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The Structural Design of "China Zun" Tower, Beijing

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Abstract

The "China Zun" tower in Beijing will rise to 528 meters in height and will be the tallest building in Beijing once built. Inspired by an ancient Chinese vessel, the "Zun", the plan dimensions reduce gradually from the bottom of the tower to the waist and then expand again as it rises to form an aesthetically beautiful and unique geometry. To satisfy the structural requirement for seismic and wind resistance, the structure is a dual system composed of a perimeter mega structure made of composite mega columns, mega braces, and belt trusses, and a reinforced-concrete core with steel plate-embedded walls. Advanced parametric design technology is applied to find the most efficient outer-perimeter structure system. The seismic design basically follows a mixed empirical and performance-based methodology that was verified by a shaking table test and other specimen lab tests. The tower is now half-way through its construction.

Keywords: "China Zun" tower, Super high-rise building, Mega frame, mega column, Composite shear wall, Concrete-filled steel tube, parametric design

1. The Background

Compared with the frequently refreshed building height records in other China cities, Beijing's skyline is rather stable. It is partially because of the strict height limit in the central Beijing to protect the view of the Forbidden City area, and also due to the extra difficulty due to the high seismic fortification intensity set for Beijing (PGA= 0.2 g for 475-year return period) which is higher than any other major China cities. The 330 m tall China World Trade Center held the record of height since 2008.

In 2010 the plan of the CBD Core Area was initiated including 17 towers with height ranging from 50 m to 500 m. Located east to the East Third Ring road, it immediately raises the skyline of east Beijing by about 200 m. The "China Zun" tower is the topmost building among them, containing 350,000 m^2 GFA above ground and reaching 528 m high with 108 floors. Developed by CITIC Heye Investment Co. Ltd, it will become the tallest building in the similar high seismic zone around the world, compared with cities like Taipei and San Francisco.

The building's shape is inspired from an ancient Chinese vessel "Zun", which is used to hold wine during religious ceremonies. The dimension of the corner-rounded square floor plan is reduced from 78 m at bottom to 54 m at 3xxm high and then enlarged to 70 m at top. This is different from most other super high-rise buildings that have smallest dimension at top to reduce the wind load and seismic mass. On the other hand, it brought more value to the client by elevating more floor area to higher levels considering the tower is surrounded by a cluster of buildings 200~400 m tall in the same area.

The structural height 522 m (to the main roof) and the structural width at ground floor 72.7 m (measured from the outer surface of mega column at ground floor) denoted an aspect ratio of 7.2, a reasonable value to ensure an efficient lateral resisting system.

Figure 1. Rendering elevation view and the size of "Zun" (© Kohn Pedersen Fox Associates).

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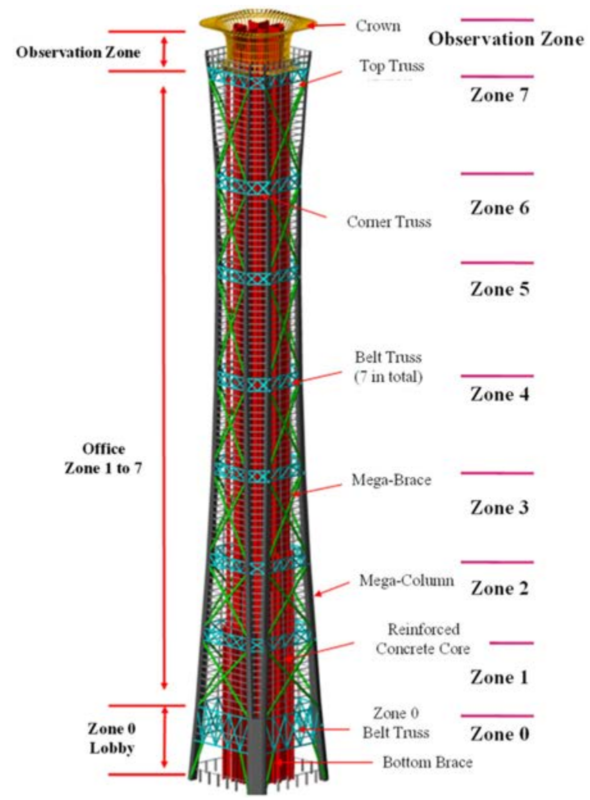


Figure 2. The final scheme.

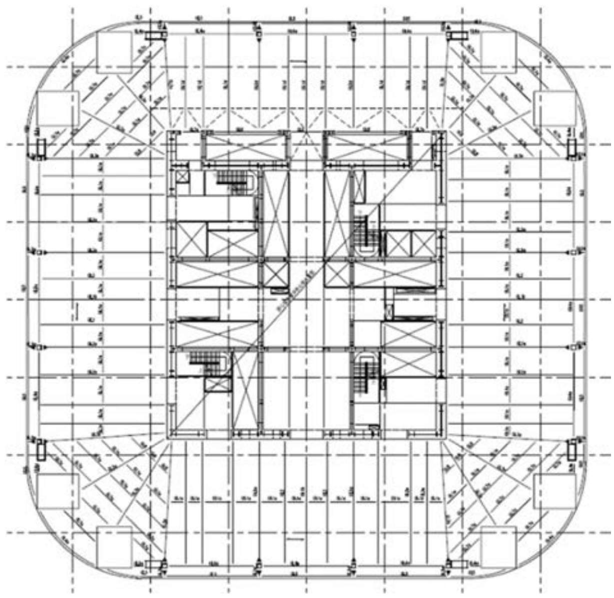


Figure 3. Typical floor plan.

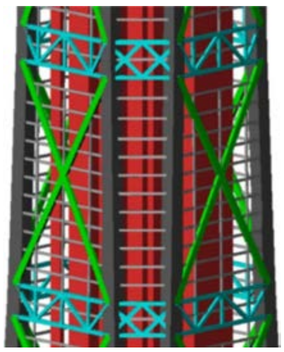


Figure 4. The mega structure and the sub-frame.

2. Structural System

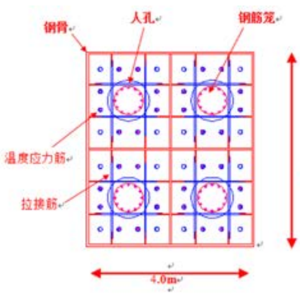
For tall buildings which are used as office, the structural members are normally located at the central core and the perimeter to leave the usable area between them column free. In many of these structures the core provides the majority of the lateral stiffness, supplemented by the perimeter structure with or without the leverage of outriggers at intermediate floors. The examples are the International Financial Centre and the International Commerce Centre in Hong Kong, and Shanghai Centre in Shanghai. For the China Zun tower, it was found at early study that the above strategy is unable to provide sufficient lateral stiffness as needed. Thus the perimeter structure should act as the primary structure that, together with the central core, constituted a dual lateral resisting system.[1]

2.1. The Perimeter Mega Frame

A truss-like mega frame was finally chosen for the perimeter, composed of mega columns, mega braces and transfer trusses.[2] The mechanical/refuge floors split the floors of the tower into 9 zones, to which the segments of the mega frame were consistent (Fig. 2). For most zones eight mega columns are located at corners of the plan are concrete-filled tube (CFT) sections (Fig. 3) while at zone 0 two columns at one corner are merged into one. The steel cross braces at each side are maintained in one plan for each zone although the tower profile was curved. In each zone there are sub-frames which take down the gravity of the specific zone only. The gravity is then conveyed to the mega columns by the one-story high transfer truss (Fig. 4). The tension force in the mega columns to resist the lateral overturning moment is thus balanced. The cross-section of the columns and beams in the sub-frames can be kept small.

The geometry of the mega frame had been carefully studied with the architect Kohn Pedersen Fox Associates using advanced parametric structural design approach and BIM technology. The floor dimensions at ground, top and the “neck”, the height of the “neck”, the tower profile curvature and corner round-up radius are the critical parameters controlling the architectural functions as well as structural performance.

The central line of mega columns, being straight in each zone, defines where the mege frame is laid. Several factors



a). Cross section at zone 1 (20m2)

b) Cross section at zone 0 (60m2)

c) Under construction

Figure 5. The mega columns.

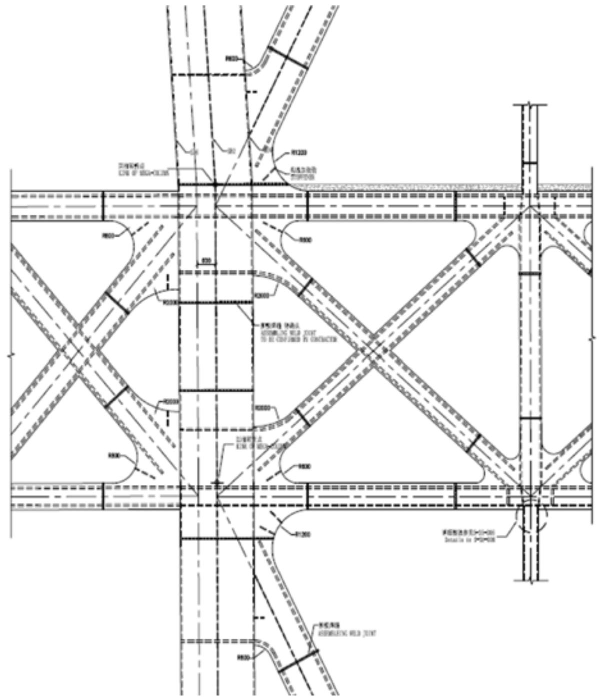


Figure 6. The major connection of the perimeter mega frame.

were considered when determining the final setting-out: i) The varying distances between the mega frame and the facade on every floor to be minimized; ii) the lever arm of the push-pull action of the mega column and the stiffness of the mega brace defined by its angle; iii) whether or not to introduce two kink points at the top and bottom level of the transfer truss, which increase the overall lateral stiffness but causes higher shear force flow between core and perimeter. Numerous options have been analysed thanks to the automated design process with parametric modeling.[3] The final setting-out of the column line achieved a minimum structure-facade distance for most floors, and a satisfied structural behaviour.

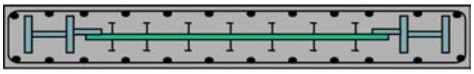
Mega columns adopt concrete-filled steel tube section, being rectangular section in most zones (Fig. 5a). The steel plate at the outmost of the section eases the connection with transfer trusses and mega braces and eliminates the necessity of formwork for the concreting. At the zone 0 the mega column is a huge polygon profile combined by a few CFT sections (Fig. 5b & 5c). The in-filled concrete grade is C70~C50. To improve the ductility and minimize the adverse effect of creep and shrinkage of the concrete, steel reinforcement cages are distributed through the man-holes in the section. Vertical stiffeners are arranged symmetrically on the inner face of the steel plates, and tied with rebars to restrain out-of-plane buckling of the steel plates.[2,4]

The members of mega braces and transfer truss are all steel box sections welded with the mega columns (Fig 6).

2.2. Core

The core extends from the top of the pile cap up to the roof of the mega tower. It is square in plan and situates at the centre of the building. The base plan is nearly 39 m × 39 m, which is reduced to a smaller square at 34th floor with a symmetric layout. Double lintel beam is used extensively at core perimeter which accommodate the service ducts at a higher level and increase the clear headroom of the outside-core corridor. It introduces more ductile mechanism to the overall structure.

The core walls use C60 grade concrete and are embedded with steel sections. At the low zone, alongside steel sections, steel plates are embedded to form composite shear walls (CSW) (Fig. 7). It is a perfect match: the steel plate, free from buckling with restraint from concrete, increases the shear strength and axial capacity significantly; the concrete part contributes to the stiffness and provide inherent fire resistance to the steel plates. Shear studs are


a) The schematic cross section


b) The steel plates of the core before rebar fixing and concreting

Figure 7. The Composite shear wall.

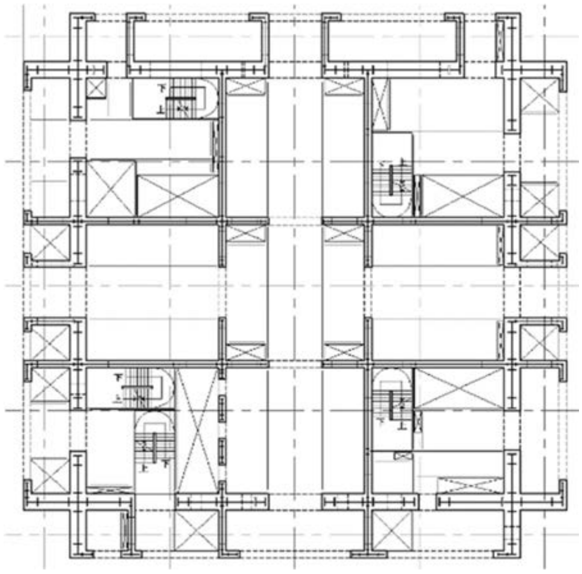


Figure 8. The core plan at lower floors.

welded on the steel plate to enable the composite action.

The core wall thickness gradually reduces from 1200 mm thick at the base of tower to 400 mm at the top, while the wall inside the core reduces from 500 mm to 400 mm gradually. The steel plate inside CSW is 60 mm thick single plate at the bottom and reduced to 30 mm thick at 41st floor. For the top zone of the core, where the perimeter structure stops, steel plate is also set inside the main walls after elasto-plastic analysis finds excessive damage occurs under severe earthquake event.

Steel stanchions are set at the corners or ends of the walls for almost all stories to resist possible tension force and also improve the ductility. For the "neck" floors, where the stiffness of the perimeter structure is reduced due to a smaller lever arm for lateral resistance as well as a worse mega bracing angle, the core is subject to larger demand of capacity and demonstrates more damage in elasto-plastic analysis. Additional steel embedded braces are set between the steel hidden columns inside the perimeter walls so as to enhance the performance.

2.3. Floor system

Composite decking is generally adopted. Typical slab thickness is 120 mm supported by steel beams of 3 m spacing. To ensure reliable transfer of shear forces between the core and external frame, and the main structure and the basements, the slab thickness is 200 mm on strengthened floors (the floor of top and bottom chord of the transfer truss). The transfer truss creates a sudden change in story stiffness and thus results in enormous shear transfer between the perimeter and core. The floors are designed with solid concrete slab of grade C40 concrete with top-and-bottom two-way reinforcement, and further strengthened with horizontal steel struts underneath the slabs.

2.4. Foundation and Basement

Due to the limited site area, the basement has 6 stories (partially 7 stories), deeply down to 38-meter below ground. Underneath the tower footprint a 6.5 m thick raft of C50 concrete supports the super structure, and in turn is supported by bored piles of 37 m long, founded on Level 12 layer with cobble and round gravel at about 75 m below ground. The piles are 1.0/1.2 meter in diameter, and the characteristic capacity value of single pile is 1450/1600 tons with grouting at pile shaft and toe. To distribute the huge gravity load of the mega columns, fin walls are attached to the mega column on two directions coordinated with architecture layout.

3. The Analysis

3.1. Elastic Analysis

Several software are used in the analysis including ETABS, MIDAS, and YJK. Arup used LS-DYNA for the elasto-plastic analysis to evaluate the building behaviour under severe earthquake.

In the computer model, the composite shear wall and the mega column are modelled with homogeneous material with the equivalent sectional properties; the mega diagonal brace and transfer truss are connected with the mega column through rigid arms simulating the corresponding size and eccentricity effects.

The total seismic mass for the tower (including B1/F) is 658,000 ton, equivalent to 1.83 ton per meter square of floor area. The first three modal periods of the structure are 7.30 s, 7.27 s and 2.99 s, vertical vibration modal 0.60 s. The first two modes are translational modes with a torsion/translation period ratio of 0.41.

Seismic force is obviously the dominant load case. According to the requirement of the Expert Review Panel (EPR), the analysed base shear shall be at least 2% of the

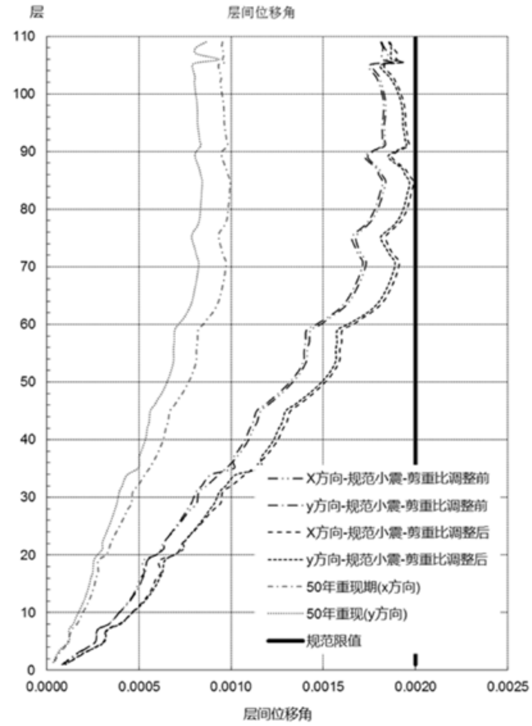


Figure 9. Inter-story drift by earthquake and wind.

total seismic mass which should be increased to 2.4% for drift calculation and strength checking. The maximum bottom shear force in 100-year return period wind tunnel test is 59MN.

The maximum inter-story drift for 50-yr return period wind load and frequently occurred earthquake are 1/999 and 1/513 respectively (Fig. 9). The latter has been increased according to minimum base shear. The top displacement under 50-year wind load and frequently occurred earthquake are 330 mm and 570 mm respectively.

A group of seven sets of earthquake acceleration records have been studied in the static dynamic time history analysis. The results are compared with those from the response spectrum analysis (RSA). The seismic forces are correspondingly increased in some top floors to reflect the contribution of the higher modes which are underestimated in the RSA.

The perimeter structure carries 40~50% of the relevant story shear force caused by earthquake. However the story shear taken by the perimeter structure shall still satisfy 20% base shear requirement. About 67% of the total overturning moment is taken by the perimeter structure.

The perimeter structure takes half of the story shear and the majority of the overturning moment (Fig. 10). The demand to the core is thus reduced except the “neck” floors which is discussed in 4.2.

3.2. Wind Tunnel Tests

Wind tunnel test was carried out in Rowan Williams Davies and Irwin Inc (RWDI) in Canada and third-party checked by China Academy of Building Research. The model is 1:500. The test considered two different proximity scenarios with or without the other tall buildings proposed to build near the tower (Fig. 11).

The test result shows that the acceleration of ground motion is 0.1 m/s^2 under 1.5% damping rate in 10-years return period, which well satisfies the Chinese code (Fig. 12).

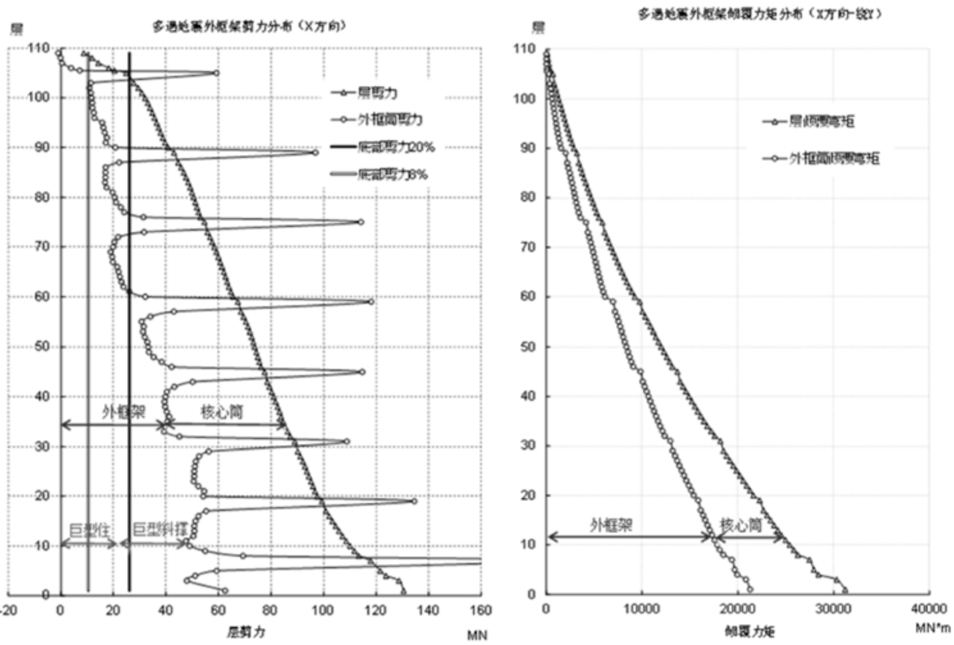


Figure 10. Shear force and overturning moment distribution between perimeter and core.

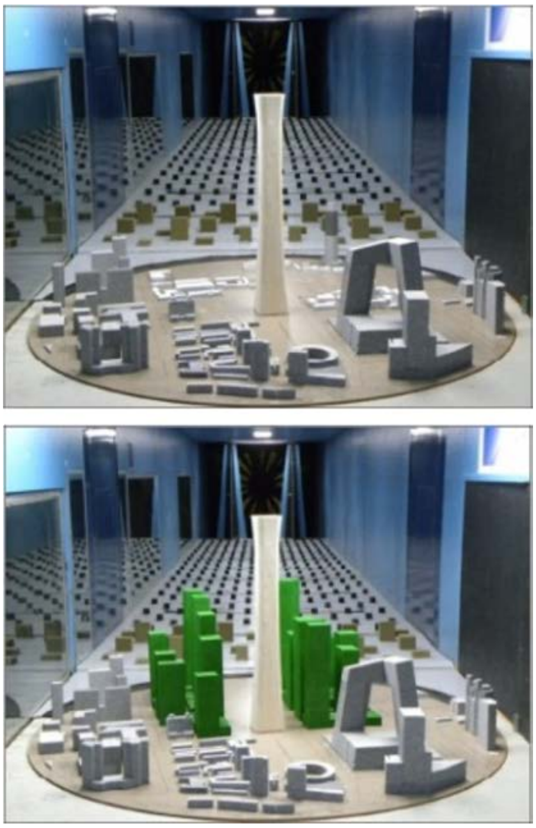


Figure 11. Different proximity scenarios in wind tunnel tests (© RWDI).

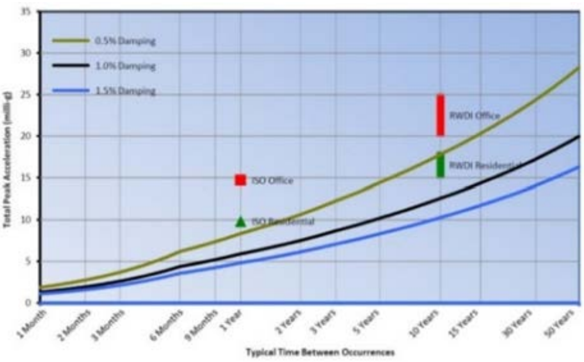


Figure 12. Maximum acceleration of ground motion of structure in different damping ratio and return period (© RWDI).

3.3. Elasto-plastic Time-History Analysis for Severe Earthquake Event

To achieve the seismic performance objective of no collapse under a severe earthquake, the design adopts the member plasticity development limits and analysis methods and procedures suggested in ASCE41-06 5. Material and tensile and compressive strain data of member are used for

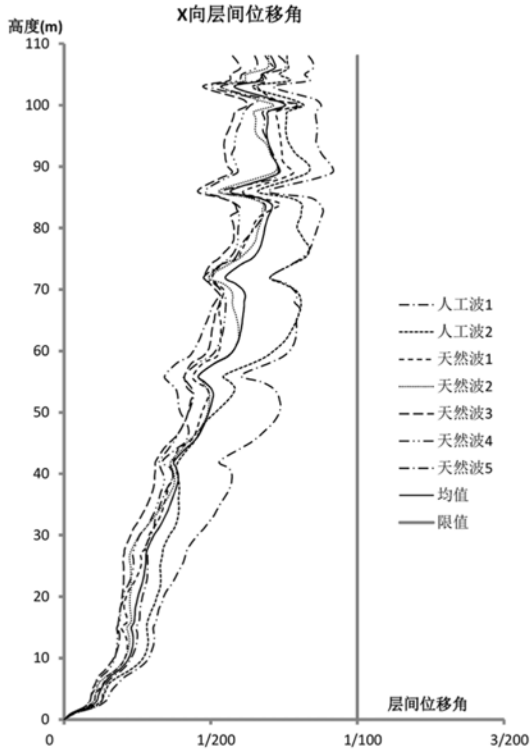


Figure 13. Story drift in elastic plastic analysis under severe earthquake.

overall evaluation on shear wall. The non-linear seismic analysis is carried out with the general non-linear dynamic finite element analysis software LS-DYNA considering geometric non-linearity as well as material non-linearity.

The fiber model is used to calculate section capacity, bending moment curvature and deformation limit of beam and column. For the walls, beam elements are used to model the boundary elements and nonlinear shell elements model the wall panels. For the composite shear walls, steel and concrete shell elements are built at the same position.

Five sets of natural records and two sets of artificial records are used. The overall performance is evaluated by the elasto-plastic story drift, shear-weight ratio, top story drift, bottom shear time-history curve, plastic time-history development and plastic zone. The member's plastic deformation is checked against the limit value. Fig. 13 showed lateral drift curves of 7 records in elastic plastic dynamic analysis under severe earthquake. The story drifts in X direction all satisfy the code limit 1/100.

3.4. Robustness Analysis

For the code-required robustness analysis, a specific member in sub-frame or transfer truss member is removed in the model and the elastic static analysis is carried out to check the capacity of the remaining members. For the mega columns, due to the low probability of whole member failure, an 80 kN/m^2 is applied for the checking.

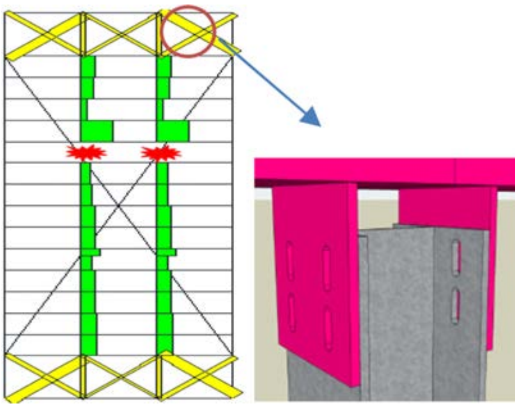


Figure 14. The second load path of sub-frame in extreme event.

Figure 15. Shaking table test.

For the extreme event shown in Fig. 14, the sub-frame and the transfer truss above are connected through a sliding connection which does not take force in normal situation. When a small column completely destroyed, this connection allows structural floor hung on the upper truss, and the second loading path is formed.

4. Other Topics

4.1. RSA for structure with long modal period

For super high-rise buildings which are tall and slender, RSA using acceleration spectrum generally gives low base shear force because the spectral value drops when the period is very long. In the practice in China, the base shear obtained by the analysis shall satisfy a minimum value which is intended to prevent excessive damage due to unexpected structural deformation.

4.2. The “neck”

As expected the “neck” together with the enlarged top portion brings some problems in the structural design. The seismic mass is moderately increased at top rather than decreased like most other tall buildings. However the major adverse effect is found to be the exaggerated capacity requirement on the core at “neck” floors because of the dual effect of a smaller thus weaker perimeter structure and the negative shear force caused by the inwards tilting of the perimeter columns. As it is inappropriate to thicken the core wall anymore, steel diagonals are added to the core wall forming an embeded steel skeleton together with the steel stanchions. This strengthening is justified by the elasto-plastic nonlinear analysis to be appropriate. A very strong strengthening design like composite steel plate shear wall might attract more seismic force and create new weak area.

4.3. The choice of the material

It might have been a very arguable choice not using pure steel structure for such a tall building on a high seismic zone 20 years ago. The China World Trade Centre, 330 m tall and also designed by Arup 10 years ago, adopted a “highly composite” solution in which composite columns and walls were adopted and all other members used steel sections.[6] Subject to the environment of authority approval and the development of tall building design and construction, a concrete-based core strengthened by steel plates or steel “skeletons” is now justified safe and acceptable to the industry. The nature of the composite construction used in this tower demonstrates a state-of-xxx balance of safety, structural cost and construction speed.

4.4. Shaking table test

A 1: 40 physical shaking table test was carried out on December 2013 at China Academy of Building Research to test the structural performance under time history records corresponding to the three levels of earthquakes (Fig. 15). The major results are consistent with the elsto-plastic analysis and justifies the structural safety as designed.

4.5. Site Progress

The construction of the China Zun tower has been progressing fast. The formal kick-off ceremony was held at June 2013. At April 2014 all piling works were finished and the pile cap was concreted. Since February 2015 the construction of above ground structure was commenced led by the erection of steel members inside the core. On Apr 2016, the tower has reached 200 m (Fig. 16).

Figure 16. The site at April 2016.

5. Summary

The construction of the 528 m tall China Zun Tower is ongoing smoothly. It will mark a new height record in similar high seismic zone and presents the maturity of the design and construction of composite structure in tall buildings. As a code-exceeding structure in China because of its height, it underwent rigorous review procedure by the Expert Review Panel procedure to justify the safety and validity. The seismic design basically follows a hybrid empirical and performance-based methodology[7,8] which was verified by the shaking table test and other specimen lab tests.

Acknowledgements

Arup is the structural engineer responsible for the schematic design and preliminary design for this project. Beijing Institute of Architecture and Design is responsible for the detailed design stage. East China Architectural Design & Research Institute is the project consulting company. CABR Technology Co., LTD conducted the independent third party elasto-plastic analysis, the shaking table test and independent wind tunnel test audit. This project was approved by National Review Panel for Seismic Design of Code-exceeding High-rising Buildings in February 2013.

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