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Raffles City in Hangzhou China -The Engineering of a ‘Vertical City’ of Vibrant Waves-

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Abstract

This mixed-use Raffles City (RCH) development is located near the Qiantang River in Hangzhou, the capital of Zhejiang province, located southwest of Shanghai, China. The project incorporates retail, offices, housing, and hotel facilities and marks the site of a cultural landscape within the Quianjiang New Town Area. The project is composed of two 250-meter-tall twisting towers with a form of vibrant waves, along with a commercial podium and three stories of basement car parking. It reaches a height of 60 stories, presenting views both to and from the Qiantang River and West Lake areas, with a total floor area of almost 400,000 square meters. A composite moment frame plus concrete core structural system was adopted for the tower structures. Concrete filled steel tubular (CFT) columns together with steel reinforced concrete (SRC) beams form the outer moment frame of the towers’ structure. The internal slabs and floor beams are of reinforced concrete. This paper presents the engineering design and construction of this highly complex project. Through comprehensive discussion and careful elaboration, some conclusions are reached, which serve as a reference guide for the design and construction of similar free-form, hybrid, mix-use buildings.

Keywords: High-rise building, Design and construction, Composite structures, Seismic design, Digital engineering, Performance-base design

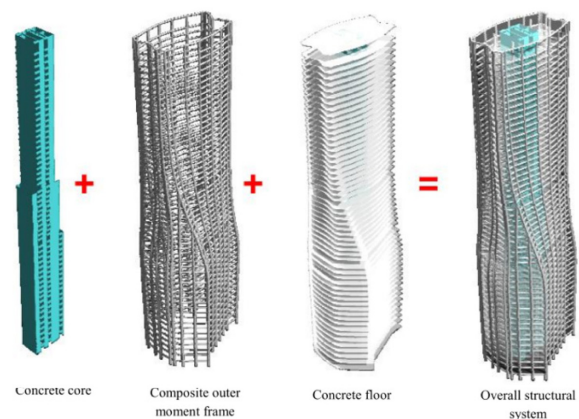
1. Introduction

This mixed-use Raffles City (RCH) development is located near the Qiantang River in Hangzhou, the capital of Zhejiang province, located 180 kilometres southwest of Shanghai, China. Raffles City Hangzhou will be CapitaLand’s sixth Raffles City, following those in Singapore, Shanghai, Beijing, Chengdu and Bahrain. The project incorporates mixed functions of retail, offices, housing and hotel facilities and marks the site of a cultural landscape within the Quianjiang New Town Area. It is located in the central business district of Hang Zhou, China. The project is composed of two 250 m tall super high-rise twisting towers of an outlook of vibrant waves and a commercial podium and 3 storey basement car parking. Fig. 1(a) is the artistic image of the Project. It reaches a height of 60 stories, presenting views both to and from the Qiantang River and West Lake areas, with a total floor area of almost 400,000 m². The philosophy of the planning of this iconic project is to create a ‘vertical city’ of mixed functions providing ‘24-7’ services to the consumer and with open and vibrant building outlook.

The owner of the Project is Raffles City China Fund, and CapitaLand was responsible for the overall project



(a) Artistic image



(b) Tower structural system

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Figure 1. Raffles City Hangzhou, China.

management. The design architect and engineer are UN-Studio and Ove Arup respectively. Shanghai Construction Ltd. and China Construction Third Engineering Bureau were appointed as both the main and MEP contractors respectively. The overall construction cost of the project is over CNY 6,250 Million e.g., approximately GBP 620 Million. The construction of the Project commenced in 2012 and it is to be fully completed and opens to public since middle 2017.

Composite moment frame plus concrete core structural system was adopted for the tower structures. Concrete filled steel tubular (CFT) columns together with **steel reinforced concrete (SRC) beams** form the outer moment frame of the tower structures. The internal slabs and floor beams are of reinforced concrete. The structural frame works of the tower is shown in Fig. 1(b). This paper presents the structural engineering design and construction of this highly complex iconic Project. Through the comprehensive discussion and careful elaboration, some conclusions are reached, which will serve as the guidance and reference of the modern composite design and construction of similar tailor shaped hybrid buildings.

2. Structural Design and Analysis

Both China local and international structural design codes (MHURD, 2010; MHURD, 2011; BSI, 2005; AISC, 2005; SCI & BCSA, 2002; BSI, 2004) were considered during the design of the steel-concrete composite structures of the Project. The detailing of the composite joints is always a frontier to conquer during the design of modern high-rise composite buildings. The rigidity and ductility requirements of composite joints are also covered

in various design codes (BSI, 2005; AISC, 2005; SCI & BCSA, 2002; MHURD, 2011).

2.1. Tower structure

During the structural design of the tower structure, various possible structural forms were explored to achieve the optimum results in building functions, structural performance, cost effectiveness and overall buildability. A total of three outer frame forms were studied as follows for the 250 m tall tower structures:

Option 1: Steel floor beams together with concrete filled steel tubular (CFT) columns;

Option 2: Concrete floor beams together with steel reinforced concrete (SRC) columns, and

Option 3: Steel reinforced concrete (SRC) beams together with CFT columns.

Cost comparison and work breakdown analyses were conducted for a typical tower floor. The results are shown in Tables 1 and 2 respectively. It was concluded that that Option 3 of SRC **floor** beams together with CFT columns share a similarly low construction cost as the reinforced concrete dominant Option 2. While overall construction cycle of Option 3 is much shorter by breaking through the critical path of column construction with permanent formworks of steel tubular columns. The construction cycle per typical floor is approximated to be 5 days as shown in Table 2.

Thus, Option 3 was selected to be the outer moment frame of the tower structures with a relatively low cost, controllable constructability and reasonable building functions. Fig. 1(b) shows the structural frameworks of the

Table 1. Cost comparison on structural schemes

Item	Option 1: Steel floor beams + CFT columns	Option 2: RC floor beams + SRC columns	Option 3: SRC floor beams + CFT columns
Concrete (m ³ /m ²)	0.54	0.97	0.97
Rebar tonnage (kg/m ²)	110	117	102
Steel tonnage (kg/m ²)	118	62	68
Formwork (m ² /m ²)	0.97	2.1	2.1
Profiled steel decking (m ² /m ²)	0.82	-	-
Overall cost (%)	149	100	105

Table 2. Work breakdown analysis of a typical floor

Option 1: Steel floor beams + CFT columns		Option 2: RC floor beams + SRC columns		Option 3: SRC floor beams + CFT columns	
Work breakdown	Days	Work breakdown	Days	Work breakdown	Days
Erection of steel tubular columns	0.5	Circular column formwork	1	Erection of steel tubular columns	0.5
Erection of edge beams	0.5	Erection of cloumn rebars	1	Erection of edge beams	0.5
Erection of floor steel beams	1.5	Erection of edge beams	0.5	Erection of floor steel beams	1.5
Rebar erection in slab and walls	1.5	Erection of floor steel beams	1.5	Rebar erection in slab and walls	1.5
Concrete pouring	1.0	Rebar erection in slab and walls	1.5	Concrete pouring	1.0
		Concrete pouring	1.0		
Total	5.0	Total	6.5	Total	5.0

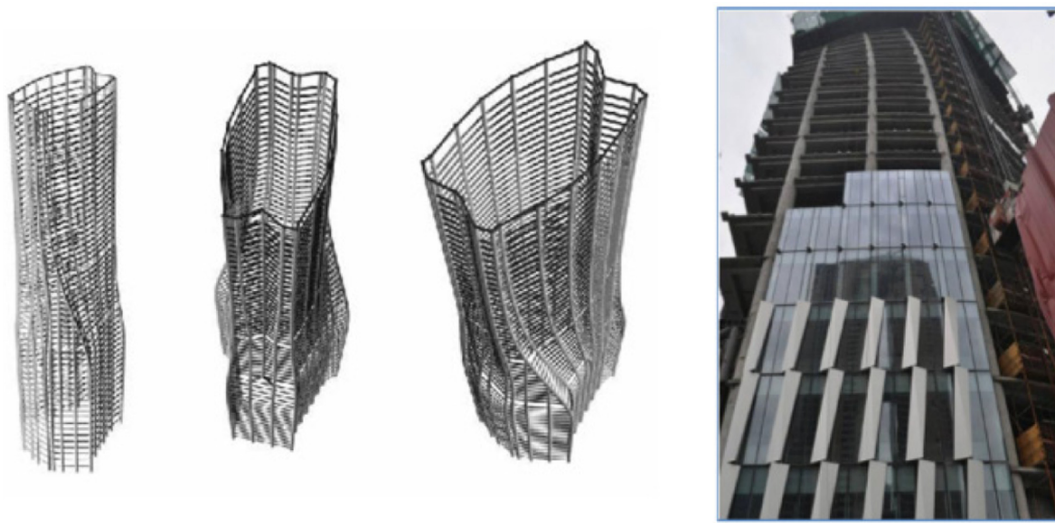


Figure 2. Integrated building skin.

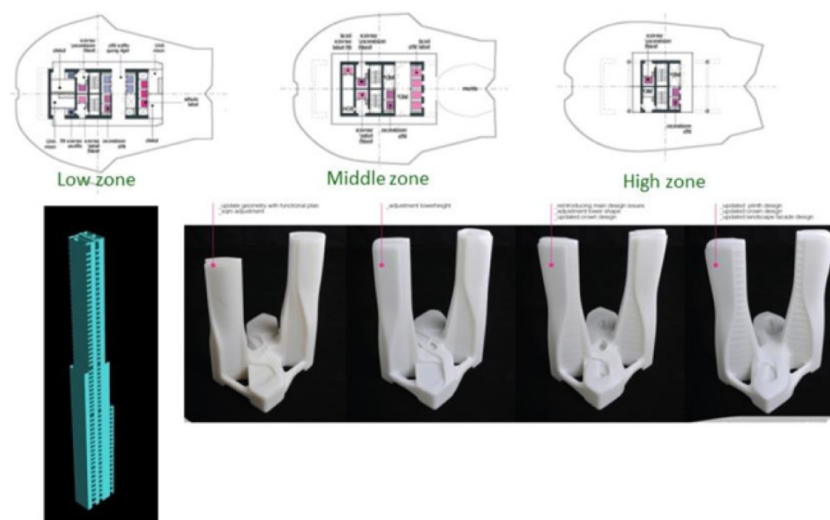


Figure 3. Evolution on building envelope.

tower structures. Main structure has been topped up near the middle 2015. It was demonstrated that 5-day-cycle was achievable with the adopted structural form.

The tower building envelope was formed through the outer moment frame of the structure, and each segment of the column is inclined at a different angle and with a different orientation as shown in Fig. 2. This ensures the maximum possible efficiency of floor area, and in the meantime, achieves the tailored twisting shape of the tower. Fig. 3 shows the design evolution on the shape of building envelopes to ensure manageable spans floor beams and continuous structural cores in the meantime. As such, a uniform beam depth and ceiling height can be achieved in the same floor. Advanced three-dimensional design tool and technology were adopted to ensure the

right setting out of each column on each floor.

The structural design of the composite joint between CFT columns and SRC beams needs to safeguard the overall structural stability through the fully rigid joints and to avoid scarifying any tailored space in the meantime. The conventional ring beam type composite joint is regarded to be bulky and not suitable because of its [interface](#) with the façade erection and interior decoration. An innovative and high performance corbel type composite joint is proposed with a minimum intrusion into the interior space to achieve the fully rigid joint. Fig. 4 presents geometrical configurations for both types of joints.

The proposed corbel type composite joint includes the following key components as shown in Fig. 4(b):

- The corbel and ring stiffener as butt welded to the

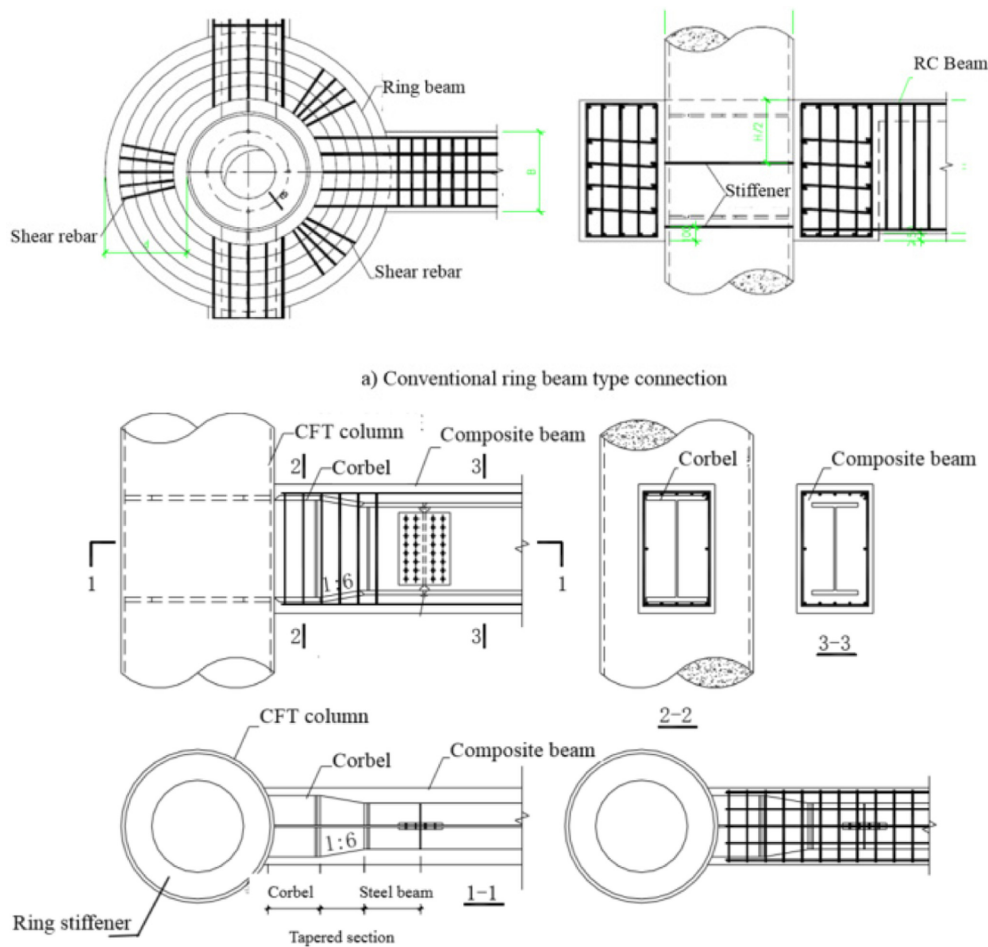


Figure 4. Composite connections.

CFT column:

In order to ensure a full strength rigid joint, the I-section corbel is enlarged and stiffened together with a ring stiffener as welded inside the steel tube, so that the overall rigidity and load carrying capacity of the joint is not less than that of a typical SRC beam section.

- The tapered section from the corbel to the steel beam:

In order to ensure a smooth loading and stress transfer from the corbel in the joint region to the ordinary SRC beam, a tapered steel section is proposed with a slope of 1:6.

- The steel section in the SRC beam:

The ordinary I-steel section in the composite SRC beam is fully connected to the outer edge of the corbel through full bolted joints on both flanges and webs.

- Lapped reinforcement bars:

All the longitudinal reinforcements are lapped around the flanges of the steel corbel, so that both the loads and stress can be transferred from the longitudinal main reinforcements onto the corbel in the joint region.

- Concrete encasement

All above mentioned components are encased with C35

concrete to ensure a composite action.

In order to achieve a full strength joint between the SRC beam and CFT column, the corbel together with the ring stiffener is strengthened to the strength and rigidity of an ordinary SRC beam. Thus, satisfactory deformation and plastic energy absorbing capacities can be achieved with a preferred failure mode and location of the plastic hinge.

2.2. Podium Structure

Reinforced concrete moment frame and shear wall system was adopted for podium structures. In the location of central atrium, a total of 4 number of tailored steel column is designed together with steel-concrete composite floor system, which created a column free retail space of approximately 2000 m² per storey. Rigorous finite element analyses with beam-column and shell elements were conducted for the structural adequacy of this atrium area. Detailed construction sequence simulation was also conducted which is to be introduced in Section 3 of this Paper.

A total of 4 number of linking bridges of approximately 45 m span each were designed and link the towers and the

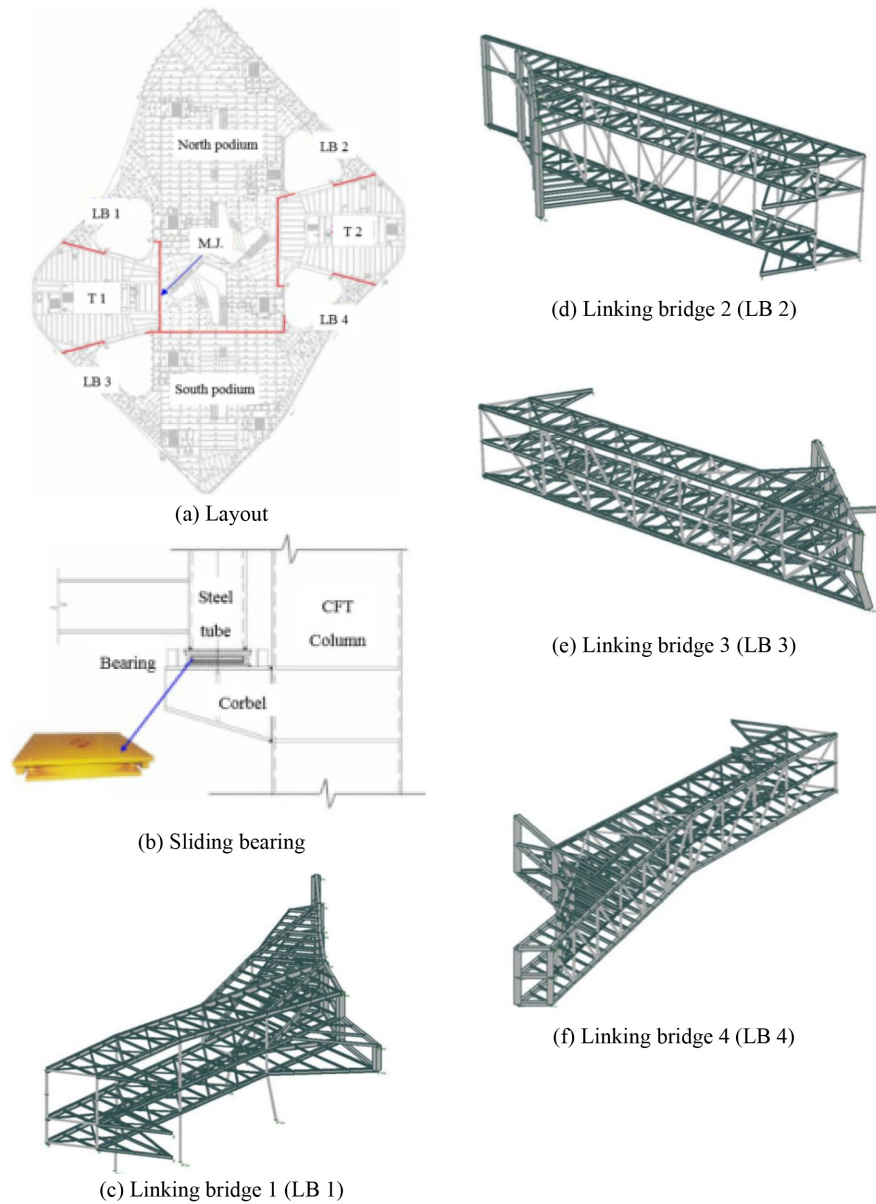


Figure 5. Design of linking bridges.

podium as shown in Fig. 5. Fig. 5(a) shows the general layout of the linking bridges, while Figs. 5(c) to 5(f) show their structural configurations. Steel trusses were adopted for Linking Bridges LB2, LB3 and LB4, while for Linking Bridge LB1, a steel moment frame was designed with columns allowed down to the building base. All of the linking bridges were design to be rigidly connected to the podium structures, while they are connected to the tower through roller connections of sliding bearings to allow for the relatively big movement of the tower under the excitation of both wind and seismic actions. Fig. 5(b) shows the details of the sliding bearing. The maximum relative movement between linking bridges and tower bearings were carefully calculated under Level 3 earth-

quake (MHURD, 2010), which defined the minimum sliding capacities of the sliding bearings.

2.3. Structural Analysis

[Performance-base seismic design approach](#) was adopted to analyze the structure (Wang, 2016), including both static and dynamic elasto-plastic analyses under various levels of wind, earthquake and gravity actions. Structural analysis software of both ETABS (2010) and ABAQUS (2004) were adopted to conduct the global structural analysis and counter-check the results with each other. In the global structural model, beam-column elements were adopted to simulate the moment frame and bracings of the structures, while 4-noded shell elements were adopted to simulate

the structural behaviour of shear walls and slabs. The equivalent strength and stiffness were adopted to consider the contribution of the steel and concrete sections to the overall stiffness and strength of composite columns and beams (Wang & Chung, 2006; Wang, 2012; Wang, 2014). In the elasto-plastic nonlinear simulation, the solution procedure requires the full load to be applied in a series of small increments so that the solutions may follow the load-deflection closely. A value of 5% is recommended as the maximum plastic strain increment in each incremental load. In order to accurately model the large deformation at critical locations after steel yielding as well as local concrete crushing and splitting, both material and geometrical non-linearities were incorporated into the finite element model. As this is a highly nonlinear problem, the solution is obtained through a number of equilibrium interactions for each load step. This is accomplished by an arch-length procedure in which the nodal displacements, the out-of-balance forces and the tangent stiffness matrix of the structure are updated after each equilibrium interaction. A force-based convergence criterion is adopted which requires the imbalance force is less than 0.5% of the average applied force in each equilibrium interaction.

3. Physical Tests and Verification

3.1. High Performance Composite Joint

The detailing of joints is always a frontier to conquer in the design of modern high-rise composite buildings. The rigidity and ductility requirements of composite joints are covered in various prevailing design codes. (BSI, 2005; AISC, 2005; SCI & BCSA, 2002; MHURD, 2011).

In RCH, the structural design of the composite joint between CFT columns and SRC beams need to safeguard the overall structural stability through the fully rigid

joints and avoid scarifying any tailored space in the meantime. The conventional ring beam type composite joint was regarded to be bulky and not suitable because of its **interference** with the façade erection and interior decoration. An innovative and high performance corbel type composite joint was proposed with a minimum intrusion into the interior space to achieve the fully rigid joint (Lou & Wang 2015; Wang 2015).

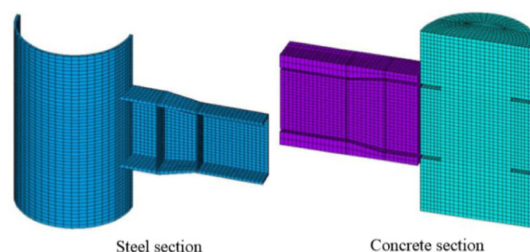
Physical tests under both static and cyclic loads were conducted to investigate the load carrying capacities and deformation characteristics of this new type of composite joint according to both ASTM (2011) and CABR (1997). Fig. 6(a) shows the overall set up of the tests. A total of four specimens were tested. Specimens SP1 and SP2 were tested under monotonic loads, while Specimens SP3 and SP4 were tested under cyclic loads. Table 3 shows the material properties of both steel and concrete of the test specimens.

A variety of structural responses are examined in detail, including load-deformation characteristics, the development of sectional direct and shear strains, and the history of cumulative plastic deformation and energy. A three-dimensional finite element model built up with solid elements was also proposed and carefully calibrated incorporating the material, boundary and geometrical non-linearities as shown in Fig. 6(b). Both the experimental and numerical studies demonstrated the high rigidity, strength and rotation capacities of the corbel type composite joints, and give detailed structural understanding for engineering design and practice. Figs. 8(a) and 8(b) present the results of the monotonic tests on Specimens SP1 and SP2, while Fig. 9 presents a typical failure mode. Table 4 is the summary of the results of the test.

A close observation on the strain development also shows that the direct tensile strain at the top flange is 30 to 50% higher than the compressive strain of the bottom



(a) Test set-up



(b) Finite element model

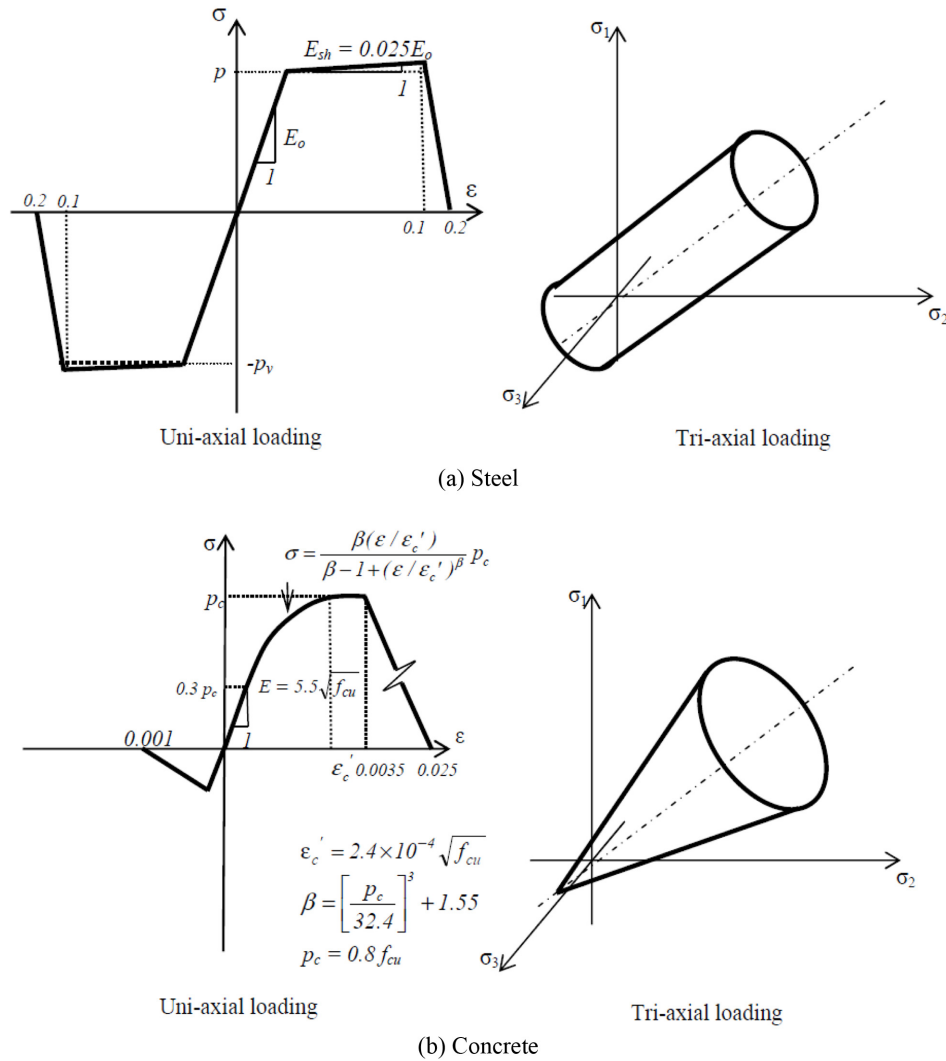
Figure 6. Connection test and numerical simulation.

Table 3. Material properties of steel

		Thickness, t (mm)	Yield strength, f_y (N/mm ²)	Ultimate strength, f_u (N/mm ²)	Elongation limit, δ (%)	f_u/f_y
Beam and corbel	Web	6	354.7	505.1	33.3	1.42
	Top flange	14	386.0	533.3	34.7	1.38
	Bottom flange	16	365.8	540.9	34.7	1.48
Column	Steel tube	16	390.4	504.1	31.3	1.29

Table 4. Summary of static tests and finite element simulation

		Applied load, P (kN)	Deflection, Δ (mm)	Applied moment, M (kNm)	Rotation θ (rad)	Strain in top flange, ε_t ($\times 10^{-6}$)	Strain in Bottom flange, ε_b ($\times 10^{-6}$)	Shear strain in web, ε_x ($\times 10^{-6}$)
SP 1	First yield	235.0	30.6	470.0	0.0042	1780	-1188	267
	Failure	321.0	98.3	642.0	0.0176	29635	-10684	2264
SP 2	First yield	211.1	25.6	422.2	0.0035	1782	-1047	316
	Failure	332.1	148.1	664.2	0.0178	33617	-6547	9352
FEM	First yield	220.3	29.3	450.2	0.0038	1780	-1134	304
	Failure	335.5	110.3	673.2	0.0175	30043	-8653	6345

**Figure 7.** Material model.

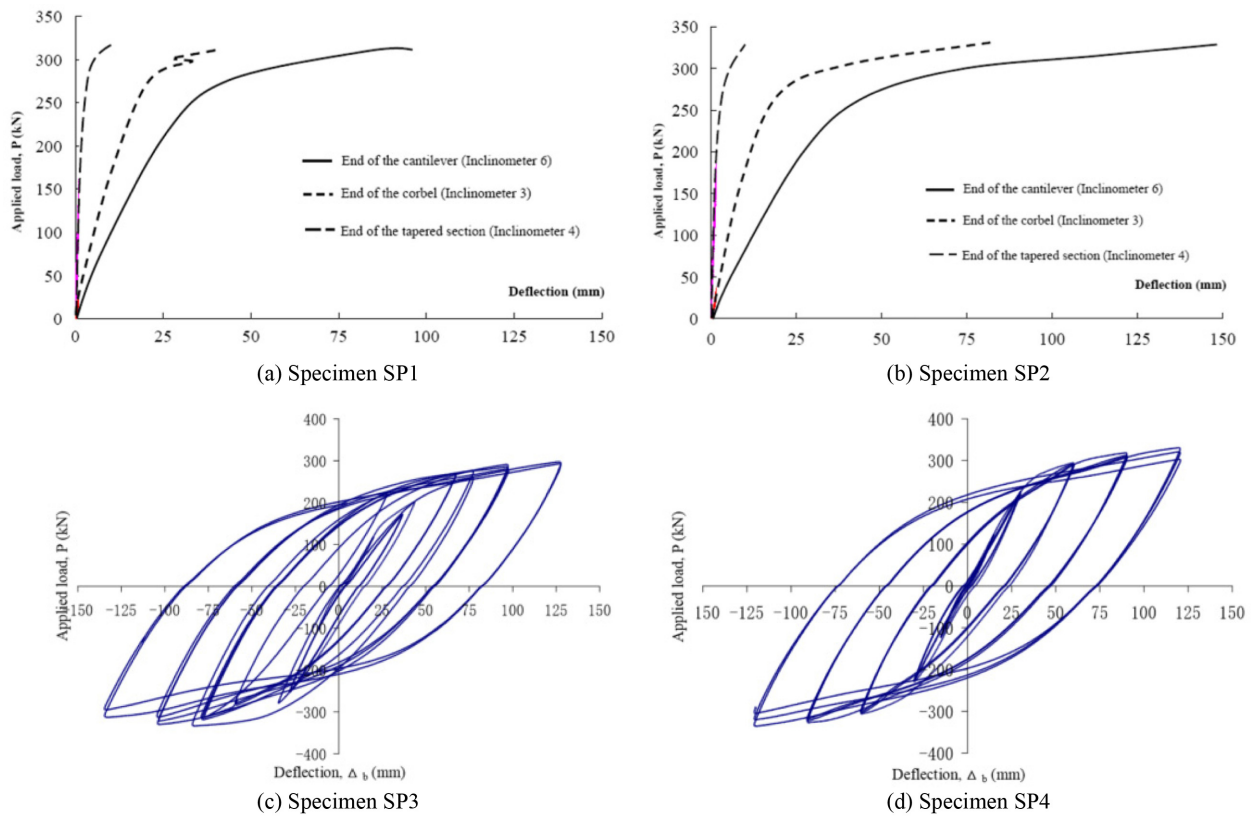


Figure 8. Load-deflection curves.

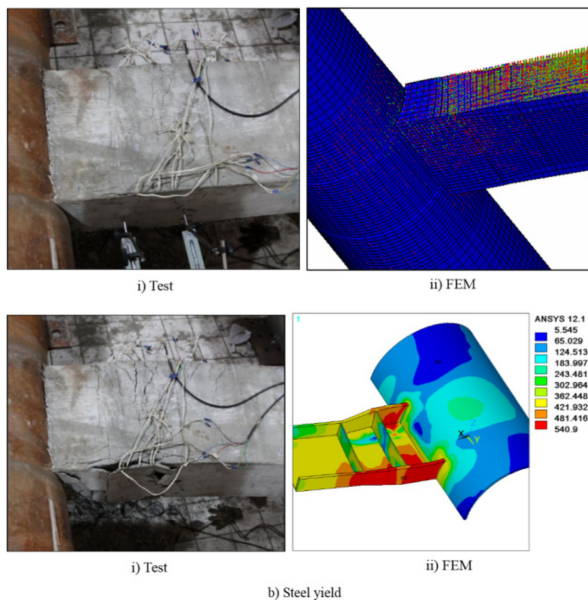


Figure 9. Typical failure mode.

flanges due to the contribution of the concrete material. It is noted that the shear strain in the web is significantly smaller than the strain in the flange, which is just above the yield strain. This is preferred for a high-rise building

in a seismic sensitive region like Hangzhou, where the Project is located. The quasi-static cyclic loading tests were conducted on both Specimens SP3 and SP4. Figs. 8(c) and 8(d) present the load-deflection and moment-rotation curves of Specimens SP3 and SP4. The cumulative plastic deformations of both Specimens SP3 and SP4 are 0.3 and 0.24 rad respectively, which are corresponding to 88 and 80 times the first yield rotation of the composite joints. This, again, demonstrates the high ductility and energy absorbing capacities of the corbel type composite joints.

To further study the structural behaviour of the corbel type composite joint, a generalized nonlinear three-dimensional finite element model was set up using the commercial finite element package ANSYS 12.1 (2011). The meshes of the finite element model are shown in Fig. 6(b). In order to simplify the problem and save computational time, only half of the specimen was modelled. The finite element simulation gives a quite close prediction of the load-deformation characteristics in the joint regions, which is demonstrated through the comparison of the load-deformation curves at the end of the joint corbel (Lou & Wang, 2015; Wang 2015). The nonlinear of material models for both steel and concrete materials are shown in Fig. 7. Fig. 9 shows the typical failure models of the corbel type composite joints from both physical tests and numerical simulation. As such, the corbel type

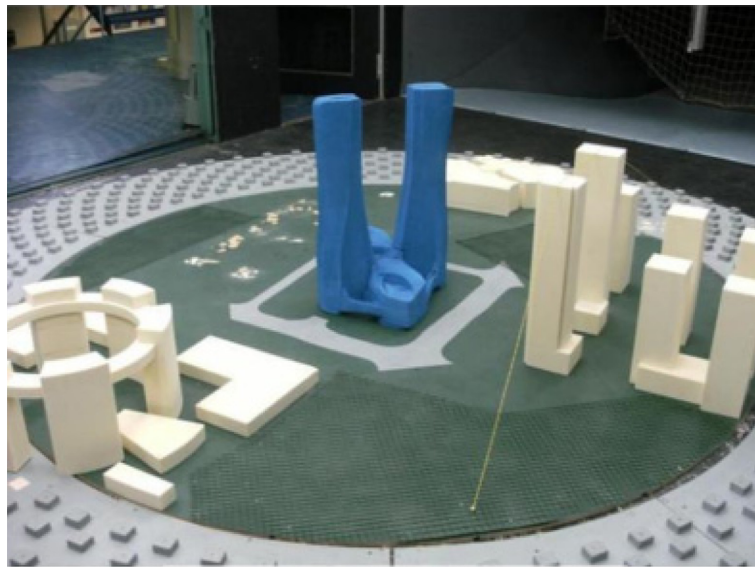
composite joint was verified to be of high strength, rigidity and ductility and suitable for highrise buildings in seismic sensitive regions.

3.2. Wind tunnel studies

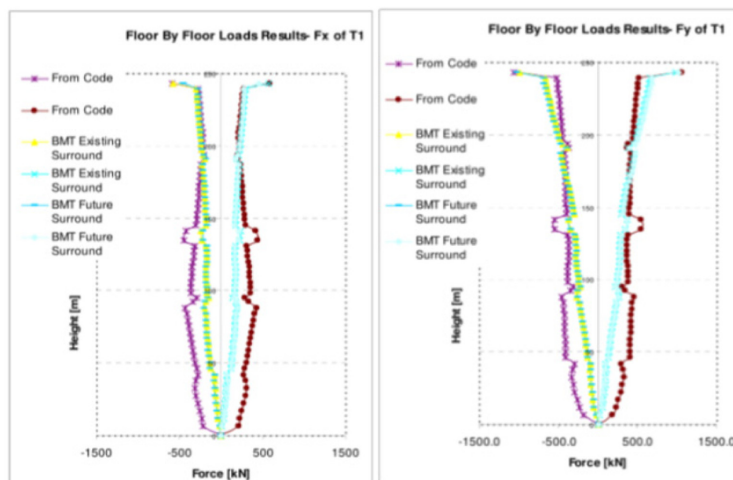
Wind tunnel test is regarded to be necessary to quantify the structural and cladding wind pressure of buildings with complex envelopes, whose distribution of wind loads cannot be quantified accurately through the prevailing loading codes (BSI, 2005; MHURD, 2008). Fig. 10(a) shows the physical model with a scale of 1:250 in the Wind Tunnel Laboratory of BMT Fluid Mechanics of UK. The results of the tests give detailed distribution of wind pressures at various location of the Project. A total of 1200 sensors are placed over the surface of the buildings. In some geometrically complex areas, the sensors are

more closely spaced to capture a more accurate distribution of wind pressure data. Fig. 9(b) is typical contours of wind pressures on the surface of the building envelopes with a return period of 100 years, which were adopted as part of the design wind load input in the structural and façade engineering design.

Meanwhile, in the view of the distribution of multiple closely spaced buildings around RCH, possible turbulence and relatively high speed wind may also be formed at the pedestrian level inside the court yard of the Project. It is regarded to be important to conduct pedestrian wind conformability studies to ensure the acceptable wind speed at the pedestrian level inside the Project. The computational fluid dynamic (CFD) software FLUENT (2002) was adopted for numerical simulation and analysis of wind environment at the pedestrian level.



(a) Test setup



(b) Results

Figure 10. Wind tunnel tests.

4. Digital Engineering

The advances of the modern computerized design tools allow the designer to break through the barrier of conventional modular design, and work on three-dimensional platform. Nowadays, more free-form and tailor-shape building skins are able to be achieved in a more creative way. Structural engineer shall equip themselves with three-dimensional design technology, and produce structural frameworks well suits the building skins. As shown in Fig. 2 for RCH, structural members are tailored to support the facade outer skin, and form the modern and vibrant outlook of the tower building. Both the structural engineer and main contractor work collaboratively on a three-dimensional platform to establish the setting out information on site. Due to the high complexity of the Project, multiple provisions based on digital technology

were conducted before and during the construction phases including both building information modelling (BIM) and virtual construction planning and simulation.

4.1. Building information modelling (BIM)

Building Information Modelling (BIM) was adopted in RCH to sort out the complex building shapes and possible conflicts at especially complex locations like basement, mechanical floors and cores. A BIM protocol was set up among various design parties and contractor to streamline the management procedures, and a BIM manager was employed to administrate the daily model operation and design coordination. The civil and structural BIM model was issued to the tenderers as part of the tender documents to assess the possible major dynamic conflicts due to the complex site logistics, like installation of steel works and facade panels, etc. Fig. 11(a) is the building informa-

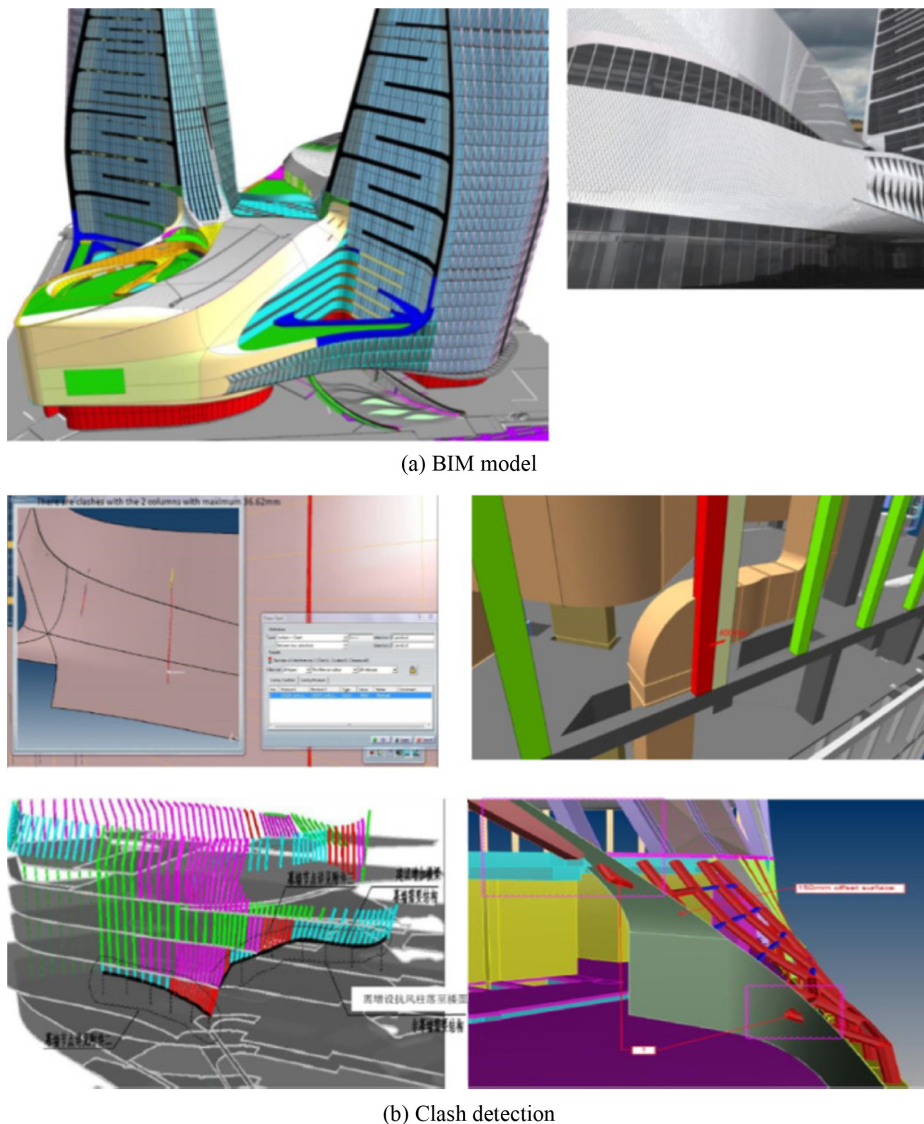


Figure 11. Building information modelling.

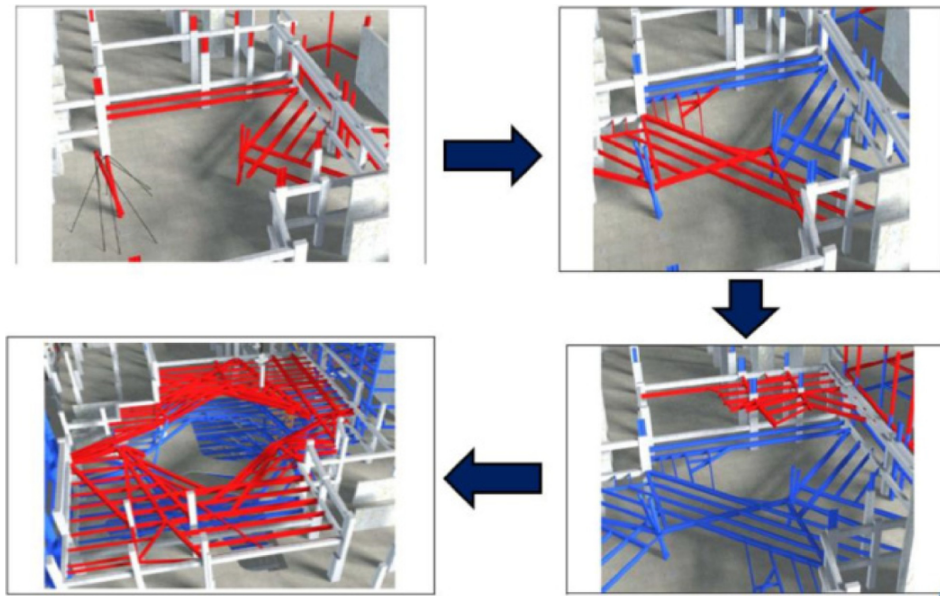


Figure 12. Virtual construction of atrium steel works.

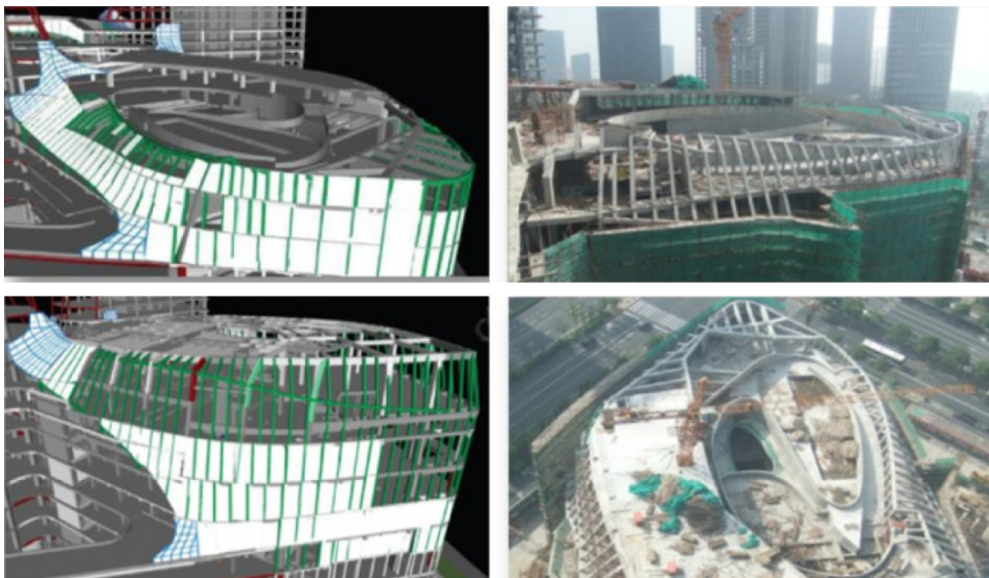


Figure 13. Virtual and real construction of podium roof.

tion model for the podium structure. Various clash detection and coordination were conducted as shown in Fig. 11(b) based on the model as a common platform drawing together all design consultants and contractors.

This project also faced the challenge of composite joints among various column beams and beams, leading to especial complexity and possible conflicts between the reinforcement and the steel section for each of such typical joints. Both construction mock-ups and a three-dimensional computer tool are adopted to assess the erectability allowing for reasonable construction tolerance and work

space in the meantime. Stiffener and gusset plates were adopted to transfer the loading from the reinforcement into the joint region without scarifying the structural continuity and rigidity.

4.2. Virtual construction

Virtual construction technology was conducted in some especially complex portion of the building structures through an extension of both building information modelling and finite element simulation. For the construction of large span steel and composite floors system around the podium

central atrium as shown in Fig. 12, each step of the erection of the steel beams was carefully planned and simulated to ensure the structural stability at each erection step without any necessary propping. In the meantime, structural analyses were also conducted following the erection sequence to ensure the structural and load bearing adequacy at each step of such construction.

Similar technology was also conducted on the highly complex and tailor shaped podium roof as shown in Fig. 13. The construction of each wind post is carefully planned and simulated through the building information modelling before it was put into the construction on site.

5. Construction Technology

5.1. Construction Mock Up

Another feather of RCH is its free form and tailored podium roof as shown in Figs. 1(a) and 11(a). The struc-

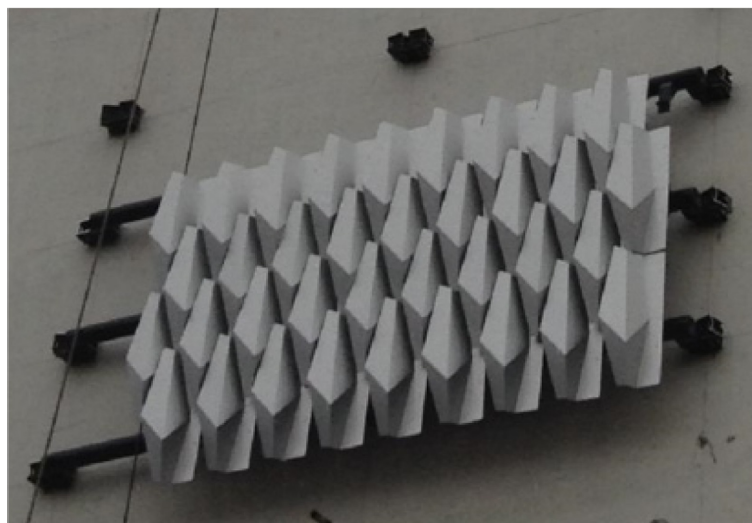
tural frameworks of the podium are shown in Fig. 13. Various wind posts were precisely positioned and set out on site to fit the façade control surface of this part of the buildings. Open type façade panel is adopted, and both thermal isolation and water proof was designed to be on the concrete backing panels cast between wind posts. Shot-concrete was adopted and its construction mock up was conducted as shown in Fig. 14(a). Core penetration and compressive loading tests were conducted on the specimen to ensure a proper concrete mixing design and constructed concrete strength. In the meantime, the mock up for the open-type façade panels were conducted, as shown in Fig. 14(b), to assess both the erectability and aesthetics of the design.

5.2. Tower construction

The construction of twisting towers was challenging, and multiple advanced construction technologies were



(a) Shot-concrete on backing structures



(b) Open type facade panels

Figure 14. Mock up of podium roof.

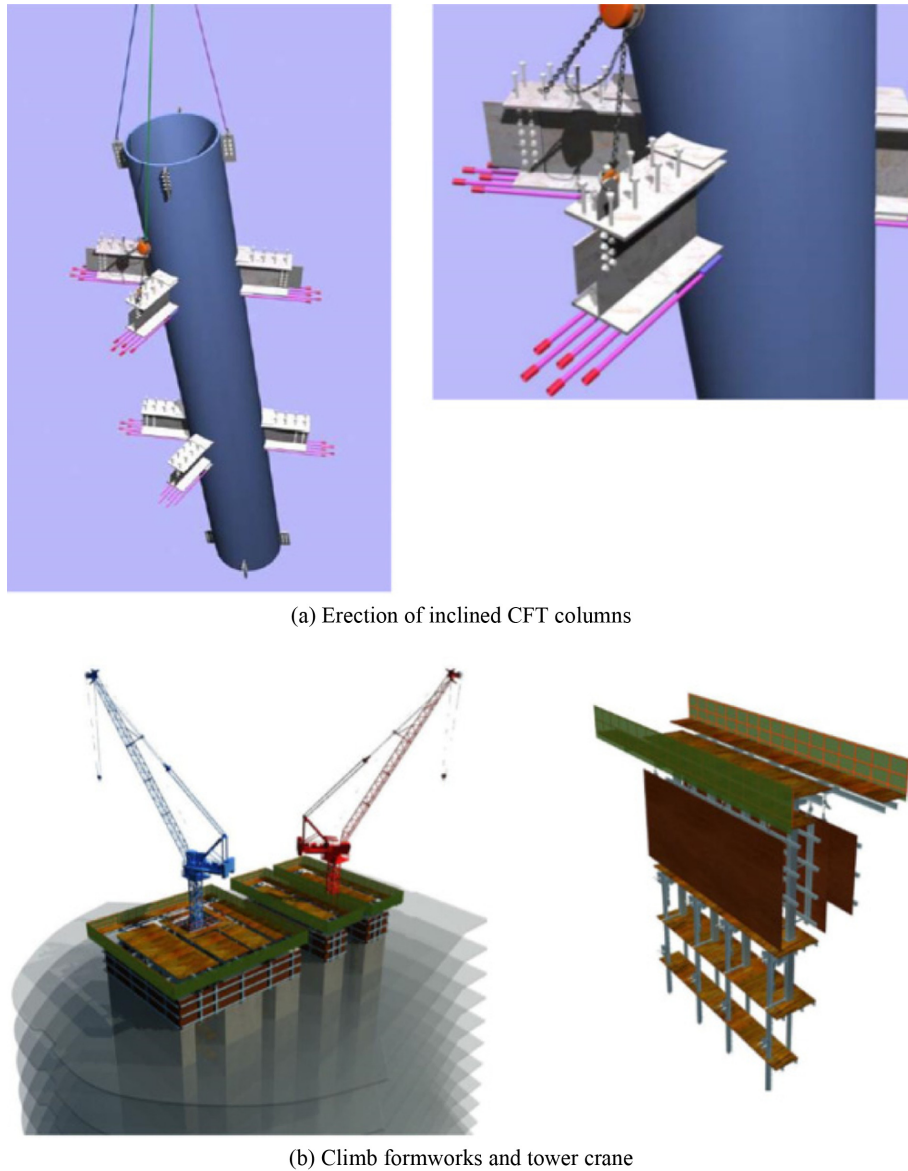


Figure 15. Construction provision for towers.

adopted. In order to conduct precise erection of segmented inclined CFT columns, the flexible jack head was adopted as shown in Fig. 15(a) to adjust the inclination of each steel column during the erection. This was done in conjunction with a dual survey and setting out system of both total stations and GPS positioning. Climbing formworks were adopted together with heavy duty tower cranes as shown in Fig. 15(b). This facilitated an effective vertical transportation and concrete casting and curing as well. Formwork thermal isolation was also adopted during the curing of the high strength concrete of C80 in the cold air during the winter season of Hangzhou. This was integrated with the climbing formworks system for the tower structure.

Fig. 16 shows the construction of the project. Monitor-



Figure 16. Construction of Raffles City Hangzhou.



Figure 17. Project near completion.

ing on the deflection and development of internal stresses was conducted during the construction of the tower structures. The monitored data as compared with the results of the construction simulation provided precious guidance on the construction sequence and pace on site. Monitoring works were also conducted on the overall settlement of the tower structures. The measured value is well within the results from the construction simulation, which proved the overall structural integrity through the construction. It was demonstrated that construction of the typical floor of the tower was within 5 day-cycle as estimated in the design phase of the project. A recent photo of the Project is shown in Fig. 17. With the coverage of the façade, the Project is to be open to public in middle 2017.

6. Conclusion

This paper presents the multiple complexity, challenges and their engineering solutions and provisions during the structural engineering design and construction of Raffles City project in Hangzhou China. Through the introduction and careful elaboration, the following engineering and technical features were addressed:

Design, mock up and construction of free form iconic structures;

Value engineering incorporating the consideration of both cost effectiveness and construction efficiency;

Incorporation of construction and planning in the design phase of the project;

Digital engineering through building information modelling, virtual construction planning and simulation; and

Advanced construction technology adopted for the construction of free form hybrid towers structures.

Multiple physical verification studies and mock ups were also conducted in several major and recognized laboratories,

which benchmarked and served as precious design and technical guidance to improve the engineering design and construction of highrise hybrid buildings. The paper serves as good reference of the design and construction of the other free form iconic building structures.

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