Council on Tall Buildings and Urban Habitat Korea

International Journal of
High-Rise Buildings
www.ctbuh-korea.org/ijhrb/index.php

International Journal of High-Rise Buildings
September 2022, Vol 11, No 3, 189-196
https://doi.org/10.21022/IJHRB.2022.11.3.189

Several Issues Closely Related to Construction in the Structural Design of Wuhan Center

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Abstract The practical difficulties of construction will impose many restrictions on the structural design, and the construction method can also provide unexpected ideas for solving design problems. Through the discussion of three issues closely related to construction in the structural design of Wuhan Center, this paper illustrates the importance of in-depth consideration of the construction situations in the structural design stage. The topics of "Connection between Embedded Steel Plates in Steel Plate Composite Shear Wall" and "Connection Joint between Outrigger Truss and Core Wall" are about how to facilitate on-site construction by simplifying and optimizing detail design. The topic of "Adjusting Internal Force Distribution by Optimizing Construction Sequence" is about how to make the construction process a tool for structural design.

Keywords Super High-Rise, Frame-Core-Outrigger System, Steel Plate Composite Shear Wall, Outrigger Joint; Construction Sequence

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1. Project overview

Wuhan Center is located in Hankou District, Wuhan City. It is the first super high-rise building in the Wangjiadun Central Business District built on the original site of an airport. The total height of the building is 438m, with 88 floors above ground and 4 levels of basement below ground. (Figure 1). Building functions include offices, apartments, hotels, and sightseeing (Figure 2). The super-structure of the project was capped in 2015, and the facade construction was completed in 2017. Wuhan Center has won the Best Tall Building 400 meters and above in 2021 Award of Excellence of CTBUH. In the structural design of Wuhan Center, several issues are closely related to its construction.

2. Design Criteria

The design service life of the tower is 50 years, and the durability of the main components is 100 years. Building seismic fortification is classified as class B. The design seismic intensity of Wuhan City is 6 degree, with the basic design seismic acceleration of 0.05 g. Based on the importance of the project, the site seismic safety evaluation was carried out and the basic design seismic acceleration was adjusted to 0.08 g, which is close to the value of the fortification intensity of 7 degree and the maximum earthquake influence coefficient α_{max} under frequent earthquakes is 0.075. The basic wind pressure value based on the 50-year return period is 0.35 kPa.

3. Foundation and basement design

The basement of the tower is buried at a depth of about 22 m with a pile-raft foundation (Figure 3). Two types of 1000mm diameter with 43 m~46 m long rotary excavated bored cast-in-place piles with the bearing capacity of 13000 kN (type A inside core wall area) and 12000 kN (type B outside core wall area) were used. The soil at the pile end and the pile side is reinforced by post-grouting. The pile ends are supported on the slightly weathered mudstone layer. The static load test results indicated that the pile side reaction force and the pile end reaction force account for about 72% and 28%, respectively.

The thickness of the raft is 4 m, and the concrete strength is grade C40. Since the piles have covered the entire range of the raft, the bending moment and shear force in the peripheral area of the core wall are very high. The stiffness of the raft is strengthened by 8 wing walls starting from top of basement to top of raft foundation. Therefore, the pile reaction has been improved to be more uniform by adjusting the load path. The wing walls also helped increase the lateral stiffness of the basement and enhance the embedding of the tower.

4. The Superstructure System

The roof elevation of Wuhan Central Tower is 393.9 m, and the core wall continues to rise to 410 m. The crown starts from roof level to the highest point of 438 m. The tower crown, being the key part that shows the top image of Wuhan Center, was composed of four cantilever trusses extending from core wall, and four sets of lattice columns stand on the roof, and the ring truss connecting cantilever trusses and lattice columns. Roofing system, curtain wall and window cleaning machinery are all supported by the tower crown structure (Figure 4).

The lateral force resistance system of the tower is a Frame-Core wall-Outrigger truss system (Fig. 5~6), which consists of a square reinforced concrete core wall, a frame formed by giant CFST columns and steel beams with 5 ring belt truss reinforcements, 3 outrigger trusses connecting the core wall and 8 giant columns.

The concrete strength used was grade C60 for the core walls, and grade C50~C70 for the CFST columns. The steel strength used was grade Q345B for both steel beams and CFST columns, and Q390GJC for the outriggers, the belt trusses, and the embedded steel plates of the shear wall.

5. Several Designs Based on Construction Considerations

5.1 Connection between Embedded Steel Plates in Steel Plate Composite Shear Wall

The shape of the core wall is square in plan, with the four corners chamfered by about 2.8 m from 64th floors, and the exterior walls retreats inward by 500 mm from 78th floor. The changing of the outline is accomplished by tilting and overlapping, respectively (Fig. 5).

In order to minimize the wall thickness of the bottom floors, the exterior walls with a thickness of 1.2 m and the interior walls with a thickness of 0.55 m are embedded with single-layer steel plate to form composite shear walls. The design code requires the axial compression Demand-Capacity Ratio of the walls in the bottom reinforcement floors to be less than 0.5 so as to assure the ductility of the shear wall. The steel plate is placed for core walls below the 12th floor (Figure 5), and the thickness of them are 30 mm and 20 mm. The four corners are embedded with steel columns along full height of the core wall.

Considering the transportation conditions and on-site hoisting capacity, the maximum size of each embedded steel plate produced in the workshop is 3.5 m × 12 m. There are two options for on-site connection between steel plates: welded or bolted connection. The bearing capacity of welded connection is higher than high-strength bolted connection, whereas the construction is more complicated in comparison. Since the primary role of the steel plate here is to bear the axial force so as to reduce the axial compression Demand-Capacity Ratio of the wall, the connection between the upper and lower steel plates needs to ensure to be equal in strength. For a 30 mm - thick Q390 steel plate, the number of high-strength bolts required for an equal-strength connection is about 5M24@100 on both sides. Steel plates at the corner of the core walls are even thicker and requires more bolts for connecting. The excessive number of bolts brings difficulties to the

Figure 1. Panorama.

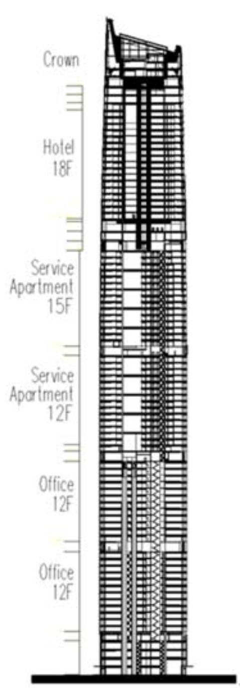


Figure 2. Function zoning.

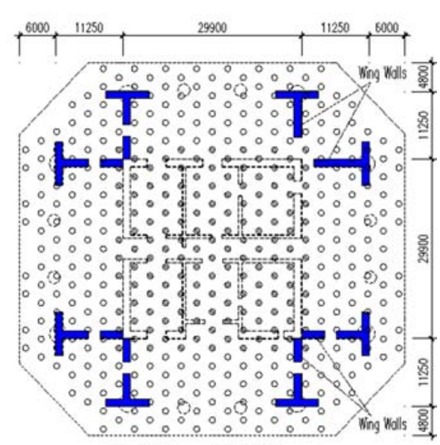


Figure 3. Pile-raft foundation.

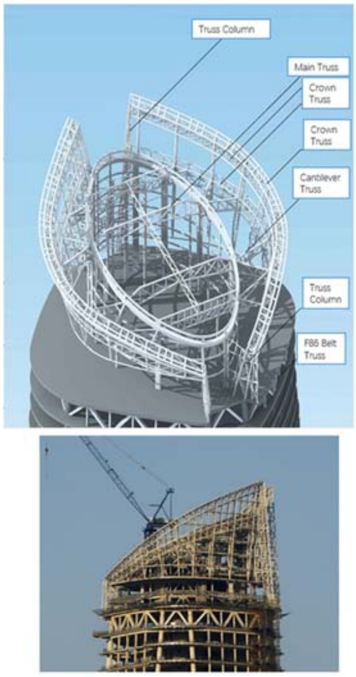


Figure 4. Tower crown.

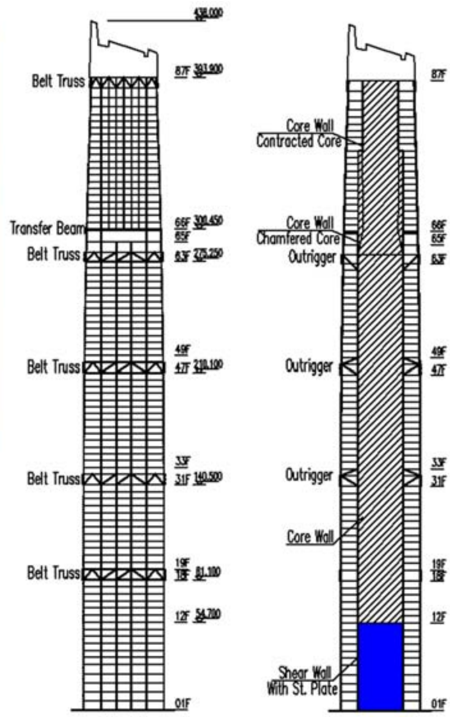


Figure 5. Lateral resistance system.

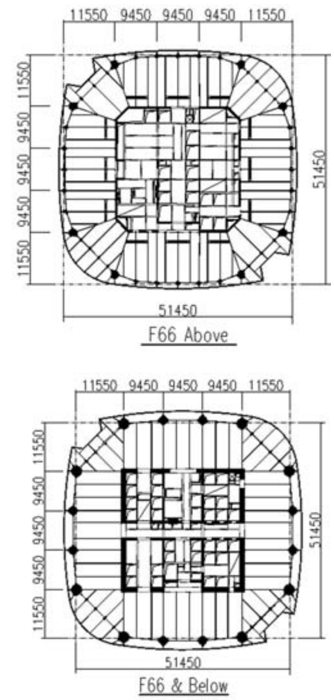


Figure 6. Structural layout.

arrangement of bolts, the requirements for construction accuracy are also sharply increased, along with significant increase in cost. Compared with welding, the original advantages of bolts no longer exist, so welding is used for the connection between the upper and lower plates. Since the upper steel plate is always in a vertical unconstrained state during the welding construction process, the deformation caused by welding has little effect on its installation.

The connection between adjacent steel plates has no obvious force transmission requirement under the action of axial force, which makes bolt connection possible. But the existence of steel plates also plays a very direct role in improving the shear resistance of the shear wall. Therefore, it is necessary to evaluate the influence of the bolt connection on the shear force resistance.

We performed a finite element analysis of embedded steel plates with bolted connections. Fig. 7 is the layout of the embedded steel plate of the wall W-V1 on the ground floor, and Fig. 8 is the stress distribution diagram of the steel plate and the shear force of the plate

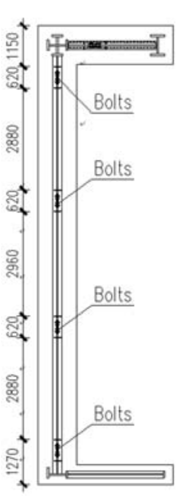


Figure 7. Layout of the embedded steel plate.

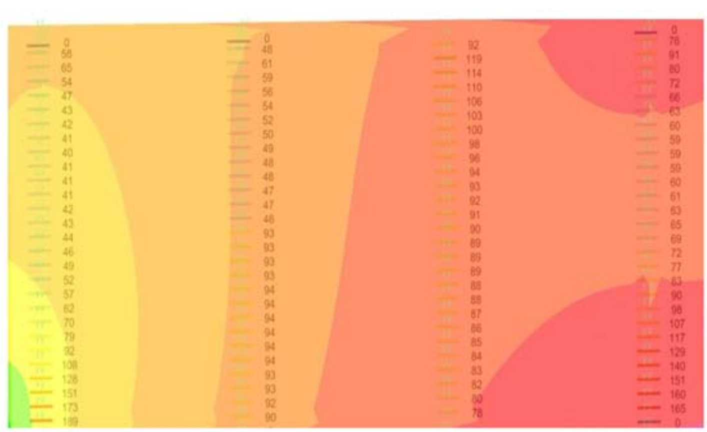


Figure 8. Stress distribution.

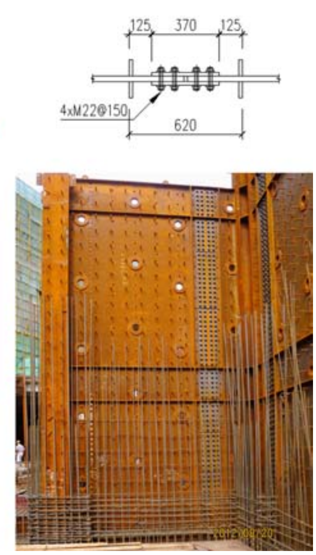


Figure 9. Bolt connection.

connection under the action of fortification earthquake. It can be indicated that each 150 mm connection unit bears a maximum shear force of 189 kN. As long as the strength of the bolt connection used exceeds this value, the overall steel plate can be guaranteed to be in an elastic working state, which is the same as the effect of an all-welded connection. Therefore, a relatively weak high-strength bolt connection was used between adjacent steel plates, and the bolt density is 2M22@150, which greatly simplifies the construction on site. (Figure 9)

Shear studs are set on the surface of the embedded steel plate to ensure the co-deformation with the outer concrete. At the same time, holes are set in the part of the steel plate to allow the stirrups to pass through so as to tie the two sides of the concrete separated by the steel plate into one.

5.2 Connection Joint between Outrigger Truss and Core Wall

There are 3 outrigger trusses arranged in this tower, each of which is two stories high, located on floor 31st to 32nd, 47th to 48th and 63rd to 64th, respectively. Belt trusses are arranged on the floors where the outriggers are located and on the 18th and 86th floors, one story height each. The form of the outrigger truss comprehensively considers its intersection with the belt truss and the needs of the aisle layout. The lower two outriggers are arranged in a "K" shape, and the upper one is a single inclined member (Figure 10).

The outrigger trusses in indicated directions meet at one point at the corner of the core wall, where the stress is concentrated, and the connection structure is complex. In order to ensure the construction reliability of the connection, research and experiments were carried out.

The section of the three outrigger chord members are 1000 × 800 × 25 × 100, 1000 × 700 × 25 × 800, 1000 × 630 × 30 × 60 ($HxBxt_wxt_f$), and the thickness of the wall connecting with the core wall are 1000 mm × 700 mm and 600 mm, respectively. A commonly used outrigger-to-wall connection is to embed the chord into the wall. This method is very effective when the wall is thick enough to wrap the chord member. The outrigger trusses on the 31st and 47th floors have adopted this joint, by narrowing the 800 mm wide outrigger chord members to 400 mm near the core wall, and then buried it into the wall. Solid forged steel members are used where the two orthogonal outriggers meet. The flange plate extends in the wall from one end to the other, and the two flange plates are connected by vertical connecting plates at 500 mm interval (Figure 11).

For the outrigger located on the 63rd floor, since the thickness of the core wall is only 600 mm, it is difficult for the concrete to effectively wrap the intersecting outrigger members at the corner. At the same time, it is very difficult to bind steel bars and pour concrete at this corner. Therefore, it is difficult to guarantee the construction quality of the connection. In order to ensure the reliability of construction, we tried a new connection joint: adjusting

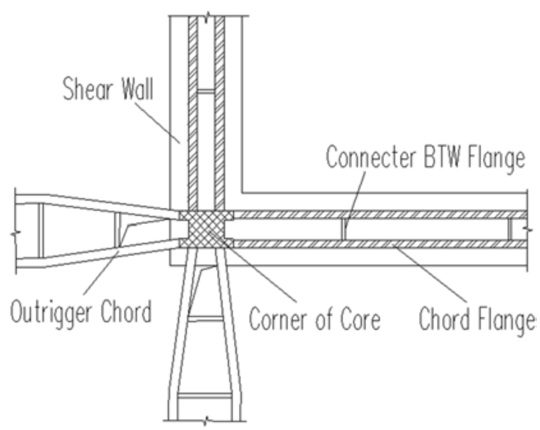
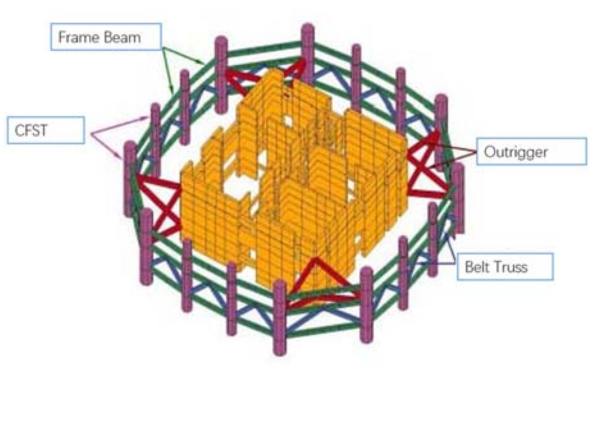
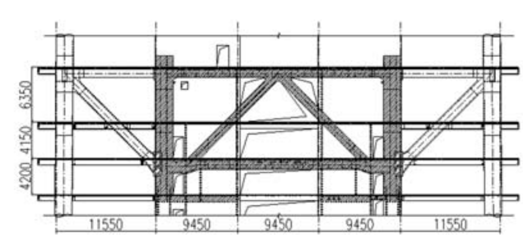
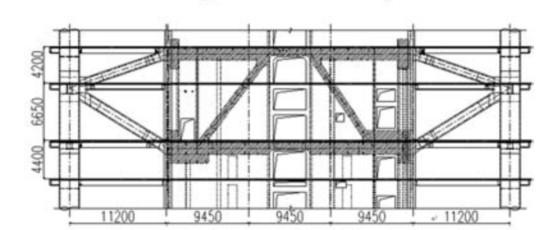


Figure 11. Steel plate embedded type.





Single inclined shape



K shape

Figure 10. Arrangement of outrigger truss and ring belt truss.

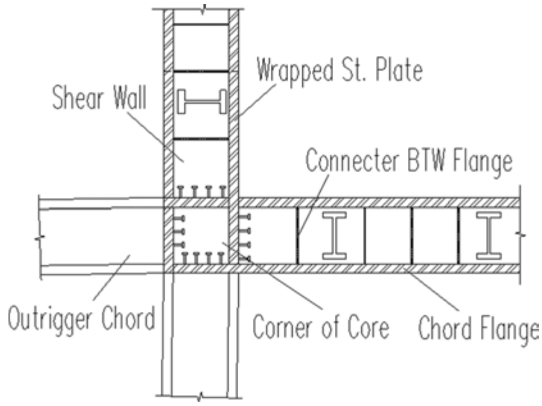


Figure 12. Steel plate out-wrapping type.

the width of the outrigger chord to match the wall thickness, directly wrapping the flange plate of the outrigger on the outside of the wall, and arranging the vertical rib plates between the two flange plates.

The corners of the core wall on the two floors of the outrigger height range are also covered with steel plates (Figure 12). Therefore, by using the new connection joint, space for tying steel bars can be increased, concrete pouring can be more convenient and construction quality can be easily guaranteed.

In cooperation with Tsinghua University, the design team conducted a scaled test study on these two different connection joints. For the convenience of comparison, the outrigger on the 31st floor was selected as the prototype for the test, and two different connection types were used, which were called Steel Plate Embedded Type and Steel Plate Out-wrapping Type.

The outrigger test member is bounded by the core wall shear wall and the CFST column. The loading method is to apply a reciprocating load on the top of the CFST column. According to the loading capacity of the laboratory equipment and the conditions of the test site, the scale ratio of the test component is determined to be 1:8, and the total length, width and the height of the test component is 3.150 m × 3.150 m and 2.475 m (Figure 13).The axial force of the shear wall is applied by prestressing, and the axial compression Demand-Capacity Ratio of the wall is the same as that of the prototype design.

The test results showed that the failure modes of these two types of outrigger truss-core wall joints are all elastic-plastic integral buckling of the inclined web members in the outrigger truss, while the joints have no obvious damage. This meets the seismic design requirements of "strong joints -weak members". All of them showed the capability of transmitting internal forces from the outrigger truss to the concrete shear wall, and all of them have good bearing capacity, ductility and energy dissipation capacity (Figure 14).

The steel plate-embedded core wall has a low cracking load, and the concrete near the outrigger connection is

Figure 13. Outrigger-shear wall joint test.

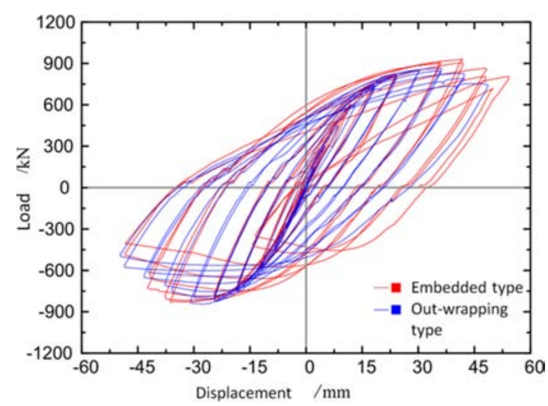


Figure 14. Load-displacement hysteretic loops.

seriously broken before the outrigger is damaged. For the core wall with the steel plate outer-wrapped joint, the cracking load is 6.3% higher, the initial stiffness is also 8.0% higher.

The visible cracks in the wall with the steel plate outer-wrapped joint are obviously reduced, and the crack width is also reduced to a certain extent. At the same time, the phenomenon of massive spalling of concrete at the lower edge of the connection joint between the outrigger truss inclined web and the shear wall is avoided. In addition, in the steel plate outer wrapping type, the inclined web member of the outrigger truss does not reduce its cross-section before connecting to the wall, so when the member loses its overall stability, the end of the member does not have obvious damage. The buckling of the inclined web members of the truss occurs later, and the corresponding displacement increases by 16.6%. In the steel plate-embedded type, the reduction of the end section of the web member leads to obvious deformation at the end when the overall

(a) Steel plate embedded type (b) Steel plate out-wrapping

Figure 15. Connection between outrigger truss and core wall under construction.

instability occurs. The restraint on the web member is weakened, thereby reducing its stable bearing capacity. Figure 15 show the implementation of these two outrigger joints in site.

5.3 Adjustment of Structural Internal Force Distribution by Optimizing Construction Sequence

The 66th floor of the tower is the hotel lobby. In order to create a space with wide view, the column number above this floor was reduced from 16 to 8. Between the 66th and 87th floor forms a mega structure, which composed of box-shaped transfer beams on the 66th floor, transfer trusses on the 86th floors, and 8 giant columns near the 4 corners. The hotel guest rooms between the 68th and 86th floors adopt a sub-structure frame (Figure 16).

The load of the sub-structure is transmitted to the giant column through three paths, the transfer beam, the transfer truss, and the vierendeel truss effect of the sub-structure frame. The transmission ratio of each path depends not only on the vertical stiffness of each part, but also affected by the formation process of the structural stiffness, ie. the construction sequence. If the conventional construction sequence is followed, that is, firstly the temporary supports are erected under the transfer beams, then installed the transfer beams and substructure floor by floor from bottom to top. After the transfer trusses are finally installed, the temporary supports under the transfer beams are removed (Figure 17). Because the depth of the 28.350 m span transfer beams are strictly constrained to 1200 mm by architect, preliminary structural analysis shows that the load acting on the transfer beam will exceed its bearing capacity. If the top transfer truss was solely used to support the load from below, the demand for the top transfer truss will be too large, and the redundancy of the structure will also be weakened. So we decided to adopt the idea of adjusting the internal force distribution of the structure by controlling the construction sequence.

To achieve this goal, the construction sequence of the secondary structural frame must be designed so that the load could be redistributed between the bottom transfer beam and the top transfer truss in a predetermined proportion. The basic idea is to change the force transfer path during construction by temporarily cut off the load path down to the transfer beam. According to structural analysis, the dead

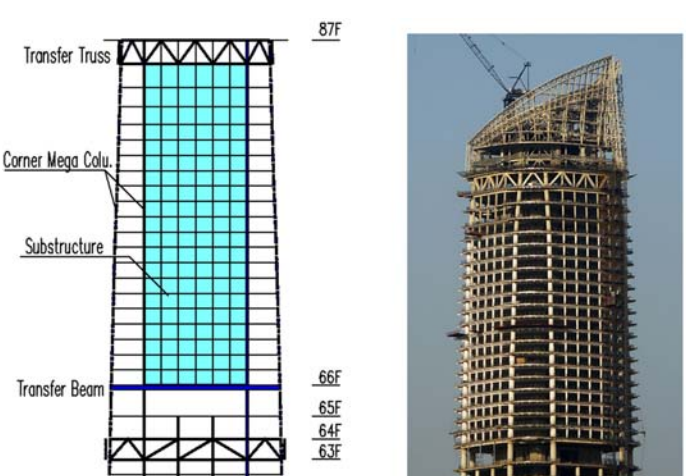


Figure 16. Mega structure between floor 66~87.

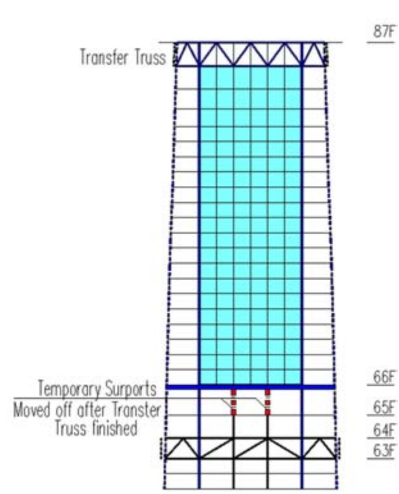


Figure 17. Conventional construction procedure.

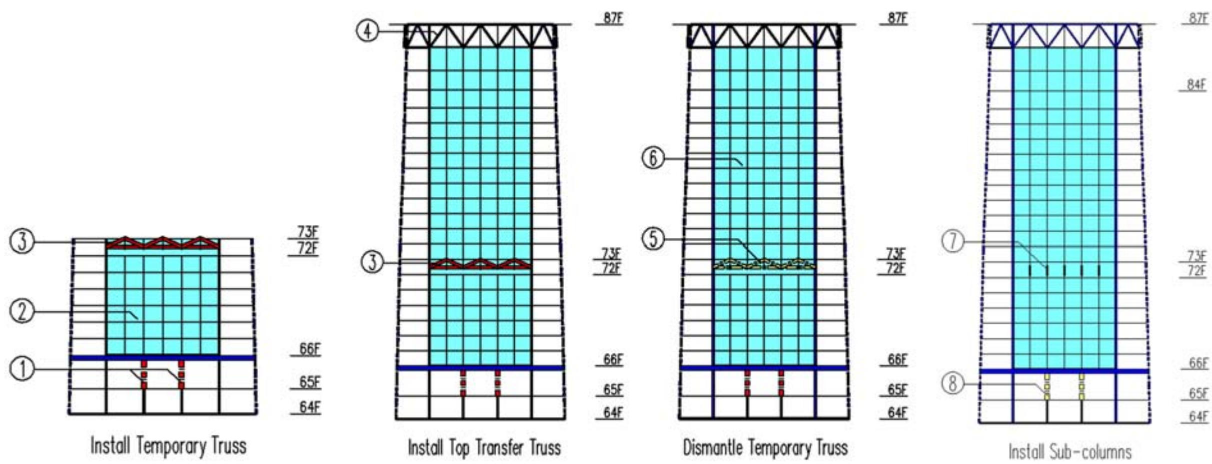


Figure 18. The designed construction sequence.

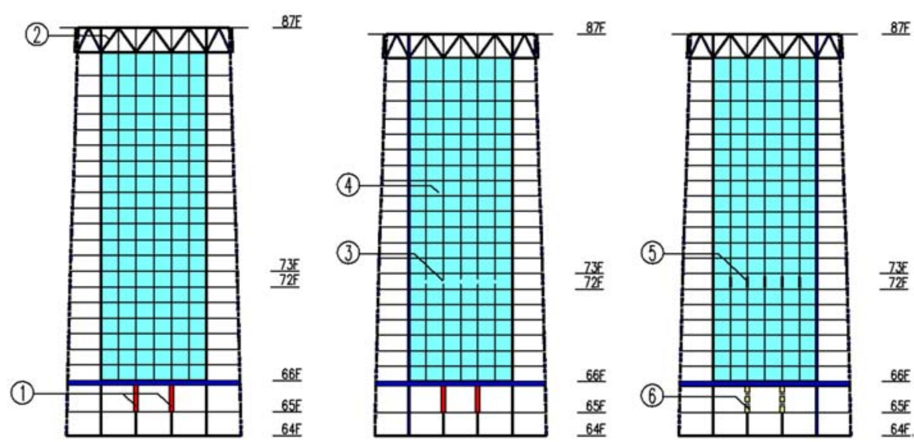


Figure 19. Dis-connecting and re-connecting method.

load transmitted to the transfer beam need to be reduced by no less than 15%.

The load path cut-off position is determined by trial-and-error calculation. The final selected position are the sub-structure columns on the 72nd floor. The designed construction sequence is: ①erect the first temporary

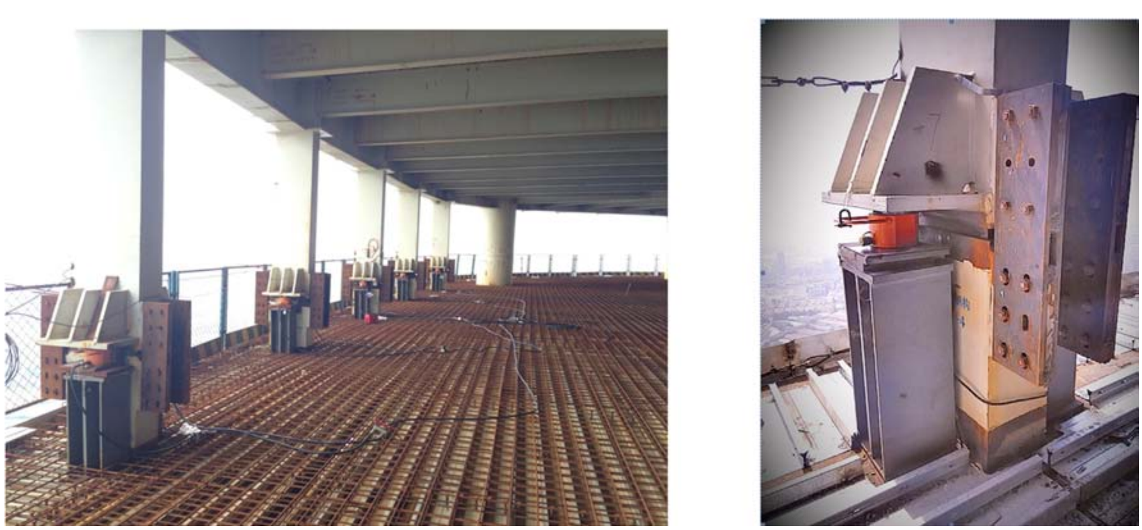


Figure 20. Dis-connecting column with hydraulic jack.

supports under the transfer beams on the 66th floor, ② install the transfer beams and the substructure to the 72nd floor, and suspend the installation of the substructure column of this floor. ③The second temporary support for supporting the floors above the 73rd floor is erected and ④all the upper floors including the transfer trusses are installed. ⑤Remove the second temporary construction support, so that the load above the 73rd floor is mandatorily suspended from the transfer trusses. ⑥After as many additional loads as possible are applied to the floors above the 73rd floor, ⑦the substructure columns on the 72nd floors are installed to form the overall rigidity of the structure. ⑧Finally remove the first temporary support (Figure 18). Comparing with the conventional construction procedure, the total load of the transfer beam can be reduced by 23%, thus satisfied the bearing capacity of the transfer beams.

The second temporary support could be a temporary truss. However, the extra cost of such a truss was too high, so a much simpler construction method named "dis-connecting and re-connecting" is adopted. The process is to ①②construct the structure to the top of the transfer truss in a conventional sequence, ③and then disconnect the sub-structural columns on the 72nd floor to release the dead load of the structure above the floor that was originally passed down. At this time, the self-weight of this part of the structure changes the force transmission path to the top transfer truss through the hanging columns in tension columns. ④After the additional dead load on the upper floors is applied, ⑤the disconnected columns are reconnected by welding. ⑥Finally remove the temporary support (Figure 19).

6. Conclusion

(1) Designing easy-to-construct connection is an important part of ensuring the reliability of realizing structural design goals and speeding up the construction progress, and should be given sufficient attention by structural engineers.

(2) In the steel plate composite shear wall, the connection between adjacent embedded steel plates does not necessarily require equal-strength welding connections. The connection method and connection strength can be determined according to its shear bearing capacity requirements.

(3) When the shear wall is relatively thin comparing to the width of the outrigger chord member, in order to facilitate the construction of the connection, the steel plate out-wrapped joint can be used between the outrigger truss and the core wall, with a better performance than the steel plate embedded joint.

(4) The difference in the formation process of the structural stiffness will lead to the difference in the internal force distribution of the structure. It can be used as an effective tool in structural design by controlling the construction process to actively adjust the distribution of internal forces in the structure.

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