Structural Design Challenges for Plaza 66 Tower 2

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ABSTRACT:

The Plaza 66 Tower 2, recently completed in Shanghai, China, balances the demands of structural engineering, architectural vision and construction constraints. This is a 46-story concrete moment frame and core wall structure. The unique design by architect KPF posed many engineering challenges to be resolved. Although modern computer technology allows for precise analyses and design of various systems for high-rise buildings, it does not readily provide insight for choosing among the alternative systems to arrive at the best overall design.

This paper presents our design and analysis process, from the conceptual phase to detailed design, during which we resolved the following engineering challenges: (1) Determining and mitigating effects of pile mat foundation differential settlement, addressed through multiple stages of finite element analysis models. (2) Determining a cost-effective structural system by developing multiple options, using approximate analysis, and a cost comparison database. (3) Designing a curved 65 m (215 ft) long span steel bridge between two towers, an important architectural and functional feature. (4) Designing and detailing another important architectural feature, a two-way sloped steel cantilever thin steel lantern, using a three-dimensional structural information model to help both analysis and accuracy in connection details to benefit all members of the project team.

1.0 INTRODUCTIONS

Plaza 66 Tower 2 is the new addition to the Plaza 66 Tower 1 project in Shanghai, China. Tower 1, with a height of 281.5-m (925-ft) and 62- stories, which was completed in 2001, is the tallest concrete building in the Pu Xi area. Tower 1 has a five-story retail podium and three levels of below-grade parking. Together the complex forms a three-million square-foot mixed-use commercial development. (See Figure 1)

Tower 2 is adjacent to the existing Tower 1 and its podium. Tower 2 was completed in 2006, a 46 story tall building reaching 223.5 m (735 ft) high. The structural system is a concrete moment frame with concrete shear walls. Tower 2 has three below-grade podium parking levels connecting to existing below-grade parking levels.

Finding the most appropriate structural solution for Tower 2 to fulfill a given set of performance requirements required a complex, multi-disciplinary process. This paper explores the design challenges of the tower and the solutions found for poor foundation conditions in a seismic and typhoon area, settlement effects on the new tower and the existing tower on site, a curved bridge connecting between two towers and a complex steel roof lantern with two-way slopes.

2.0 PROJECT DESCRIPTION

Plaza 66 Tower 2 is in the Shanghai Pu Xi Business District. The West boundary of the tower is Xi Kang Road. South and East of Tower 2 is the Plaza 66 Tower 1 complex completed in 2001. The current project footprint is approximately 6,800 sq. meters. Tower 2 is a 46 story structure with a three level basement to be constructed on site. (See Figure 2)

Tower 2 is approximately 235 m (735 ft) tall, with a comparatively small 40 m by 69 m (131.2 ft by 223 ft) footprint. At levels 13 and 14, a two-story curved pedestrian bridge connects with existing Tower 1. At level 45, partial shear core walls with four super composite steel columns standing on four one-story high concrete transfer girders lift the steel roof lantern 43.5 m (142.5 ft) towards the sky.

FIGURE 1 - PLAZA 66 TOWER 2 (NEW) AND TOWER 1 (EXISTING)

Where three underground parking levels connect to adjacent existing underground parking levels, special details were developed to permit removing a temporary slurry wall between the existing structure and new structure while maintaining the same floor elevation.

The tower stands on a thick concrete mat supported on deep piles. The adjacent podium stands on a thinner pile-supported mat.

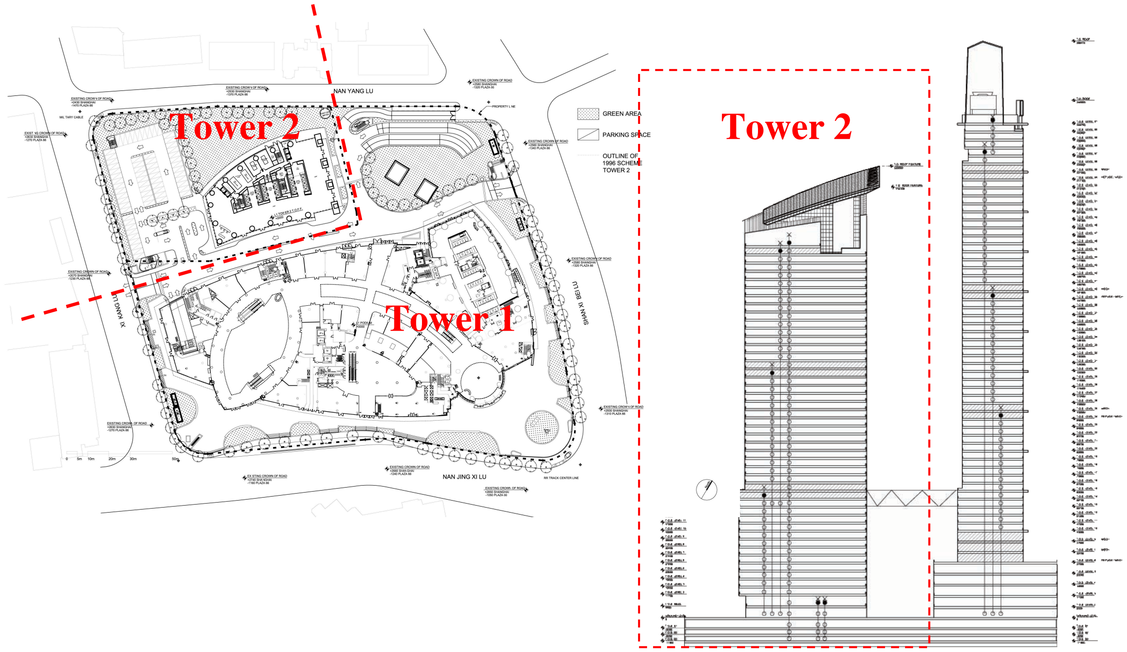


FIGURE 2 - PLAZA 66 TOWER 2 SITE MAP AND BUILDING ELEVATIONS

3.0 DESIGNING THE STRUCTURAL SYSTEM

3.1 Foundation System

The city of Shanghai is situated on an estuary of the great Yangtze River, which has deposited sediment here for millennia. As a result, subgrade conditions across most of the city are poor, consisting of thick strata of highly compressible clays and sands. The top 15 m (49 ft) of soil is usually silty clay or silty sand with poor friction and bearing capacities. Nine layers of sand and clay alternate to great depth, more than 120 m (394 ft) below grade. Bedrock is beyond reach of any practical foundations.

Because near-surface soil conditions in Shanghai are not conducive to supporting the heavy loads of tall towers, concrete bored piles are used for the foundations. These piles were drilled from grade, extending about 80 m (262 ft) down for 9 m embedment within a dense sand-bearing stratum (Layer 9-1). The piles are 800 mm (2 ft 7 in) in diameter. One pile diameter and end depth length is used for both the podium and tower. This approach was selected to save equipment set up time and permit using a single group of workers for the entire construction site.

The piles support concrete mats under the tower and the podium; different pile spacing provides different load bearing capacities for the two very different structural conditions. (See Fig. 4a) Design elevation of the mat foundation is -12.25 m to allow room for three basement levels. A 3.5 m thick mat is provided below the tower and 1.2 m thick mat is below the remaining podium/basement areas.

As a riverside city, Shanghai has a water table close to the ground surface. The presence of sand strata makes permanent dewatering beneath the buildings impractical, so the design considers worst-case load combinations covering both maximum downward loads and minimum gravity loads combined with uplift pressure on the mat from buoyancy. The design capacity of a single pile is 6200 kN for compression and 2800 kN for tension.

The exterior basement walls are cast as slurry wall panels. In the podium area they are 800 mm (32") thick and 25 meters (82 feet) deep. (See Figure 3) In the tower area the wall is 1 m (39 inches) thick and 30 m (98 feet) deep. These slurry walls are used as permanent foundation walls as well. The great toe depth reflects the need to develop sufficient friction resistance to keep the wall from sinking into the soft soil under its own weight and minimal superimposed load. A temporary slurry wall on the East and South sides was constructed during phase one of construction. It is selectively demolished in this phase to permit linking basement parking levels.

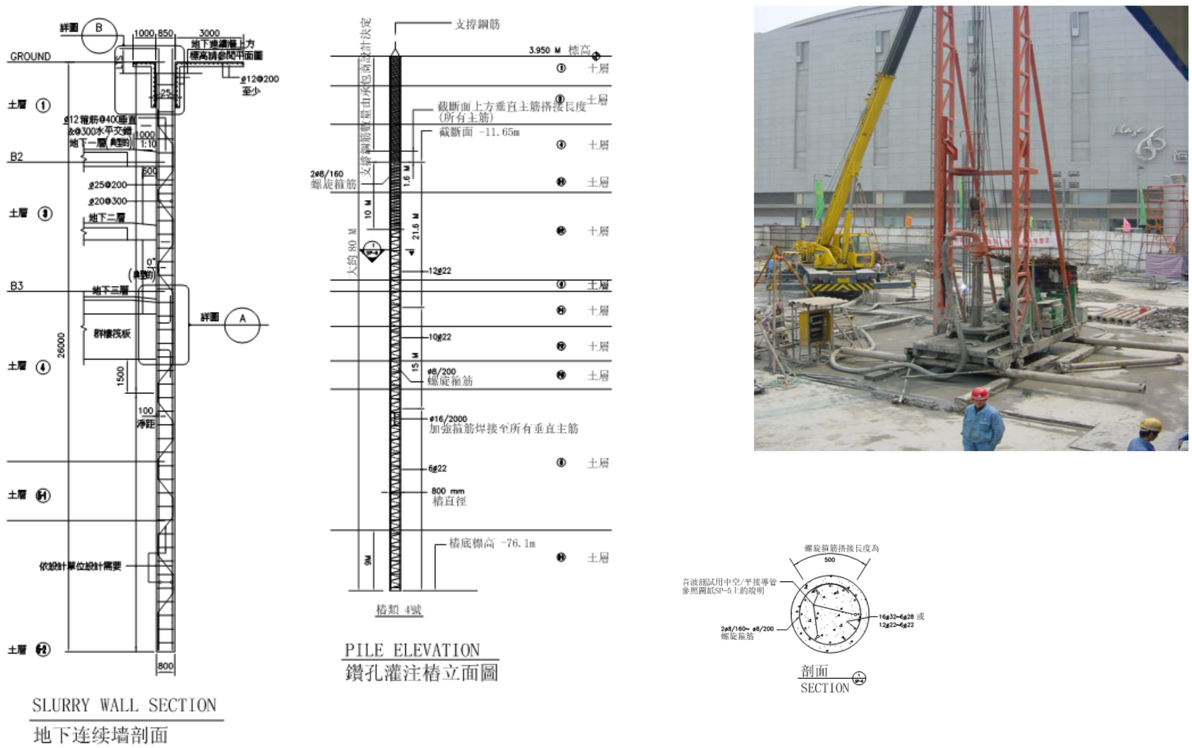


FIGURE 3 - SLURRY WALL SECTION, BORED PILE ELEVATION AND SITE PHOTO

Any foundation design must consider two main requirements: strength and serviceability. Strength was addressed through pile diameters and lengths, engaging enough friction and end bearing capacity to resist the anticipated loads. A larger challenge on this project was serviceability considering foundation settlement. The clay strata under the site are normally consolidated, so they behave like a saturated sponge: any increase in compression stress squeezes water out of them over time. This behavior, combined with great strata thicknesses, led us to anticipate significant settlement during and after construction of the tower. Compression of sand layers can also contribute to settlement. Because the loads applied are so different, tower and podium settlement was also anticipated to be quite different. As settlements in clay are relatively localized to the loaded footprint, we incorporated a pour strip between the podium and the adjacent towers to allow some differential movement before casting concrete that would link them together (See Figure 4a). We conducted settlement analyses and studies considering different stages of construction and loading:

Stage A: Construction phase with two separate mats for settlements and design

Since 80% to 90% of the design dead load is structural concrete self-weight, most short-term settlement in the short term will take place during construction. Since the pour strip between tower and podium mats would initially be open to allow initial independent settlement of the two structures, two separate mats on different pile spring supports were reflected in one SAFE model. Settlements at different control points on the SAFE model were found, recorded and compared to actual settlements for the tower and podium mats. Most of the settlements were consistent with the computer analysis results. The anticipated worst settlement point at the podium was about 30 mm and under the tower about 78 mm during this stage. Floors were cast at different elevations to help compensate for this. The late pour strip was also designed wide enough to permit handling any remaining differences in as-built elevations by local slopes.

Stage B: One mat settlement and design in service

Once tower construction topped out, the majority of the short-term settlement has occurred, and concrete is placed to fill the mat pour strip for service use. From that point on, as the podium mat and tower mat are integrated into a single mat, further differential settlement generates forces in the combined system. Those forces also influence subsequent settlements in a redistribution process. We studied this using another SAFE model with a combined mat. The worst long-term settlement during service was estimated to be 35mm, which is controllable. The total estimated settlement for the tower is the sum of short and long term settlements.

The three-story below-grade foundation walls are almost entirely below the water table, so lateral pressures are large. Construction used an innovative combination of slurry wall panels and concrete cross-lot bracing. Three horizontal planes of temporary bracing were placed in the podium and horizontal four planes of temporary bracing were used in the tower area. Bracing was constructed from top down: as excavation depth approached the elevation of a basement floor, excavation paused and trenches were dug in which the temporary concrete braces were cast. The braces tied into previously-cast temporary piles for gravity support. Then excavation proceeded below, leaving the braces exposed as 'flying beams.' Once the excavation bottom was reached, the mat was cast and each basement floor was constructed working bottom-up. As a basement floor was completed, the corresponding bracing plane just above it was released and dropped onto the floor for disposal. (See Figure 4b)

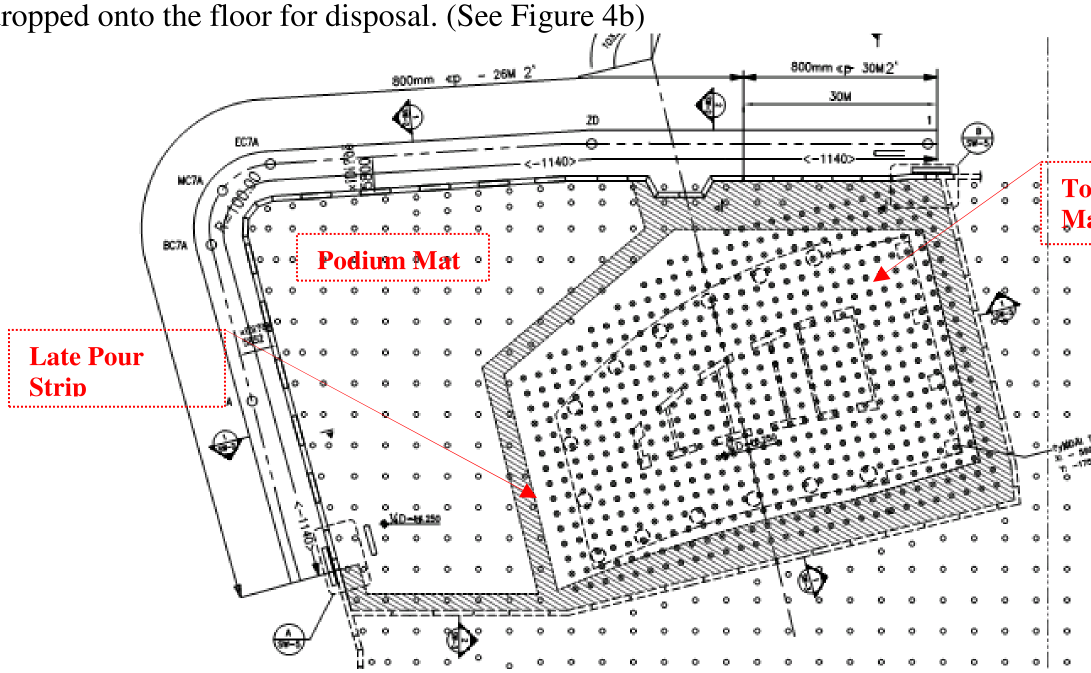


FIGURE 4a- PILE FOUNDATION AND PILE LAYOUT

FIGURE 4b - BASEMENT CONSTRUCTION SITE

3.2 Gravity Floor System

The typical floor footprint represents a truncated angular arch (See Figure 10) 60 m (196.8 ft) long and up to 38.5 m (126.3 ft) wide. The typical exterior columns are uniformly spaced at 9 m (29.5 ft) along a north-south axis and 10.5 m (35 ft) on an easterly axis. The span between the exterior perimeter columns and interior core walls is 11 m (36 ft). To achieve the most economical gravity floor system, several gravity floor systems were studied. See Table 1 for the studied schemes.

Among all of the gravity floor systems, a typical one-way beam and slab approach with filler beams at third points along main girders (Scheme 1) was selected, with 125 mm (5-in) 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 500 x 975 mm (20 x 39 in) deep framing to columns. Typical interior beams spaced 3 m (10 ft) on center span between the core and perimeter frame. The typical interior beam size is 450 x 600 mm (18 x 24 in) deep.

The beam depth-to-span ratio is 16.9. 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 core wall bearing and 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.

We studied possibilities for making beam framing and formwork more regular such as parallel or radial beam layouts, but found they 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 Slab with 2 Filler Beams	SCHEME 2 Slab with 1 Filler Beam	SCHEME 3 Deck with Steel Beams	SCHEME 4 Concrete Two-way Slab	SCHEME 5 Two-way Post-tensioned Slab
Beam Depth	600 mm	700 mm	600 mm	280 mm (no beam)	215 mm (no beam)
Construction Speed	Normal & more form works	Normal & less form works	Faster	Normal & least form works	Slower
Construction Coordination	No coordination required	No coordination required	Coordination required	No coordination required	Coordination required
Construction Ability	Common practice	Common practice	Not common practice	Not common practice	Limited contractor

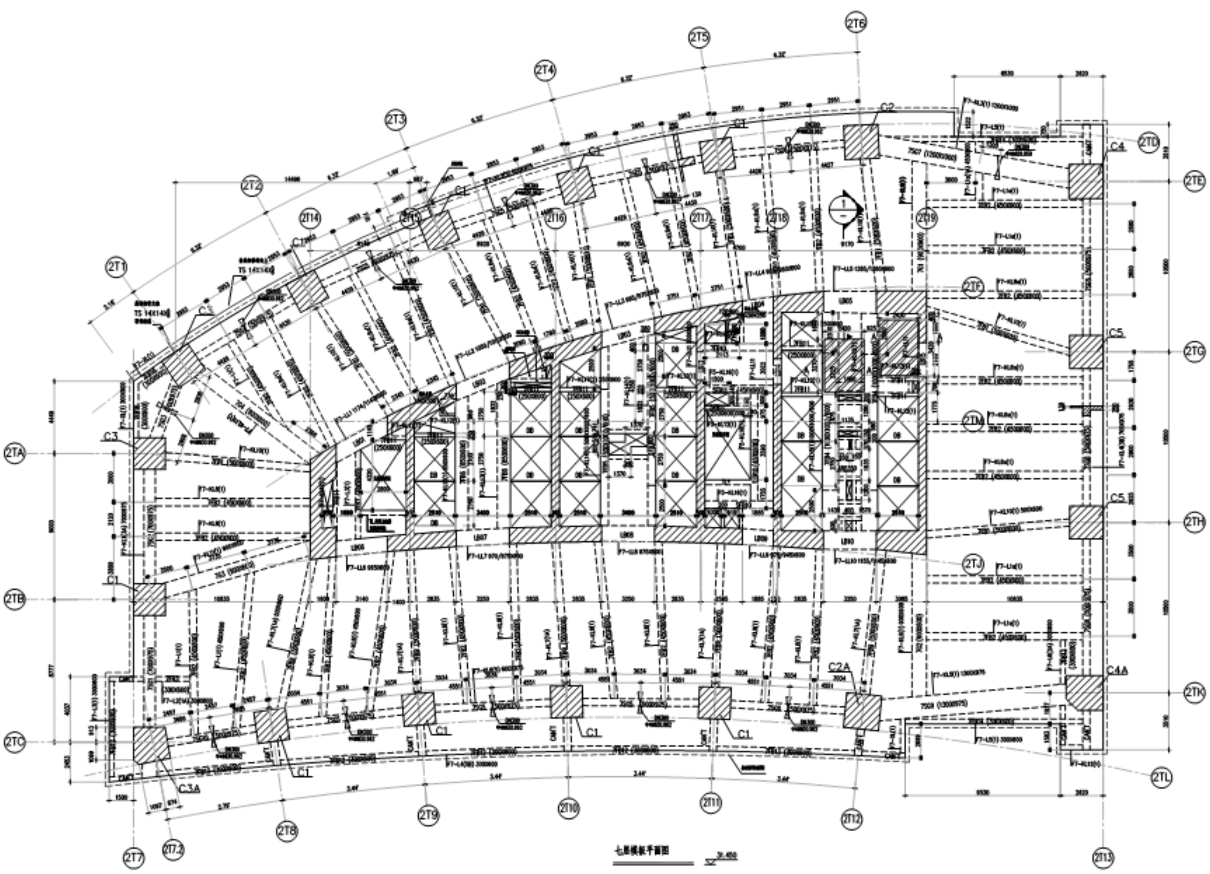


FIGURE 5a - TYPICAL FLOOR FRAMING

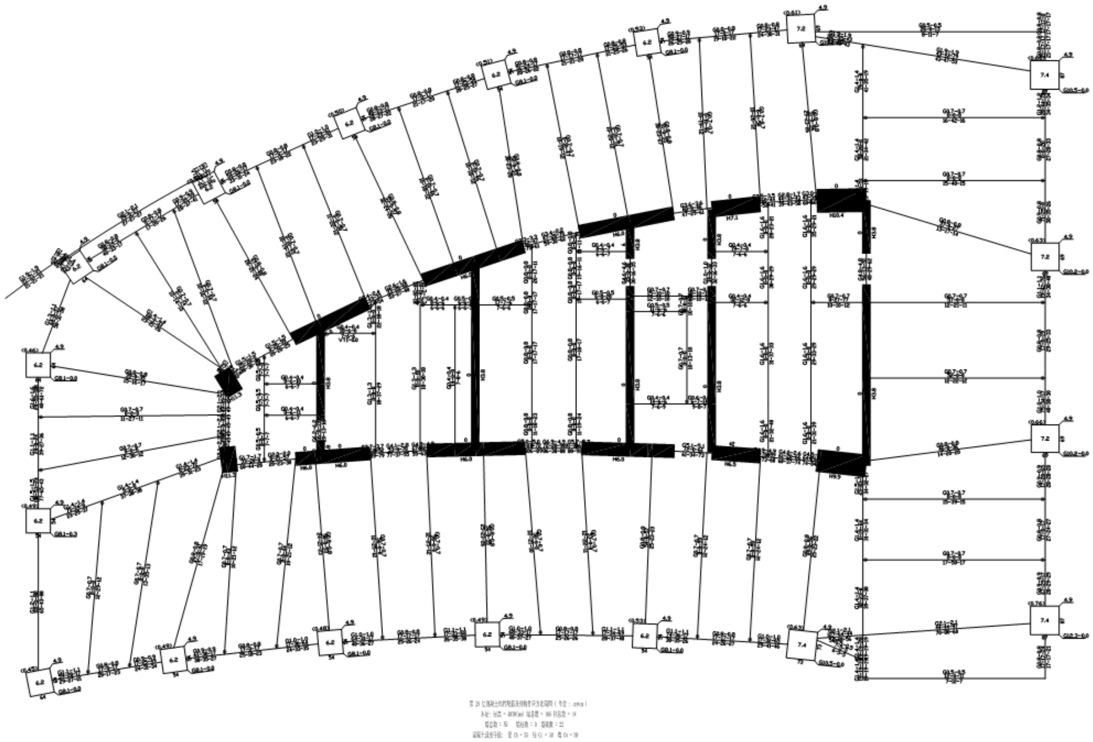


FIGURE 5b - TYPICAL FLOOR REINFORCEMENT OUTPUT FROM PKPM MODEL

3.3 Lateral Load Resisting System

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 of coupled concrete shear walls with a perimeter concrete moment frame to create a dual system.

High-strength concrete C60 (approximate f'_c = 7.4 ksi) was used for the concrete columns and shear walls to minimize the column size and wall thicknesses. Typical 975 mm (39-in) deep by 20" (500 mm) wide perimeter girders were used for special seismic moment frame action. On upper floors, the column sizes are reduced and some of the core walls terminate, increasing the rental space.

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 6 to 8% to overall building lateral resisting stiffness.

Several lateral-load considerations prompted the design team to use the dual system. First, the China code requires a dual system for structures with Class B height. Second, the architectural design permitted fairly deep perimeter beams for spanning between perimeter columns at 9 m spacing. As a result, the perimeter frame, continuous from Level 3 to top, meets the requirements of a Class I frame as a special moment frame.

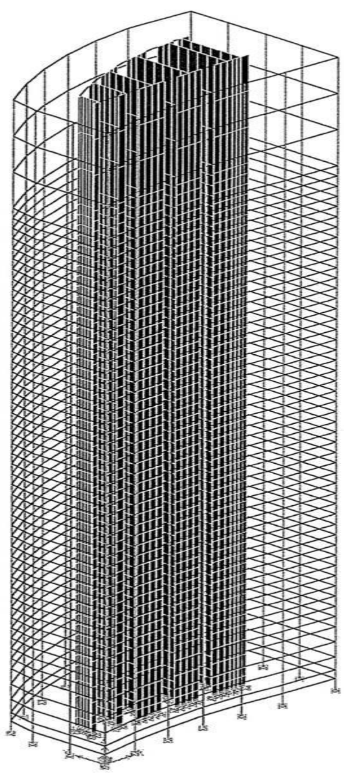
The concrete shear core walls provided most of the lateral stiffness for the structure. In the transverse direction, most of the walls were designed as I shaped walls. For efficiency, the walls that act as flanges in tension and compression for North-South loads are 1 m (3.3 ft) thick at the base. The web walls, in contrast, are just 0.5 m (1.6 ft) thick to take North-South story shear forces.

Table 2 summarizes sizes of the shear walls visible in Figures 6.

TABLE 2 - SHEAR WALLS AND COLUMN SIZES

Floor Level	CORE WALL		COLUMN	Conc. Strength
	Flange Wall (mm)	Web Wall (mm)	Size & Diameter (mm)	
B3-3F	1000	550	2000X2000 (Φ 2250)	C60
4F-8F	950	550	1550X1550 (Φ 1750)	C60
9F-17F	900	550	1550X1550	C60
18F-25F	850	525	1400X1400	C60
26F-33F	800	525	1400X1400	C50
34F-40F	800	500	1100X1100	C50
41F-TOP	750	500	1000X1000	C40

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



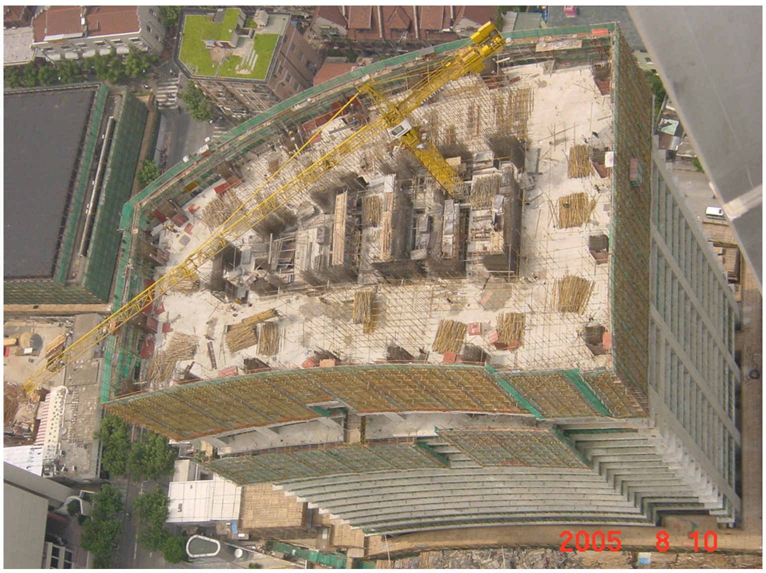


FIGURE 6 - STRUCTURAL 3D ETABS MODEL AND TOWER CONSTRUCTION VIEW

3.4 Bridge structure and connection supports

As a major feature of the project, the design architect proposed a curved pedestrian bridge spanning over 60 m (200 ft) between existing Tower 1 and newly constructed Tower 2 at floors 13 to 15. Finding an acceptable engineering solution for such a long and tightly curved bridge was a real challenge to the design team. (Figure 8)

The biggest challenge was torsion generated by eccentric bridge weight. A two-story-tall curved steel truss along both bridge elevations could span gravity loads but not torsion. Adding bracing planes at lower floor and roof levels created a complete tube. But to keep bridge cross-sections square, six unusual transverse torsional frames, mixing open and braced bays, were introduced. They tie wall, floor and roof trusses into one integrated structure. Bearings to resist both upward and downward loads take torsion out at the towers. (Figure 10)

Accommodating tower movements at the bridge was another design challenge. Different bearings allow for bridge rotations and lateral deflections as well as differential tower movements. The Tower 2 connection point has a fixed-axis steel pivot 1.5 m in diameter at floor 14 so the bridge is laterally secured but can swing when Tower 1 moves. The Tower 1 connection point, has two vertical bridge trusses embracing one tower concrete column with slide bears on the column so bridge transverse motion is restrained but in-out sliding can occur. (See Figure 9)

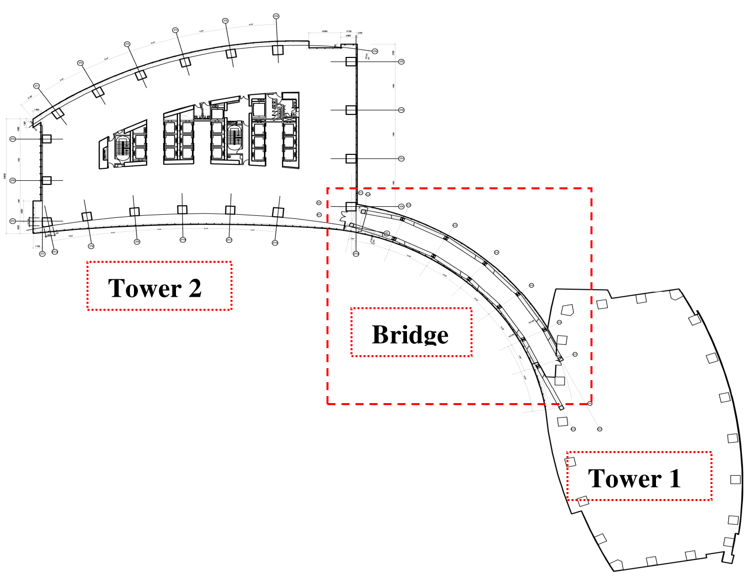


FIGURE 8 - BRIDGE PLAN AND INTERIOR IMAGE

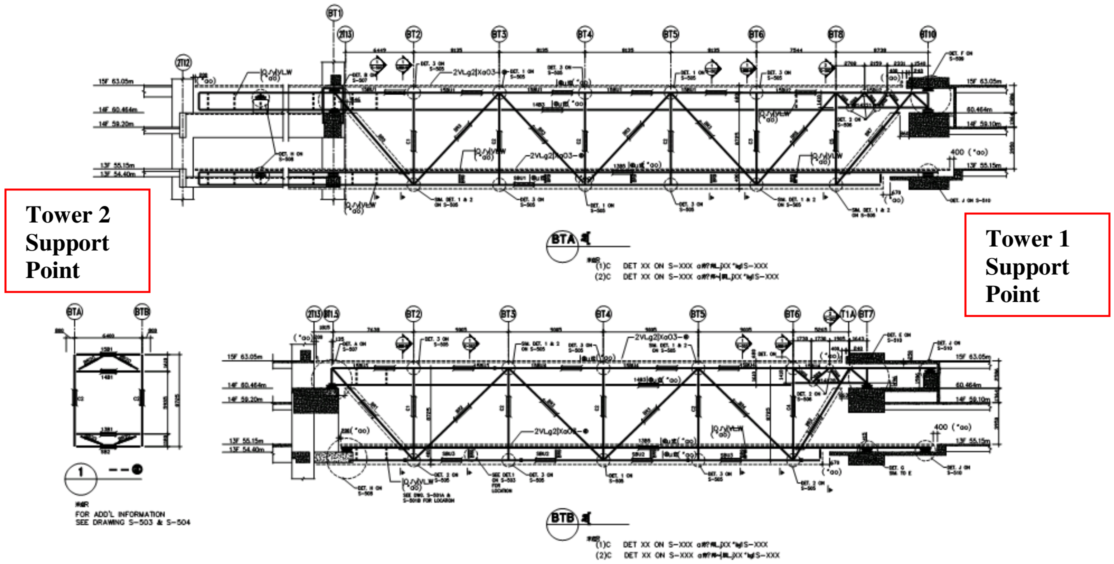


FIGURE 9 - BRIDGE TRUSS ELEVATIONS

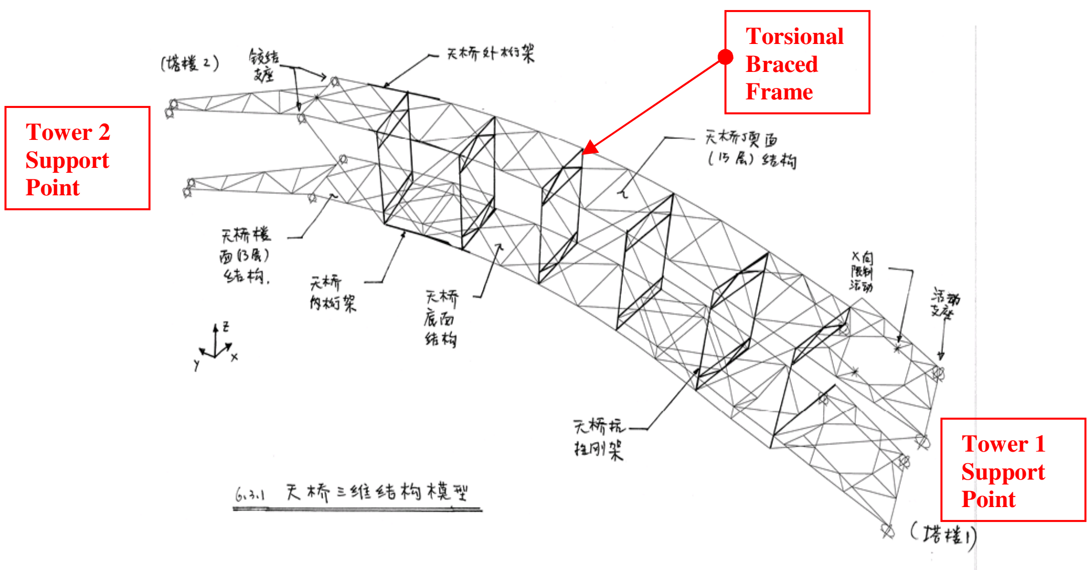


FIGURE 10 - BRIDGE 3D MODEL

3.5 Roof Top Lantern Structural System

Another major architectural feature of the tower is a 2 m thin steel truss roof lantern, sloped in two directions and standing 45 m above the main roof at elevation 179.92 m. The center of this steel structure bears on partial concrete core walls and four steel box columns. Its roof cantilevers vary from 11 m (36 ft) to 20 m (66 ft). Four steel trusses in each direction connect to the concrete core walls to resist large wind loads as well as gravity loads. (See Figure 12a, 12b, and 12c).

At 223.5 m (735 ft) above grade, the open shape of this roof lantern must resist wind from any direction. Design wind pressures acting on its top or bottom surface can be as high as

4.5 kPa (90 psf). The core walls were designed to resistance both lateral wind loads as well as torsional loads from the roof lantern. Four steel box columns are pinned to lantern framing.

Adjacent to the roof lantern a tall, thin (0.5 m wide) glass-clad vertical wall extends the tower East face. Five braced frames hide inside this vertical glass box. For economy and improved stiffness, four of the frames connect to the four steel box columns to form a composite section for flexure. All these elements bear on four major one-story deep transfer girders which also act as concrete outriggers to improve lateral stiffness of the main structure.

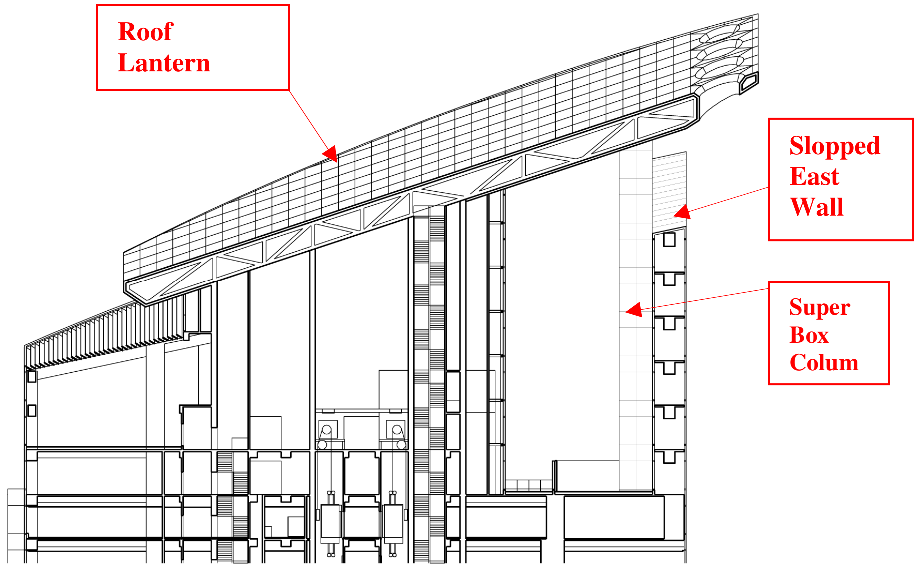


FIGURE 12a – ROOF LANTERN AND EAST WALL ELEVATIONS

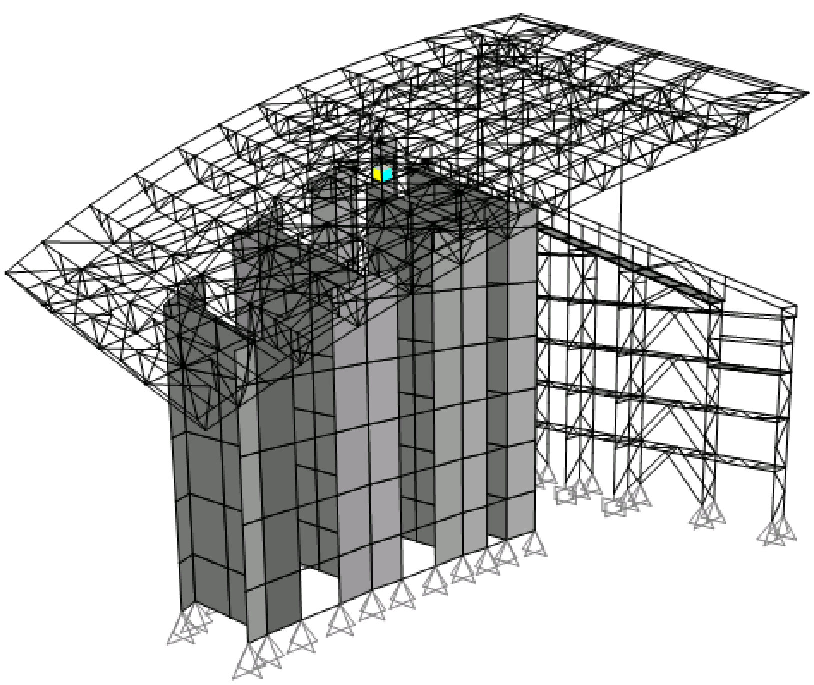


FIGURE 12b – ROOF TRUSS 3D SAP200 MODEL

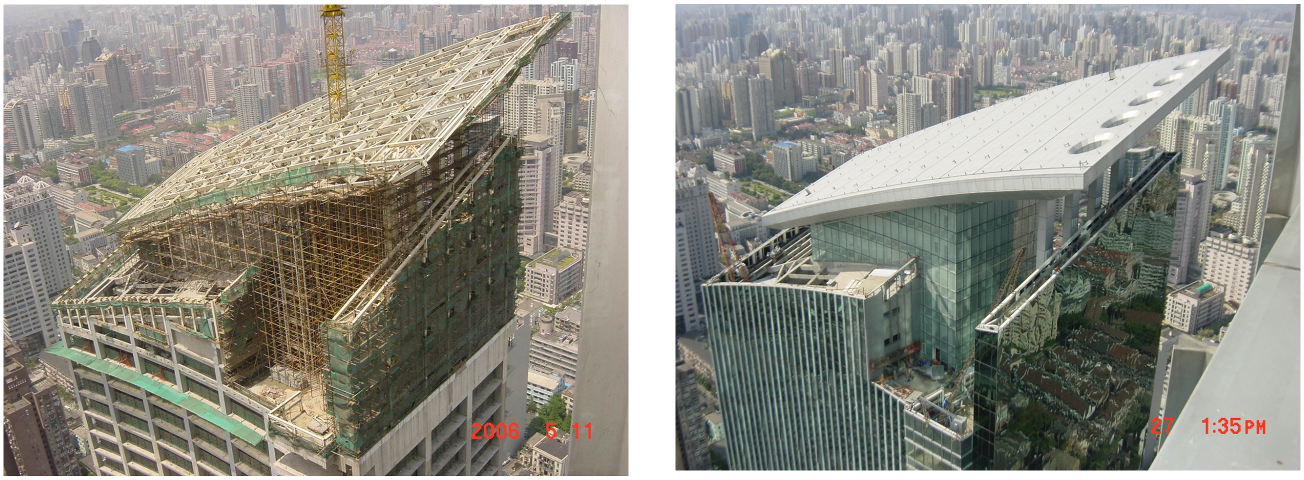


FIGURE 12b – ROOF TRUSS CONSTRUCTION PHOTO

4.0 CONCLUSION

Plaza 66 Tower 2 provides an excellent example of high rise design and construction under challenging conditions, including poor soil conditions, innovative architectural features 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 Shanghai.

5.0 CREDITS

The authors would like to thank Len Joseph, Paul Lew and Paul Fu for their work during design and would like to thank Alan Yu, Michel Jiang for their work during design and construction, especially to Len Joseph for his review and comments.

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