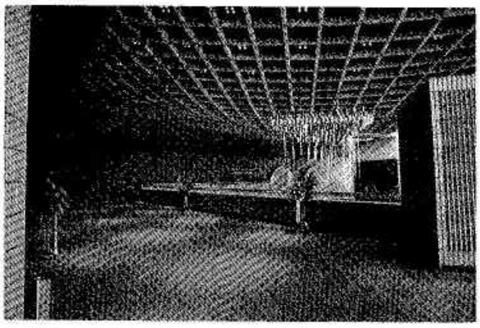


Fig. 4.11 Finished concrete lattice of roof, viewed from cantilevered gallery of foyer

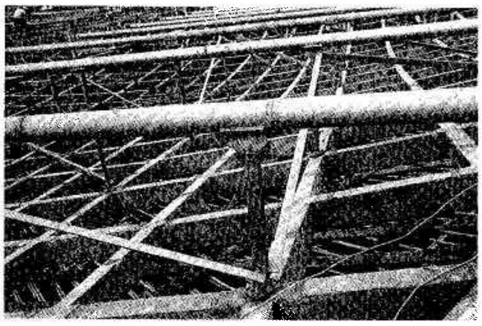


Fig. 4.12 Detail of CHS rafters supported on angle struts fixed to lattice beam

120mm × 60mm × 7mm channel section at 1.2m centres, but at 0.6m centres in areas of maximum suction. Purlins span between 140mm diameter circular hollow section rafters which run along alternate primary lattice beam lines. These rafters are supported on 75mm × 75mm × 8mm angle struts which are pinned at the bottom to baseplates on bolts cast into the lattice beam at alternate nodes at 3m centres. To achieve the required final roof surface, these struts range in length from 100 to 3000mm with lateral stability being provided by intermittent angle bracing between top and bottom pins on adjacent struts. (Fig 4.12)

All steelwork was prefabricated at a local yard to a schedule based on the specified geometry of the roof and lattice beams produced as part of the design. As this was based on theoretical geometry, it was important that the lattice beams should be cast to a high degree of accuracy to ensure that all steel would fit. Steelwork was assembled on the ground as 'trusses' up to 30m long, including rafters, associated struts and any bracing, which were then lifted on to the roof and secured to the holding down bolts. For each rafter section, a minimum of three holding down points were surveyed on the top surface of the lattice beams. At these points, the shim levels at the bases were set to compensate for any errors between actual and required theoretical

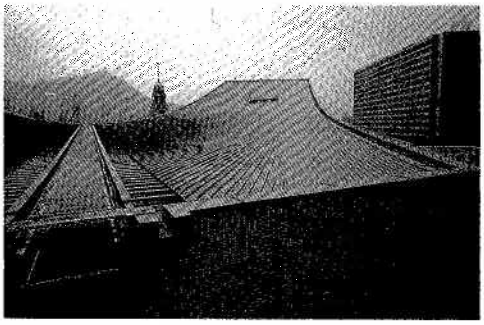


Fig. 4.13 Sweeping surface of completed Cultural Centre roof

level. Having set three points on the rafter correctly, the others tended to follow and were shimmed to their natural level.

With the rafters installed, purlins were lifted on to the roof and clamped in place. A final smoothing check was carried out visually on the purlins with the help of a flexible rod laid across several of them. Final adjustment was made by a combination of reshimming under the struts and adding extra shims

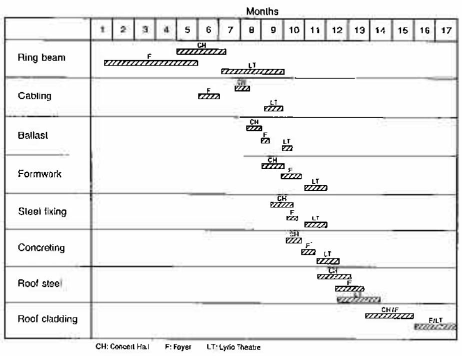


Fig. 4.14 Programme for roof construction

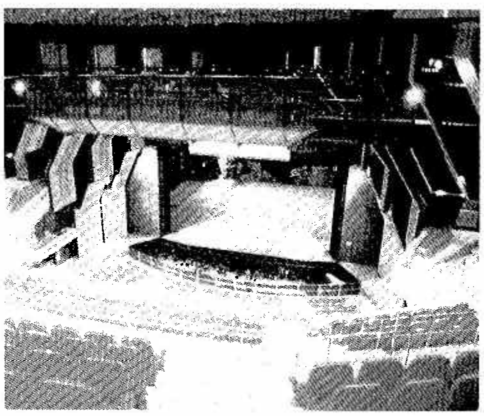


Fig. 4.15 Proscenium of lyric theatre

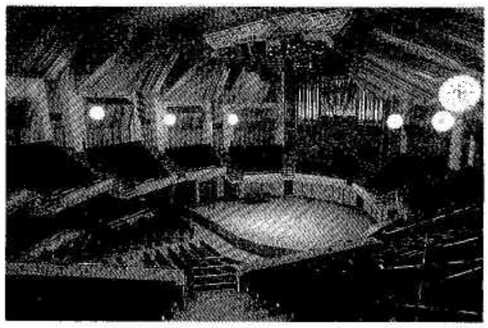


Fig. 4.16 Auditorium of concert hall

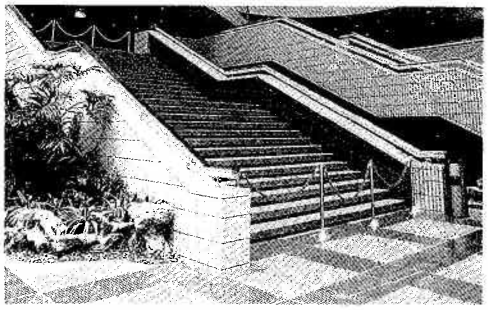


Fig. 4.17 Graceful stairways to foyer galleries

under the purlin clamps. It was found that the steelwork fitted together with little difficulty – a result of the accurate construction of the lattice grid and the amount of available tolerance in the joints.

The roof structure for the Tsim Sha Tsui Cultural Centre is undoubtedly unusual and quite complex (Fig 4.13). However, the experience of its construction showed that in spite of its complexity it was not particularly difficult to build. The actual roof construction programme detailed in Figure 4.14 was satisfied, the whole roof structure being completed within 13 months. Practicability of design was a major factor in this success, as the engineers recognised that practical methods of construction should be conceived as an integral part of their work. In addition, it proved important that the contractor should be aware not only of the need to employ specialist fabricators and subcontractors, but also of the benefit of providing his own specialist engineer, with an understanding of the complexities of the design and a willingness to develop the construction methods required to ensure successful completion of the project.

Completion of the Tsim Sha Tsui Cultural Centre was successfully achieved in 1988 (Figs 4.15, 4.16, 4.17). The complex was officially opened by the Prince of Wales in October 1989, and today forms a prominent landmark of the Hong Kong waterfront.

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