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Two International Finance Centre

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Abstract

Two International Finance Centre (2IFC) is the tallest building in Hong Kong and 5th tallest in the world at 420m high. It is located on the harbour front of the Central District, the business centre of Hong Kong. This paper describes some of the geotechnical design considerations, the structural design development and construction of the tower. Specifically, the paper addresses the large diameter cofferdam solution which was adopted for the foundation; the lateral response of the structure; the issues influencing the floor system; and the detailed design of the megacolumn and outrigger lateral stability system. The paper also addresses construction-led aspects of the design adopted to reduce the construction time of the tower.

Keywords: Tall Building, Composite Construction, Outriggers, Construction-led Design.

1. Introduction

The IFC Tower forms part of Hong Kong Station Development on the Central Reclamation Hong Kong. Developers of the site are Central Waterfront Properties (CWP) - a joint venture between Sun Hung Kai Properties Ltd, Henderson Land Development Co Ltd, Bank of China Group Investment Ltd and the Hong Kong & China Gas Co Ltd. The Mass Transit Railway Corporation (MTRC) is also a development partner of the joint venture company. Architects for the project are Cesar Pelli & Associates and Rocco Design Limited with Ove Arup & Partners (Arup) providing structural and geotechnical engineering consultancy services. Mechanical and electrical consultancy services were provided by J. Roger Preston Ltd.

The tower comprises 88 storeys with a basement level of -32.0mPD and a level to the top of the roof feature of +420.0mPD. The footprint at the base of the tower is 57m x 57m. Towards the top, a series of staggered step-backs reduce the plan dimensions to 39m x 39m at roof level. In all, it provides a gross floor area over 180,000m² of grade A office accommodation. One of the main requirements of the brief was to provide open plan floors and to incorporate a large degree of flexibility such that the

requirements of major tenants (such as financial institutions) could be accommodated.

The tower is just one element of a major new commercial development in central Hong Kong. The development as a whole provides office, retail, hotel and serviced apartment accommodation and accommodates Phase 2 of the MTRC Hong Kong Airport Railway Station in the 5 level basement below. It also houses major transportation facilities for the local bus companies together with several floors of car parking. The total constructed floor area of the Hong Kong Station Development amounts to 650,000 square metres.

The Main Contractor for the tower, and the remainder of the development, was a joint venture (ESJV) between E. Man and Sanfield Contractors. The steelwork for the tower was awarded in two stages - both as nominated subcontracts. The steel for the megacolumns below 6/F level (5,000 tonnes) was fabricated and installed by NKK Corporation of Japan. The steelwork for the tower above 6/F (19,000 tonnes) was awarded to a joint venture (NSJV) between NKK Corporation and Sumitomo Corporation.

2. Tower Configuration

The basic plan configuration of a typical floor is shown in figure 3. The core at ground level is 29m x 27m with perimeter walls 1.5m and 1.25m thick. The size of the core was essentially driven by the need to maximise the efficiency of the vertical transportation system to serve the tower. The core is of conventional reinforced concrete - studies had

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shown that there was a significant cost advantage compared to steel and steel/composite alternatives. Initial cost studies were also conducted on core designs comprising grade 60 (28-day cube strength in N/mm²), grade 80 and grade 100 concrete. These revealed that no great advantage was gained by changing from the grade 60 option, as savings in lettable floor space did not significantly compensate for the additional cost and construction implications of using a higher grade.

The fundamental requirement for flexible office layouts, and the desire to maximise the panoramic views, necessitated that the perimeter structure should be kept to a minimum. This led to an outrigger lateral stability solution employing eight main megacolumns (two per face) with small secondary columns in the four corners. The outriggers mobilised the columns directly without the need for transfer through a belt truss system. A less accentuated belt truss system was incorporated at outrigger floors, however, to accept the heavy plant room floor loadings, and transfer the secondary corner column loads (as described below) into the megacolumns.

Studies showed that three outrigger levels were required, these were located at the levels R2/F to 33/F, R3/F to 55/F and R4/F to 67/F. As the triple floor height steel outrigger trusses coincided with refuge and double mechanical floor levels, their locations were effectively a compromise between the optimum structural arrangement and constraints imposed by vertical planning and lift zoning. In any event, the system was effective in that the amount of structure occupying useable floor area was minimised.

The small corner columns support gravity load only. There are three zones of such columns, each extending a maximum of 20 storeys, which are supported off transfer trusses at each of the outrigger levels. The loads for the secondary columns are effectively 'collected' by these trusses and transferred to the megacolumns.

At the higher levels, where the corners of the floor plate are stepped back, these columns are removed with the floor plate cantilevering from the megacolumns. Consideration was also given to the removal of these secondary columns throughout. However, it was concluded that the cost of the additional weight of floor steel involved, and the likely problems associated with tolerances of large cantilever floor systems, warranted that these small columns 300x 300mm should remain.

3. Geology and Foundation Design

The geology in the vicinity of the tower is fairly typical of Hong Kong. Grade III granite, having a permissible bearing capacity of 5MPa, is found approximately 35 metres below ground level although it shelves off steeply to the west of the tower footprint. Above this are layers of decomposed granite and alluvium, and over the top 20 metres there is a layer of

fill, the land only having been reclaimed three years earlier. Dynamic compaction had been carried out on certain areas of the fill to enable diaphragm walling to take place. With a total tower load of 5,200MN, foundations obviously had to be taken to rock head.



Fig.1. Two International Finance Centre

Surrounding the tower is a five level basement with a low-level commercial podium built above it. The southern most wall of the basement box also forms part of the adjacent Hong Kong Station Phase 1 structure. With trains under operation, it was essential that movements in this vicinity should be kept to a minimum. To achieve this top down methods of construction were adopted for the general

basement construction outside the footprint of the tower which enabled potential movements to be minimised.

In order to optimise the construction programme, it was considered appropriate for the tower to be built using a more conventional bottom up technique. The initial foundation solution envisaged 2.5 metre diameter bored piles, belled out to 3 metres. A large diameter cofferdam, encompassing the plan form of the tower, would then be constructed using diaphragm walling techniques to enable excavation down to the pile cap level. This would enable the cap to be cast in open excavation and the whole tower constructed from the pile cap level within the cofferdam. The circular (compression ring) nature of the cofferdam eliminated the need for internal props to provide lateral support the excavation.



Fig.2. Excavation within the temporary cofferdam

Consequently excavation within the cofferdam could be carried out unhindered (figure 2). The cofferdam provided very stiff lateral support and consequently further ensured that the ground movements were kept to a minimum, particularly with respect to the operational MTRC tunnels running alongside the development.

The foundation sub-contractor Bachy Soletanche Group proposed an alternative foundation solution which was adopted in the final works. With a reasonably constant rock head level (as determined from site investigation) it was proposed to construct the 61.5 metre diameter cofferdam down to bedrock using a 1.5 metre thick diaphragm wall keyed into the rock. Using three reinforced concrete ring beams, and lowering the external water table by eight metres, excavation could then take place to rock head level and the pile cap/raft be cast bearing directly on rock, thus omitting the need for bored piling. Conditions on site proved to be slightly different from those anticipated, with a localised depression to the South East of the footprint of the rock head level down to a depth of approximately 55 metres, a depth too great to allow open excavation in this area. In order to

overcome this a mixed foundation solution was adopted. Over much of the area the original raft solution was adopted. Mass concrete fill was then used locally, between rock head level and the underside of the cap, in the areas where the rock head sloped away. Locally, at the location of deepest rock, barrettes were installed from ground level to transfer the pile cap/raft loads to the bearing stratum, use also being made of the cofferdam panels to transfer vertical load.

Construction of the tower commenced in earnest in January 2000 with the first pour of the 6.5m thick reinforced concrete raft. With a total concrete volume of almost 20,000 cubic metres for the entire raft, it was considered not feasible to cast the raft in one continuous pour. The first pour was 5,000 cubic metres in volume and covered the entire area of the cofferdam. A further eight pours each covered half the area of the cofferdam, with the vertical construction joints rotated on plan through 90 degrees for each successive layer. A final capping layer comprising 3000 cubic metres of concrete, again covered the entire area of the cofferdam in a single pour.

4. Lateral Response

Wind tunnel studies were performed on the tower by Rowan Williams Davies & Irwin Inc. (RWDI) in accordance with the Hong Kong Buildings Department Practice Note PNAP 150. This included topographical studies, a force balance assessment of the loads and monitoring of cladding pressures. A second confirmatory wind tunnel study was undertaken by Cermak Peterka Peterson (CPP). The two studies agreed to within 6%.

Studies highlighted the dominance of crosswind response for this particular building. The resulting global characteristic base bending moments and base shears in the orthogonal directions were 19,000MNm and 128MN. The combination factors used in the derivation of diagonal design forces were $\pm 0.79F_x \pm 0.79F_y$.

The predicted period of the building is 9.1 seconds, which accords quite well with the simple $H/46$ approximation. Lateral accelerations were predicted and compared against the NBCC, ISO and Davenport Criteria for occupancy comfort. In doing this a variable structural damping was built into the analysis equivalent to 0.8% percent at 1 year return events, varying linearly to 2.0% for 50 year return events. Under these conditions the most critical accelerations, in terms of impact on human comfort, occurred in the 5-10 year return typhoon event range. The accelerations were however deemed acceptable for office occupancy in Hong Kong without the need for supplementary damping.

The lateral deflections under wind loading, including the second order P-delta effects of gravity load, were $H/450$ in the orthogonal direction and

$H/380$ in the diagonal, where H is the height of the building above pile cap level.

5. Floor System

Above the 6/F the tower comprises typical office floors and trading floors, with a design imposed loading of $(3+1)$ kPa and $(4+1)$ kPa respectively. Below 6/F level the floors are reinforced concrete - commensurate with the podium and basement construction. Initial designs were conducted to compare prestressed concrete and composite steel/concrete floor systems for the floors above 6/F. The concrete solution comprised a 275mm post-tensioned slab with 2000mm x 650mm deep perimeter reinforced concrete band beams. The composite schemes comprised 125mm thick slabs acting compositely with permanent decking supported at up to 3m intervals on a variety of steel beam options. Although studies showed that the cost of the concrete floor was slightly less, and that the anticipated cycle times for the two systems were similar, the additional costs for the columns and foundations due to the increased dead loading showed that, overall, the composite solution was preferable.

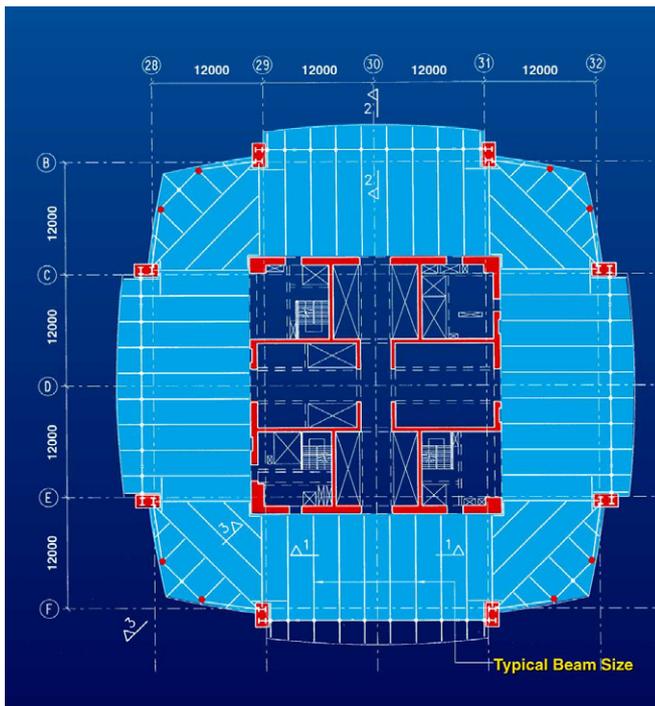


Fig.3. Typical floor framing

The typical floor-to-floor height is 4.17m with a dimension from underside of ceiling to the top of floor slab of 1.2m. A number of composite floor solutions were investigated which offered varying degrees of service/structure integration. Of these, asymmetric fabricated tapered beam and composite truss solutions presented the lightest (though not necessarily the cheapest) solutions. Such systems were, however,

not favoured due to the need for maximum flexibility in the layout of main and tenant services within the floor. The solution that was adopted comprised 460mm deep steel secondary beams spanning (11.4m to 13.5m) from the core to 900mm deep primary girders spanning 24m between the main columns. One of the key features of the layout of the floor was the inclusion of a significant diagonal beam which, in conjunction with the primary girder on the main faces, provided a continuous 'tension ring' around the floor plate. This was deemed necessary to enhance robustness and provide direct buckling restraint to the columns. This peripheral primary beam arrangement provided a zone around the core where services could be installed beneath the steel beams requiring only small penetrations through the main primary girder for minimal services which needed to access the building perimeter. The 24m primary girder comprised an asymmetric fabricated section and was structurally continuous with the megacolumns.

The limiting criterion for the design of the floor system was the vertical inter-storey deflection limits at the facade of 20mm. In achieving this limit, the effects of potential differential shrinkage and long term creep effects of the primary beams on adjacent floors, were considered in addition to patterned imposed load. These effects along with the axial shortening of the core and columns were incorporated in the construction presets to ensure that the building was constructed within acceptable tolerance.

The weight of floor beam steel is 36 kg/m^2 based on gross constructed floor area.

6. Megacolumns

The megacolumns comprise composite steel/concrete in which the steel elements are encased with reinforced concrete. Table 1 presents a summary of the columns sizes at various heights in the building together with details of steel content. Extensive studies were carried out to establish the most appropriate form of the columns in terms of cost and buildability and to optimise the size and steel/concrete ratios.

Concrete encasement was adopted to enable the maximum use of steel in the form of reinforcement rather than the less cost-effective use of structural sections. Reference 1 describes initial studies that were performed to investigate the optimum proportions of those elements that contribute to the lateral stiffness of the tower. This included an assessment of the optimum proportions of the steel and concrete in the megacolumns (as the proportions of the core walls and outrigger elements) to minimise the initial capital structural cost of the tower. In addition, reference 1 describes subsequent optimisation studies that were performed considering the value of the useable floor space occupied by the structure.

One key issue was the need to maximise the

buildability of the megacolumns. As a consequence, the structural steel component was split into a number of sub sections that could be lifted and connected with ease. In addition, it was considered at the outset the concrete encasement of the megacolumns would be formed using self climbing formwork (figure 4). This latter initiative effectively removed the reliance on the cranes in lifting column formwork between floor levels, thereby maximising their efficiency in lifting structural steel components and reinforcement. The formwork system effectively comprised hinged forms that wrapped around the column. Prior to jumping to the next floor, the form were unfolded to form a single plane on the façade side of the column. The formwork could then climb without interfering with the pre-installed main structural steel elements.



Fig.4. Column Climbform System

Following the passage of the form, a few light trimming steel members could be then added to complete the floor framing immediately adjacent to the column and façade.

Table 1: Mega column Schedule

Level	Overall Dimensions	No. of Sub-Stanchions	Averaged Weight of Steel (tonne/m)	Percentage of Reinforcement Bar	Design Effective Length (m)	Concrete Grade
B5 to 6/F	2.3m x 3.5m	6	9.7	4.0 %	30.3	60D
6/F to 32/F	2.3m x 3.5m	3	2.7	3.5 %	24.0	60D
33/F to 52/F	1.85m x 3m	2	1.1	3.0 %	19.2	60D
53/F to 69/F	1.4m x 2.6m	2	0.9	3.0 %	14.8	45D
70/F to 77/F	1.2m x 0.9m	1	0.6	2.0 %	12.8	45D
78/F to Roof	1m x 0.75m	1	0.5	2.0%	8.9	45D

The effect of the megacolumn buckling and restraints provided by the floor diaphragms was a key consideration given the massive nature of the columns and the relatively thin nature of the individual floor diaphragms. A second order non-linear analysis was undertaken to investigate the interaction of the spring stiffness of the floor diaphragms and the column buckling. This enabled the effective length of the column to be determined and quantified the forces to be resisted by the floor diaphragms (see table 1). The inclusion of a continuous substantial primary beam tie connecting all the megacolumns facilitated the mobilisation of the floor diaphragms in providing the necessary column restraint. It should be noted that this perimeter tie also enhanced the overall robustness of the tower.

The Contract for the structural steelwork was split into two parts in order to gain an overall programme

advantage.

The steelwork in the megacolumns up to 6/F level formed one contract, with all the steel above that level forming the larger follow-on steelwork contract. In the initial contract, the steel in the megacolumns was installed by mobile cranes from ground floor level into the temporary cofferdam. The optimisation studies had shown that there were significant commercial advantages in increasing the stiffness of the columns in this first contract over and above that required for strength. It effectively minimised the stiffness requirement, and hence the impact on the tower cranes (of lower capacity than the mobile cranes), in the follow-on contract – effectively onstruction-led optimisation of proportioning of structural material.

The total weight of steel in the megacolumns is equivalent to 49kg/m² over the gross constructed floor area of the tower.

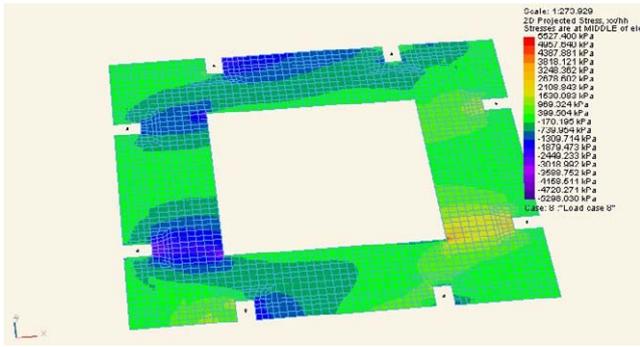


Fig.5. Finite Element analysis of floor diaphragms to assess stiffness and resulting forces in resisting column buckling.

7. Outriggers

At the outset it was apparent that, in order to satisfy the access requirements in and around the mechanical floors, a steel outrigger system would be

required. A punched concrete wall type outrigger was considered, however, this provided insufficient stiffness given the openings that had to be accommodated.

The steel truss outriggers pass through, and are cast within, the core walls. Optimisation studies had suggested that the axial stiffness ratios for the bottom, middle and upper outriggers should be 1 : 0.81 : 0.64 respectively (ref 1). Initially, it was considered that only the top and bottom booms of the outriggers would pass through the core - the longitudinal shears between the booms being resisted by the reinforced concrete of the core walls. However, due to the need for large openings through the core wall to accommodate the requirement for significant M&E access, it became apparent that a steel truss would be required to supplement the strength and stiffness of the perforated core.

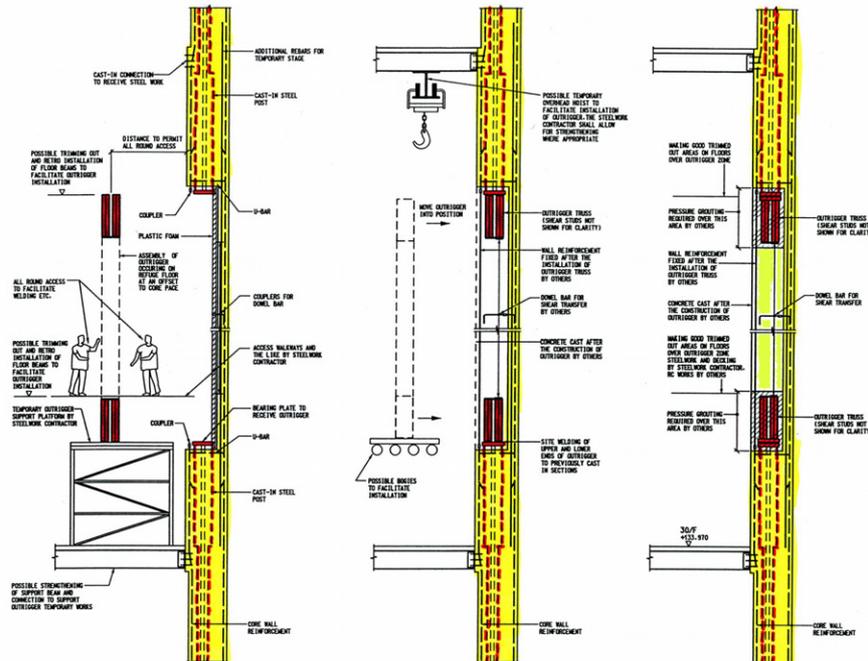


Fig.6. Diagrammatic representation of Outrigger Retro-installation

In the analysis of the outrigger system, a detailed assessment of the local deformations at the core wall/outrigger interface was made, with the stiffness of the overall lateral stability system modified accordingly. Extensive analysis was carried out to assess the precise characteristics of the interface between the steel truss and concrete wall components to ensure strain compatibility - thereby minimising the potential for cracking in the core. The analysis took into account the flexural stiffness of the shear studs and anchorage that were used to transmit the forces from the outriggers to the concrete core. From the analysis it was found that to achieve compatibility of strain, it was

necessary to restrict the stress in the shear studs to half their design capacity – essentially restricting the flexural performance of the studs to the elastic range, thereby maximising stiffness.

One of the key issues concerning steel outriggers in composite tall building construction, particularly where such large outriggers are required, is their potential impact on the construction programme. The core of such buildings can be constructed with relative speed and efficiency using a climbform system. Typical cycle times achieved on the core were 3-4 days. Clearly, stopping the climbform at the outrigger levels to permit the steelwork contractor to install the outrigger is not conducive to

optimising continuity of labour usage, or minimising the construction programme.



Fig. 7. Retro-installation of steel outrigger within the core wall

Figure 6 shows conceptually, the retro-outrigger installation approach adopted on the tower. Stage 1 involved reducing the outer core wall thickness from (typically) 1000mm to 300mm over the full height of the outrigger zone. Above the outrigger zone, the wall thickness reverts to (typically) 1000mm. Although this involved an adjustment to the climbform system at the locations of the change in wall thickness, it did permit the climbform to pass through the outrigger zone in advance of steelwork installation. In Stage 2, and with the climbform continuing to construct the upper levels, the outrigger elements were located in position and assembled. Figure 7 shows the partial installation of the outrigger within the zone corresponding to the thinned core wall. Stage 3, with the welding of the outrigger completed, the core wall was retro-concreted. This included a 100mm thick grouted layer at the top of the outrigger zone at the interface with the widened section of the wall. Figure 8 shows a view of the tower during construction with the climbform progressing beyond the outrigger zone and the partially completed steel outrigger.

8. Differential Axial Shortening

One key consideration is the differential shortening between the core and the perimeter columns during the construction of the tower and the long term effects post completion. This is particularly important in composite tall buildings in which the deformation characteristics of the steel intensive columns and the large core can be markedly different. Prior to construction a number of specific material tests were carried out to quantify the modulus and shrinkage characteristics of the proposed concrete mix. This was necessary

because of the limited data available on the precise characteristics of grade 60 concrete incorporating 25% PFA. From these studies, Arup undertook a comprehensive study to quantify the differential shortening and the necessary pre-sets to be built into the construction of vertical elements.



Fig. 8 Climbform construction of the core progressing prior to completion of the steel outriggers.

The outrigger connections to the megacolumns incorporated a series of packing shims at the contact surfaces. This enabled the outriggers to be effective during construction to enable the tower to resist any typhoon winds that may have occurred. In addition, it allowed the packing shims to be removed, or added, as required (with the assistance of small jacks) to enable differential movements between the outrigger and columns to occur during construction. Using this approach, it effectively prevented the build-up of very large internal forces that would otherwise have been generated in the event that the outrigger and columns were rigidly, and permanently, connected together from the outset.

9. Robustness and Safety Of The Tower

The 9/11 tragedy in New York occurred when the 2IFC tower was constructed up to the 33/F. In the days following 9/11 many questions were raised with regard to the robustness, integrity and egress provision within the 2IFC Tower.

Whilst it was recognised at an early stage that it would be impracticable to design a tower specifically to resist such extreme events, Arup undertook an extensive series of studies focused on the assessing the safety of the design. It should be noted that the studies were carried out in the 4 weeks after 9/11 – speed being essential so as to inform the ongoing construction of 2IFC.

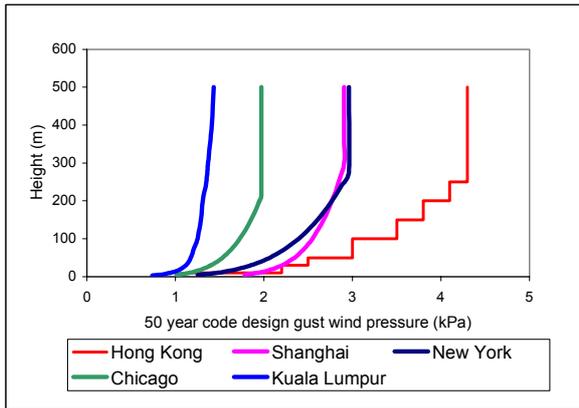


Fig. 9. Comparison of wind codes

The studies involved performing comparative assessments of international practice and code requirements with regard to structural integrity and escape, as well as detailed analytical dynamic simulations of a range of aircraft engine impact scenarios – the latter used software developed crash simulations for the automotive industry. The key findings from these studies were as follows:-

- The large composite steel/concrete megacolumn solution resulted in key elements which offered an extremely high resistance to extreme impact scenarios.
- The concrete core offers a hardened escape route to evacuees. The walls of the core are particularly good at maintaining integrity and redistributing load in the event of localised damage.
- Fundamental to enhancing structural robustness that critical elements are themselves robust and that, in the event of its critical element removal, alternative structural load paths exist to adequately resist the residual loads. The outrigger solution is capable of providing such alternative load paths. It was shown that the integrity of the building was maintained in the event that two of the megacolumns were removed at low level. The outrigger effectively acts as a ‘gravity prop’ to support the megacolumns above.
- Tall buildings in Hong Kong are designed to resist significant lateral loads (typhoon winds) compared to other tall building centers around the world (see above figure). As a result, under non-typhoon conditions, the elements exhibit significant residual strength to enhance the resistance to impact events and to tolerate load redistribution when in a damaged state.

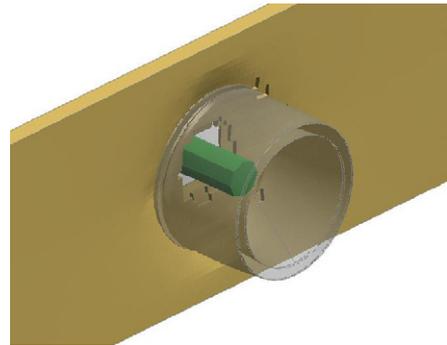


Fig.10. Analytical simulations of aircraft engine impact on core walls.

- It is evident that the provision of exist staircases in the Hong Kong code is larger than that which exist in accordance with other international codes. The main reason is that the Hong Kong code requirement is largely based on a simultaneous evacuation of the building whilst other codes permit a phased evacuation strategy (thereby reducing the overall egress requirements). Other areas in which the escape provisions in the Hong Kong code (and adopted in 2IFC) are seen to be more stringent compared to other codes are as follows:
 - Requirement to provided dedicated access for firefighters.
 - Refuge floors to be provided at not greater than every 25 storeys. This provides a ventilated place of refuge for those egressing the building and also command points for fire fighters.
 - Discontinuous stairwells to prevent the chimney effect of smoke egress throughout the height of the tower.

The conclusion from these studies (completed before much of the forensics on the WTC collapse had been completed), was that the structural form and planning of the 2IFC Tower provided a robust form with enhanced resistance to potentially damaging events. As a consequence of these studies, the construction of the 2IFC tower continued with no changes to the design.

Conclusions

This paper has presented a brief overview of some of the design and construction-led innovations that have been adopted in the construction of the IFC Tower, the tallest building in Hong Kong.

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