Analysis and Design of a 47-story Reinforced Concrete Structure - Futian Shangri-La Hotel Tower

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ABSTRACT:

The objective of this paper is to give a brief introduction of the structural design consideration, the lateral force-resisting system, sloping outer concrete columns, long span post-tensioned transfer girder and other design challenges for this project.

Many of the challenges in the design of this project are stated as follows: (1). A dual lateral structural system – exterior perimeter moment frame with core wall is adopted. (2). To enhance the seismic ductile behavior of the concrete frame columns, particularly for the sloping columns starting from level 35 as part of an architectural requirement, column with additional confined central reinforcement is used due to high compressive load ratio in the columns. (3). High-strength concrete is extensively used in the columns and core walls at low zone. (4). Deep beams at MEP level at level 35 act as outriggers to control the story drift due to severe wind loads. (5). Local bell-out design at end of interior diagonal beams is used to resolve the shear problem without increasing the beam depth. (6). 30m long span Post-tensioned girders are used for the column-free ballrooms and transfer girder structure to support an average of 1.5m thick soil on landscape roof and one retail floor.
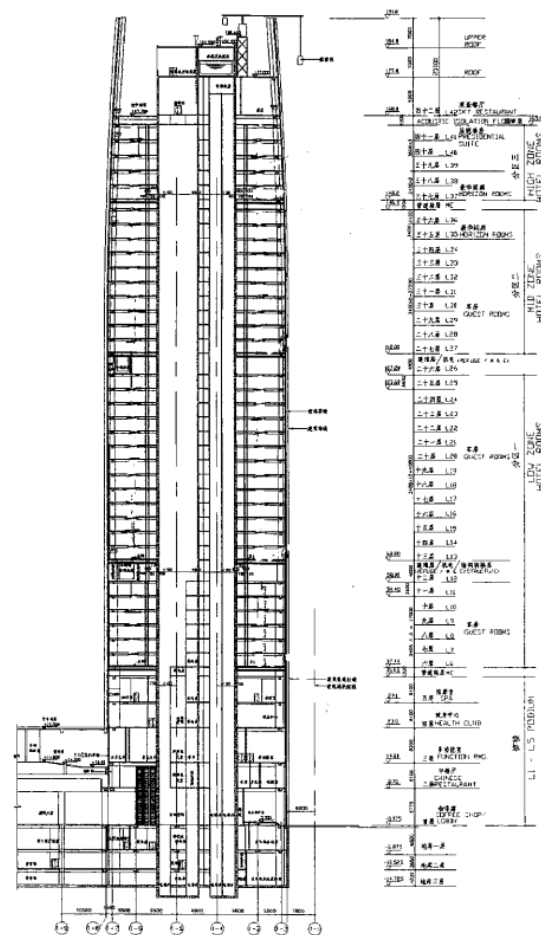
Figure 1a for Elevation View

1.0 INTRODUCTION AND PROJECT DESCRIPTION

Located in the south of central business district in Shenzhen and when completed in year 2008 with the highest point of the structure standing at 190.85m, the Futian Shangri-la Hotel project is amongst the tallest concrete buildings in the city of Shenzhen. The project site is a L shape, approximately 115m in length and 88m in width. (See Figure 1 for Site Plan)

This mix-used development is between Fuhua road and Yi Tian road, which includes a 47-story tall hotel tower, two 100-meter tall 30-story office towers, a 3-story underground car-park, and a 5-story podium. The 5-story podium level includes retail, restaurants and long span structure – a 30m wide Ball room.
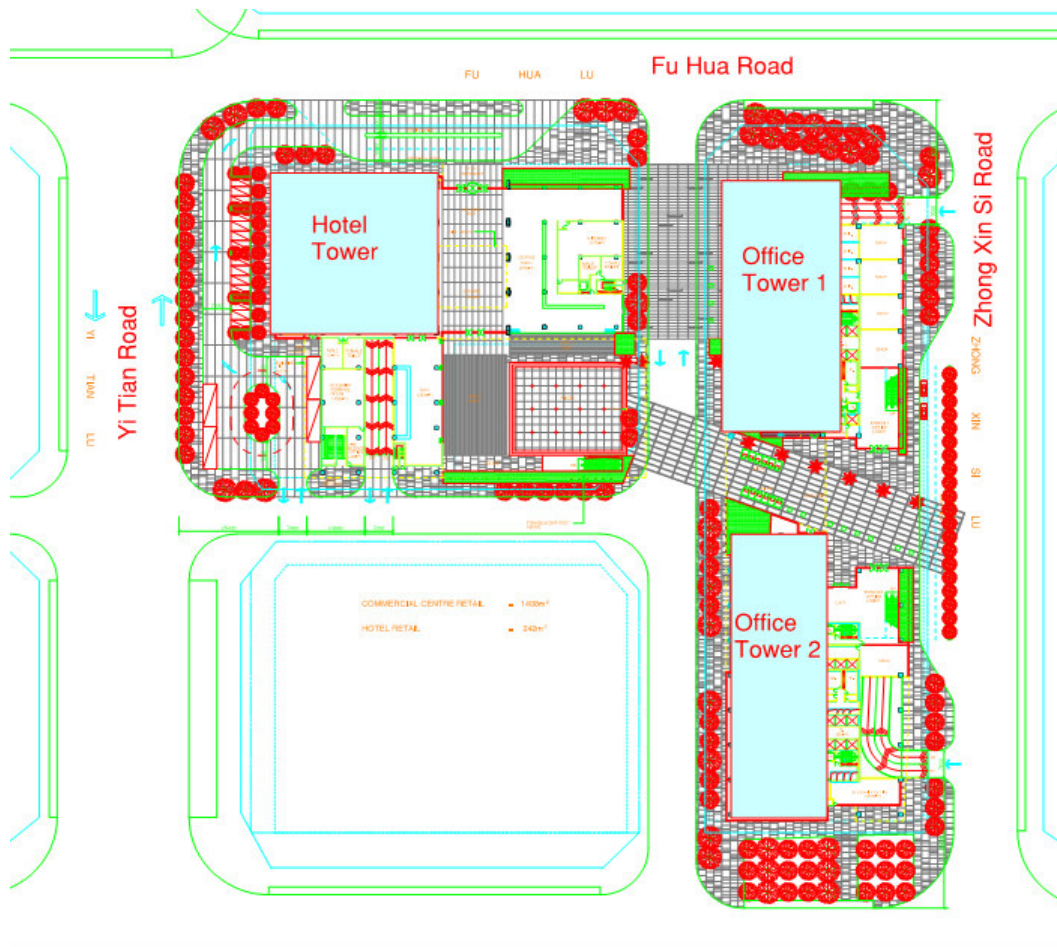

Figure 1a for Site Plan

The tower is approximately 190m tall, with a comparatively small 40 m by 40m footprint. At levels 27, the exterior concrete columns start to bend inward. Per China code requirement for safety, two refuges/MEP levels in the tower are required and located at Level 12 and level 26F. There are additional two MEP levels located above Level 5 and Level 36. At upper floor (Elev. 184.8m), a 20mx20m helicopter platform is designed to receive a 10ton chopper landing for VIP guests. The design life of the structure is 50years. The tower's architectural features posed many

challenges and required the creative use of the technologies and cutting edge innovations. (See Figure 2 for Hotel Tower Rendering and Tower Construction Photo)

Figure 2 for Hotel Tower Rendering and Tower Construction Photo

2.0   DESIGNING THE STRUCTURAL SYSTEM

2.1   Foundation System and Geotechnical Conditions

From the ground level progressing down, the strata are as follow: fill, clay/silt, sand, median coarse sand, clay, sand with rocks, intensively decomposed rock, moderately decomposed rock, and slightly decomposed rock. Elevation of various strata varies through out the site.

The B3 level elevation is -13.6m, the bottom of excavation of the tower is -17.6m and -24.6 for the bottom of the elevator shafts in the core. At the bottom of lowest basement level (24.6m below ground level). Based on the recommendation from geotechnical report and structural analysis results, slightly decomposed rock (bearing characteristic value =9000kPa) can be the bearing strata for the tower. The slightly decomposed rock can be found from 33.2m to 62m below ground level. (See Figure 3 for the contour SDR and Elevation).
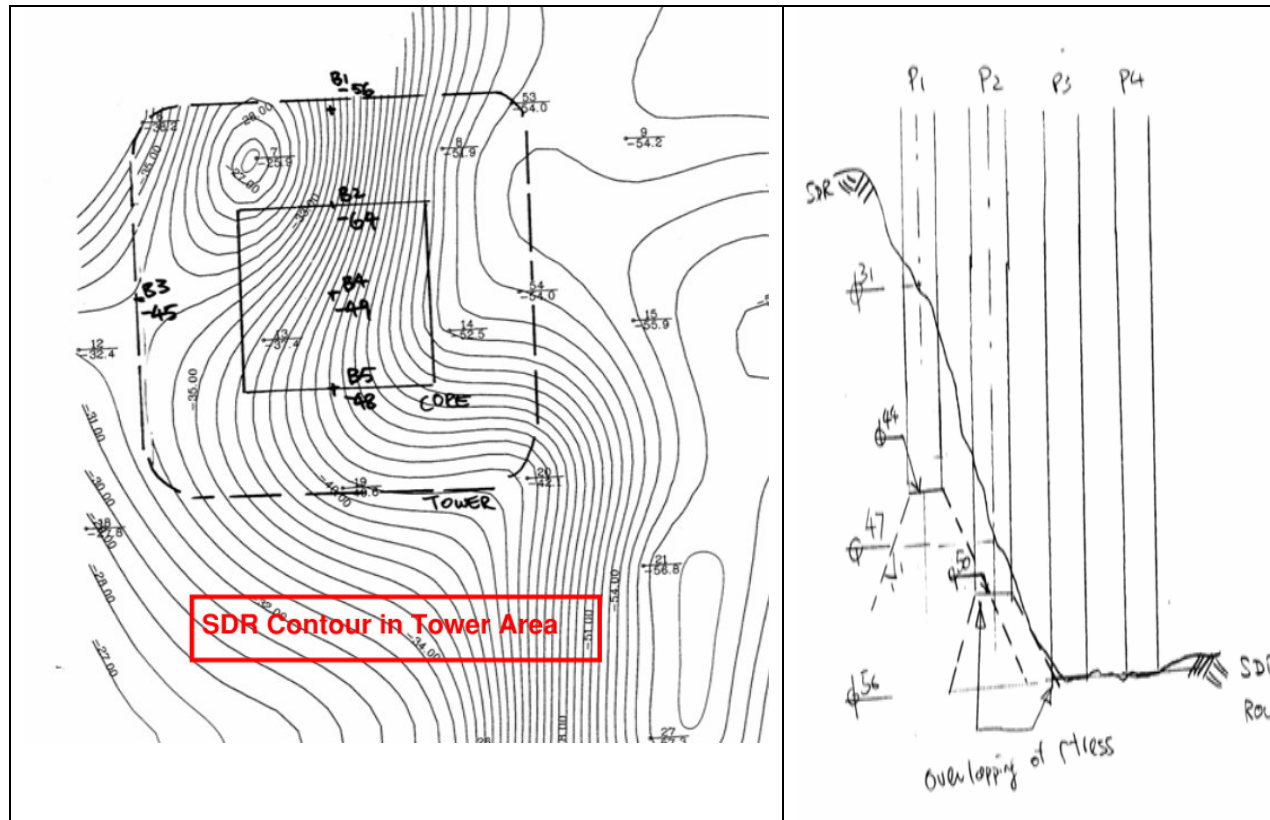
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FIGURE 3 –The SDR contour and Elevation

The length of the piles will exceed 35m even calculating from the bottom of the excavation. Hand dug caissons with diameter varying from 1.2m to 3.1m were found to be the most economical solutions for the foundation. A raft is provided under the central core of the tower to spread the loads to the caissons. (See Figure 4 for Tower Foundation Plan and Construction Photo)

Any foundation design must consider its strength and serviceability. Strength was addressed through pile diameters and lengths, engaging enough friction or end bearing capacity to resist the anticipated loads. A larger challenge on this project was serviceability considering foundation settlement. The diameters of the caisson vary to maintain the same stress level. But the stress level between caissons supporting the tower and podium could be different.

The podium caissons are founded on Moderately Decomposed Granite with smaller bearing capacity
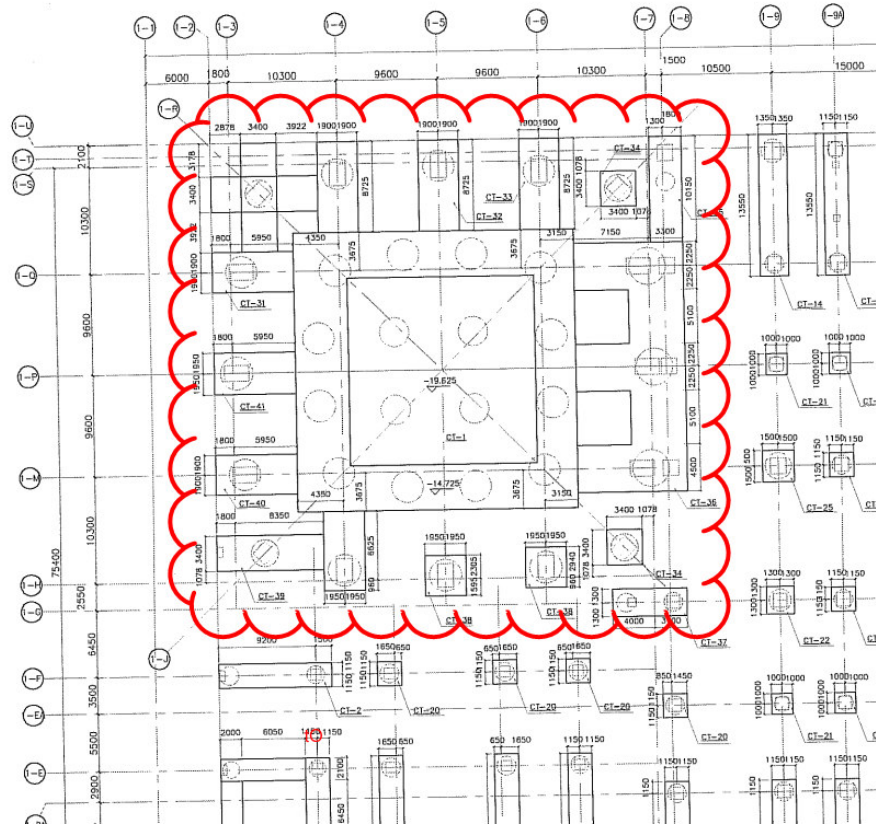

FIGURE 4a- Tower Pile Construction
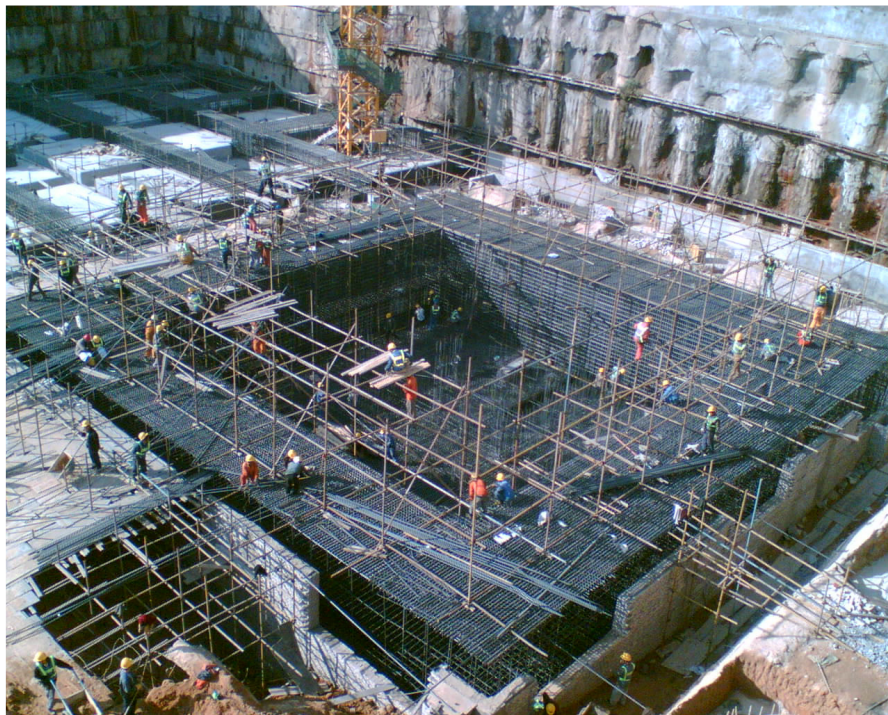
FIGURE 4b- TOWER FOUNDATION AND PILE LAYOUT

FIGURE 4 – TOWER BASEMENT CONSTRUCTION SITE

2.2   Gravity System

The typical floor footprint represents a 40mX40m square with rounded corner. The typical exterior columns are symmetrically spaced from 9.6m to 10.3m in both directions. The span between the exterior perimeter columns and interior core walls is 10.3 m. To achieve the most economical gravity floor system, several gravity floor systems were studied. See Table 1 for the schemes studied.

Among all of the gravity floor systems studied, a typical two-way framed beam and slab approach (Scheme 1) was selected, with 160 mm thick slabs, as the most economical and practical solution consistent with desired ceiling heights and story heights.

The typical perimeter moment frame consists of concrete girders 650 x 650mm (H) framing to columns. Typical interior beams spaced 4.8m on center of perimeter girders with size of 600mmx650mm (H). The beam depth-to-span ratio is 15.8. This is very high for cast-in-place conventionally reinforced concrete gravity beams. To improve their stiffness and reduce deflections under gravity load, all of the gravity-bearing beams are framed rigidly into the core walls, as the walls offer ample strength and stiffness anyway for lateral load resistance.

Because perimeter spandrel girders are not specifically reinforced to resist torsion, filler beams framing to them were considered as pinned at that end. Thus one end of the beams was released in the girder side and the other end of the beams was fixed in the walls. (See Figures 5a and 5b) This approach meant filler beams could not frame to coupling beams between core walls, leading to some beam skews.

Possibilities for making beam framing and formwork more regular such as parallel or radial beam layouts were studied, but it introduced adverse compromises not justified by potential savings. The shapes of floor edges and core walls also did not support repetitious patterns. So formwork repetition is possible floor by floor, but not within a floor.

TABLE 1 - FLOOR SYSTEM STUDY AND COMPARISON

ITEM	SCHEME 1 Two Ways Slab with Filler Beam	SCHEME 2 Deck with Steel Beams	SCHEME 3 Concrete Two-way Slab	SCHEME 4 Two-way Post-tensioned Slab
Beam Depth	650 mm	600 mm with deck	280 mm slab	215 mm slab
Construction Speed	Normal & more form works	Faster	Normal & least form works	Slower
Construction Coordination	No coordination required	Coordination required	No coordination required	Coordination required
Construction Ability	Common practice	Not common practice	Not common practice	Limited contractor
Cost and Other factors to be considered	cheapest	Much high than Concrete Schemes	Needs high reinforcement.	Higher than all of the other concrete slab schemes.
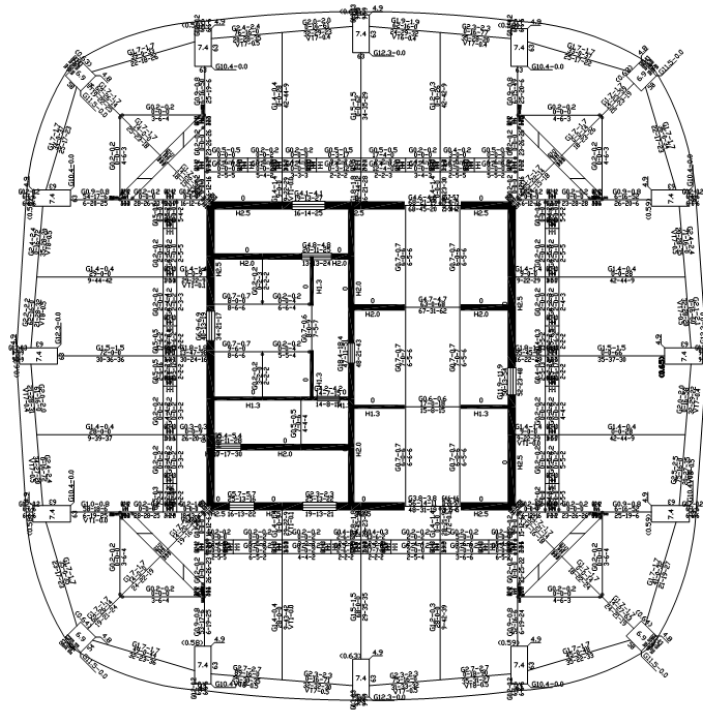
第20层混凝土构件配筋及钢构件应力比简图　单位

FIGURE 5a - TYPICAL HOTEL FLOOR FRAMING AND REINFORCEMENT FROM SATWE MODEL

FIGURE 5b –HOTEL TOWER LOWER FLOOR CONSTRUCTION PHOTO

2.3 Lateral Load Resisting System

2.3.1 Description of the Dual System and Design Considerations:

The design of the lateral system was based on several parameters including, but not limited to, desired structural lateral stiffness, typical floor center of the rigidity compared to center of mass and structural dynamic properties. The primary lateral load-resisting system for this building is a core with coupled concrete shear walls plus a perimeter concrete moment frame to create a dual system as required for such height

Several considerations prompted the design team to use the dual system. First, per China Technical Specification for Concrete Structures of Tall Buildings JGJ 3-2002, a dual system is requested for structures with Class B height (below 180m but higher than 150m). Second, the architectural layout design permitted closed perimeter frames between perimeter columns. As a result, the perimeter moment frame with concrete core meets the requirements of dual system.

High-strength concrete C60 (approximate f'_c = 7.4 ksi) was used for the concrete columns and shear walls from B3 to Level 26 to minimize the column size and wall thicknesses. In addition, the concrete columns are designed for high compressive axial force ratio by adding confined central reinforcement to improve the seismic performance and ductile requirement (See Figure 5 for Typical Concrete Reinforcement Details).

In an effort to maximize the tenant spaces, core wall thicknesses were minimized. As part of this effort, the contribution of floor beams between exterior columns and cores walls was studied to see if they could function as additional moment frames. They were found capable of contributing 3 to 4% to overall building lateral resisting stiffness.

The concrete core provides most of the lateral stiffness for the structure. For efficiency, the exterior flange walls are thicker and act as I flanges in tension and compression. The web walls, in contrast, are just 0.4 m (1.6 ft) thick to some shear forces. Table 2 summarizes sizes of the shear wall thickness and Column Sizes In Table 2. .

TABLE 2 - SHEAR WALLS AND TYPICAL COLUMN SIZES

| Floor Level | CORE WALL | | COLUMN | | Conc. Strength |
	Flange Wall (mm)	Web Wall (mm)	BXH (m)	Compressive Axial Force Ratio	
B3-5F	700	400	1.6X2 – 1.6x1.6	0.76	C60
6F-11F	700	400	1.4X2- 1.5x1.5	0.72	C60
12F-18F	700	400	1.1X2 – 1.4x1.4	0.74	C60
19F-22F	600	400	0.8X2 – 1.2x1.2	0.80	C60
23F-26F	600	400	0.8X2 - 1.1x1.1	0.72	C60
27F-32F	500	400	0.8X1.5 - 1x1	0.77	C60
33F-39F	400	400	0.8X1.2 -0.9x0.9	0.84	C50
40F-43F	400	400	0.8X1.2 – 0.85x0.85	0.54	C50
44F-TOP	400	400	0.6X0.8 – 0.8x0.8	0.55	C40

The decision to use conventional cast-in-place concrete framing, a lateral system primarily based on concrete core walls and gravity framing with two-way slabs and beams resulted in an economical and functional building constructed exclusively using local labor and materials.
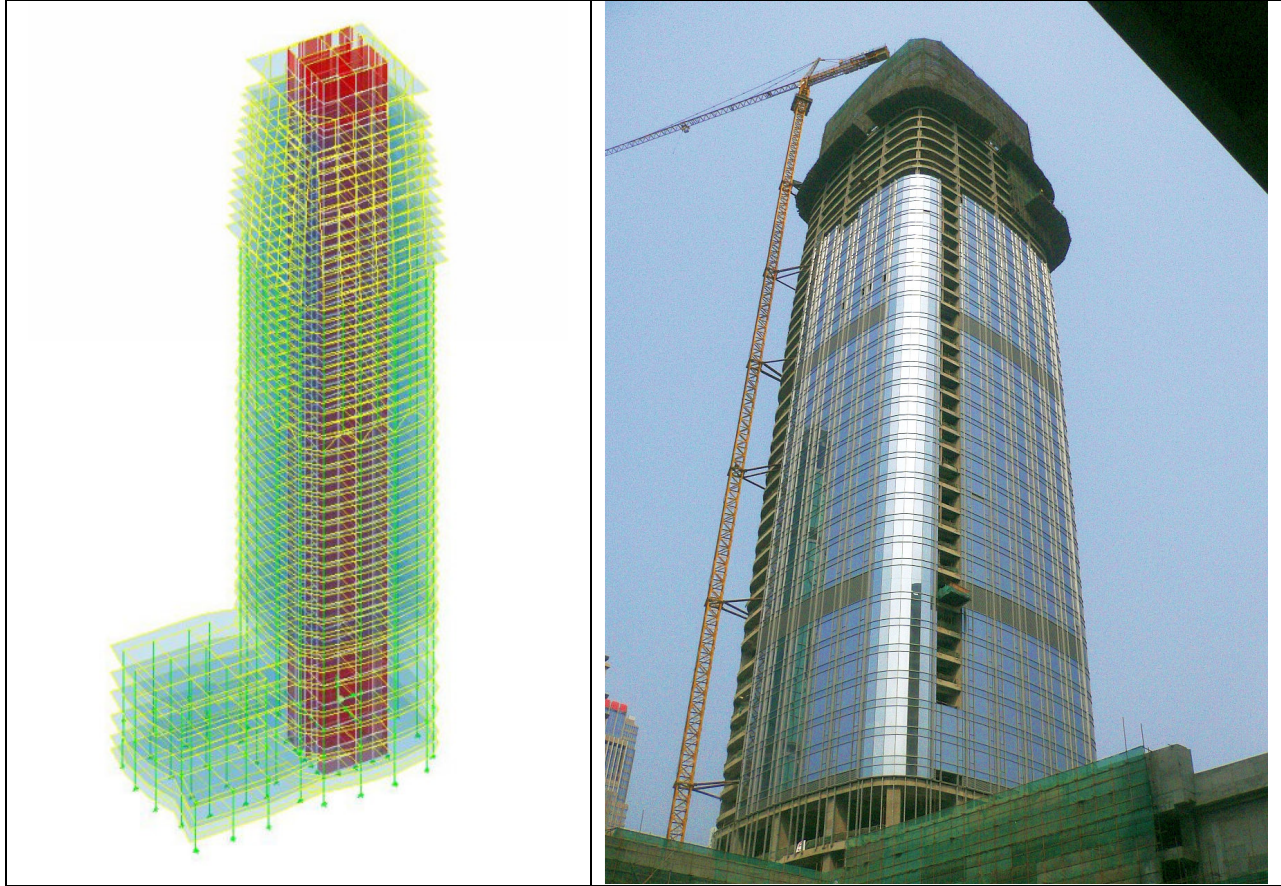

FIGURE 6 - STRUCTURAL 3D ETABS MODEL AND TOWER CONSTRUCTION VIEW

2.3.2 Response Spectrum and Elastic Time History Analysis Results

Two computer models were used in the project: The preliminary results from SATWE and ETABS are used to verify the building periods, floor mass, wind forces, seismic forces and story drifts. Response spectrum analysis was used in the seismic force design for reinforcement determination for all of the structural elements. The following Tables and graphics are the summary of the analysis results.

TABLE 3 – Primary Building Periods Summary

	Tx	Ty	Tt
SATWE	3.861	3.754	2.373*
ETABS	3.889	3.318	1.815

TABLE 4 – Story Drift Summary

Direction	Story Drift	SATWE	ETABS
Wind X	△Umax/h	1/1370	1/1127
	Floor No	48	35
Wind Y	△Umax/h	1/1388	1/1404
	Floor No	48	34
EQ X	△Umax/h	1/1609	1/1855
	Floor No	48	35
EQ Y	△Umax/h	1/1605	1/2226
	Floor No	48	34

TABLE 5 – Building Horizontal Irregularities Check

Direction	SATWE	
	Tower	
EQ X	Umax/Uave	1.28
	Floor no	4
EQY	Umax/Uave	1.20
	Floor no	4

There time history spectrums have been selected for the elastic time history analysis. The following are the summary of the results:
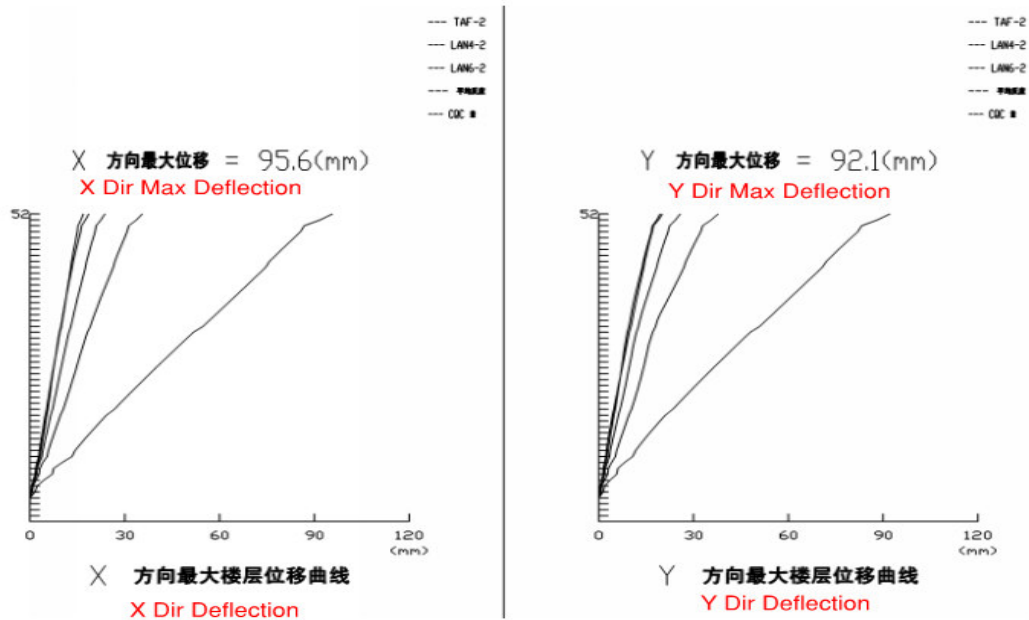

X 方向最大位移 = 95.6(mm)
X Dir Max Deflection

Y 方向最大位移 = 92.1(mm)
Y Dir Max Deflection

X 方向最大楼层位移曲线
X Dir Deflection

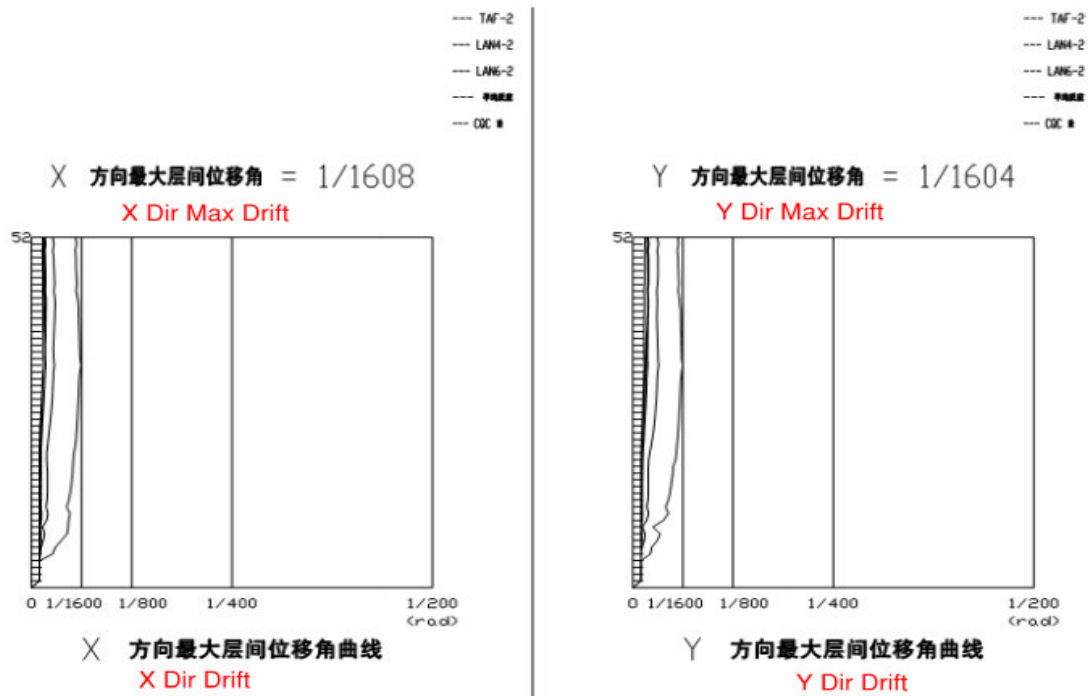Y 方向最大楼层位移曲线
Y Dir Deflection

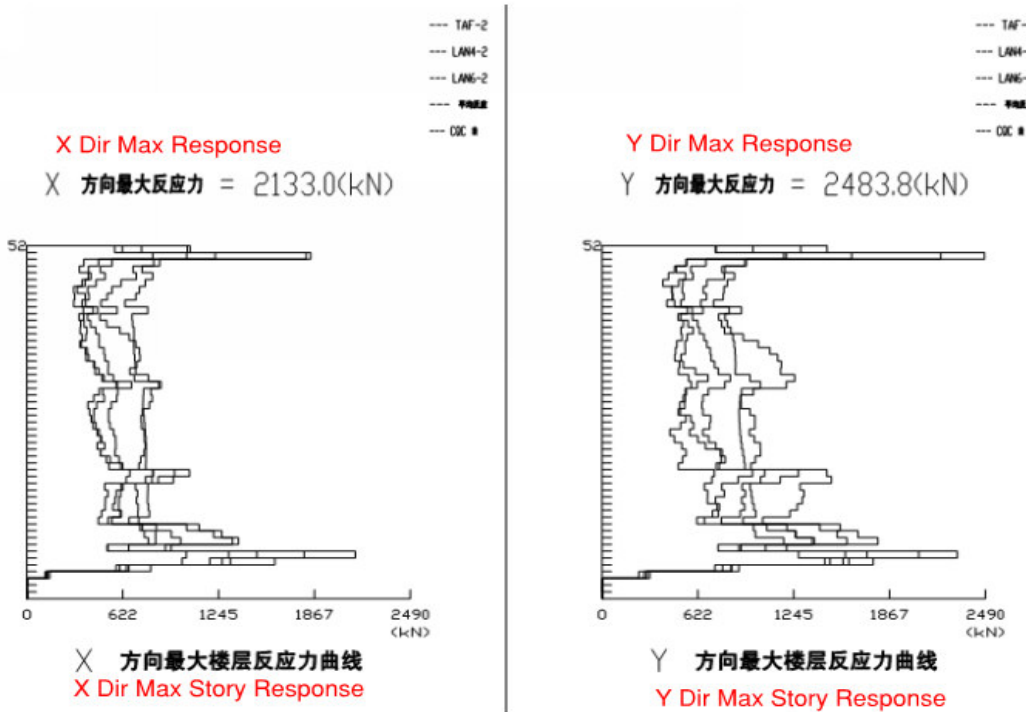FIGURE 7 – Deflections

FIGURE 8 – Inter-Story Drifts

FIGURE 9 – EQ Story Shear Response

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2.4   Podium Structure and Long Span Post-tensioned Transfer Girders

The podium structure housing the hotel's grand ballrooms presented another challenge to the structural engineer: the column-free ballroom called for a 30m-span floor structure with the smallest structural depth possible. Added to the challenge is that the long-span structure has to support an average of 1.5 meter thick soil for roof landscaping. Again several schemes had been considered: structural steel trusses, concrete-steel composite beams, and post-tensioned girders. The structural steel scheme was rejected as being the most costly option. Balancing between the remaining options of composite and post-tensioned design, the former was more favorable at first because of its more prevalent use. The post-tensioned concrete option, though quite widely use in civil structures in China, is seldom used in commercial buildings. Nonetheless, upon more detailed structural analysis and cost evaluation, TT was convinced that the post-tensioned concrete design would present a more economical solution yet provide higher headroom for the ballroom. Several post-tensioned concrete specialist sub-contractors were interviewed until adequate number of potential tenderers with capability to carry out the task was confirmed before a final decision was reached.

Construction of the post-tensioned podium structure is now complete and it stands as one of few such structures in the private sector in China.
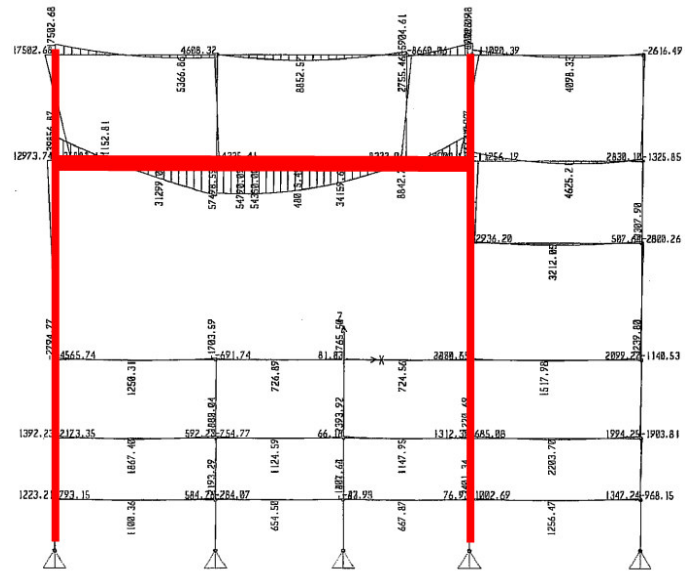

FIGURE 10 – Transfer Girder Moment Diagram under Dead Load and Live Load

TABLE 6 – 30m LONG SPAN GIRDER AND TRANSFER GIRDER STUDY AND COMPARISON

ITEM	SCHEME 1 Conventional moment frame	SCHEME 2 Steel encased composite girder	SCHEME 3 Post–tension girder	SCHEME 4 Steel truss and column
30m Long Span Girder	1.2(B)x3.0m(H)	1.0(B)x2.5m(H)	1.2(B)x2.5m(H)	2.5m(H)
30m Transfer Girder	1.4(B)x3.8m(H)	1.2(B)x2.8m(H)	1.4(B)x2.8m(H)	2.8m(H)
Construction Ability	Common practice	Not common practice	Limited contractor	Limited contractor
Cost and Other factors to be considered	cheapest	Much high than Scheme 1	Slightly higher one but architectural desired.	Highest

2.4   Special Details for Bell-out Beam End, Concrete column with central reinforcement and Sloping Column

The shear capacity of the diagonal interior beam exceeds the maximum beam shear capacity at the end connection to the core walls. To solve this problem without widening the beam width, a bell-out type detail is designed and detailed at the beam connection at the four corners of the core wall (See Figure 11).
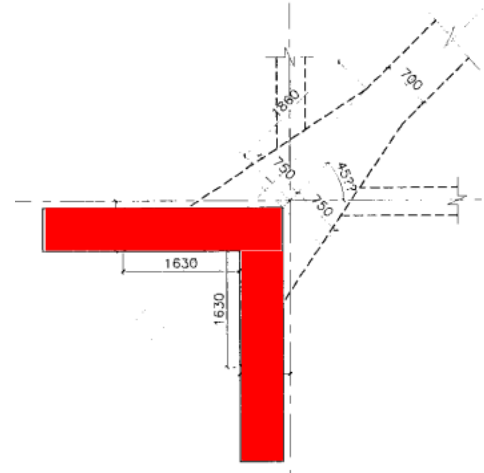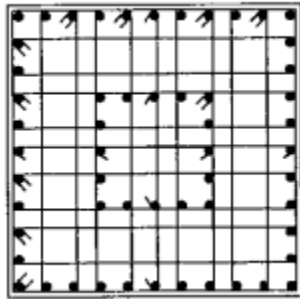

FIGURE 11 – Bell-out Beam Connection

Figure 12 Central Reinforcement in columns

From the table 2 above, some column compressive axial force ratios have exceed the maximum 0.75 for Class I structure per China National Seismic Design Code (GB 50011) and architect prefers to keep the column sizes as small as possible in certain floor level because of the architectural layout. To solve this problem, Composite scheme -Steel encased in concrete and Scheme with adding Central reinforcement in columns can be considered in the design. Obviously the second option will be cheapest to be selected by the engineer as the best economical solutions (Refer to Figure 12).

The concrete column starts to slope inwards from third floor. Although it is common practice for most of the steel structures, sloping concrete columns are used and the additional moment due to the offset of the column loads have been studied and additional reinforcement is added in the outer sloping column. (Refer to Figure 13).
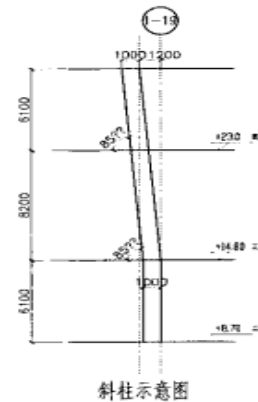

Figure 12 Sloping Column Illustration

3.0 CONCLUSION

As a sub-consultant to Canwest Consultants Ltd., a local Hong Kong engineering firm, Thornton Tomasetti's main scope of work included the design of the 180-meter tall hotel tower and the long-span podium structure.

Futian Shangri-La project provides an excellent example of high rise design and construction under challenging conditions, including severe typhoon wind load, sloping outer column, heavy loaded post tensioned transfer girder and a conservative building code. The design team achieved economical structural solutions without compromising aesthetic design integrity. The result is a beautiful new landmark for the City of ShengZhen.

4.0 CREDITS

The authors would like to thank that project manger C. Tam, Len Joseph, Zhu Yi, Torsten Gottlebe, Jase Wong, Tian Chuan Lin, Charles Mui, William Chung, and others for their work during design and would like to thank Jase Wong and Tian Gong for their work during design and construction, especially to Torsten Gottlebe for his tremendous work on this project.

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