THE STRUCTURAL DESIGN OF THE MEGA TOWER, CHINA WORLD TRADE CENTRE PHASE 3, BEIJING CHINA

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The Mega Tower, China World Trade Centre Phase 3, Beijing China is 330m high and composed of a five-star hotel, grade-A office levels and multi-purpose spaces. The height of the building and the high seismic design intensity in Beijing poses a great challenge to the structural engineers, especially considering various stringent requirements by Chinese codes. Various structural types, utilising different materials, have been explored until the final design was accomplished, which comprises a composite braced frame core and a composite perimeter frame, linked by two outrigger systems at relevant E&M floors. The tapered elevation of the tower building necessitates three transfer belt trusses that allow the reduction in number of perimeter columns in middle and high zone of the tower. The use of 8 storeys high V-shaped columns with transfer belt truss admit of the grand entrances with wide spacing of columns at ground floor for the five-star hotel and grade-A office, and meanwhile provide a smooth structural transition to the perimeter moment frame above. Composite elements of various types are extensively used and positioned carefully to satisfy the combined requirements for stiffness, ductility, redundancy, and cost-effectiveness. The composite steel plate wall (C-SPW) is implemented in the structure, possibly the first time in China, to increase the shear capacity and stiffness and improve the ductile behaviour at specific zone. Accurate finite-element analysis and advanced non-linear elasto-plastic time history analysis have been carried out to evaluate the structural behaviour and ensure the building safety under different seismic levels.

1. Introduction

China World Trade Centre Phase 3 is located in the Central Business District (CBD) of Beijing. The site is surrounded by the East 3 Ring Road on the east, China Grand Hotel to the south and Kerry Centre to the north.
China World Trade Centre Phase 3A includes:

1) A 330m tall mega tower
2) A ball room
3) A retail block

The whole phase 3 will be linked together by means of a basement. Basement level 1 will be used for retails. The structural design of the Mega Tower is introduced below.

With the elevation of helipad of 330m, the Mega Tower are totally 74 storeys above ground, in which 1/F to 4/F are for atrium and multipurpose, 5/F to 56/F are office levels, and above 56/F are for hotel. There are 3 underground levels, B3 is for parking and mechanical spaces, while B2 and B1 are reserved for commercial and mechanical spaces.

2. Structural Challenges and Design Criteria

Lateral stability under seismic load and wind load is the key issue for the design of such a tall building.

The seismic design code of China[1] classifies the seismic resistance requirements by earthquake fortification intensity, which is basically VI, VII, VIII and IX, corresponding to a design ground acceleration of 0.05g, 0.10g, 0.20g and 0.30g respectively with 10% probability of exceedance in 50 years. This project is located in Beijing where the design intensity is VIII, equivalent to zone 2B in UBC97. The seismic design philosophy of the Chinese code is to design to "three levels" of seismic fortification, as described in Table 1 specific for buildings in Beijing.

Table 1 Seismic Design Criteria

Seismic Fortification Level	1	2	3
Description	Minor (frequently occurred)	Moderate (seismic protection)	Severe (rarely occurred)
Design PGA	0.07g	0.20g	0.40g
Probability of Exceedance	50yr 63%	50yr 10%	50yr 2~3%
Fortification Criteria	No damage and remaining elastic	Damage allowed but repairable	No collapse

The structural design of the Mega Tower has to satisfy the fortification criteria for all the three seismic fortification levels. This is further complicated by the fact that the building of this nature does not fall within

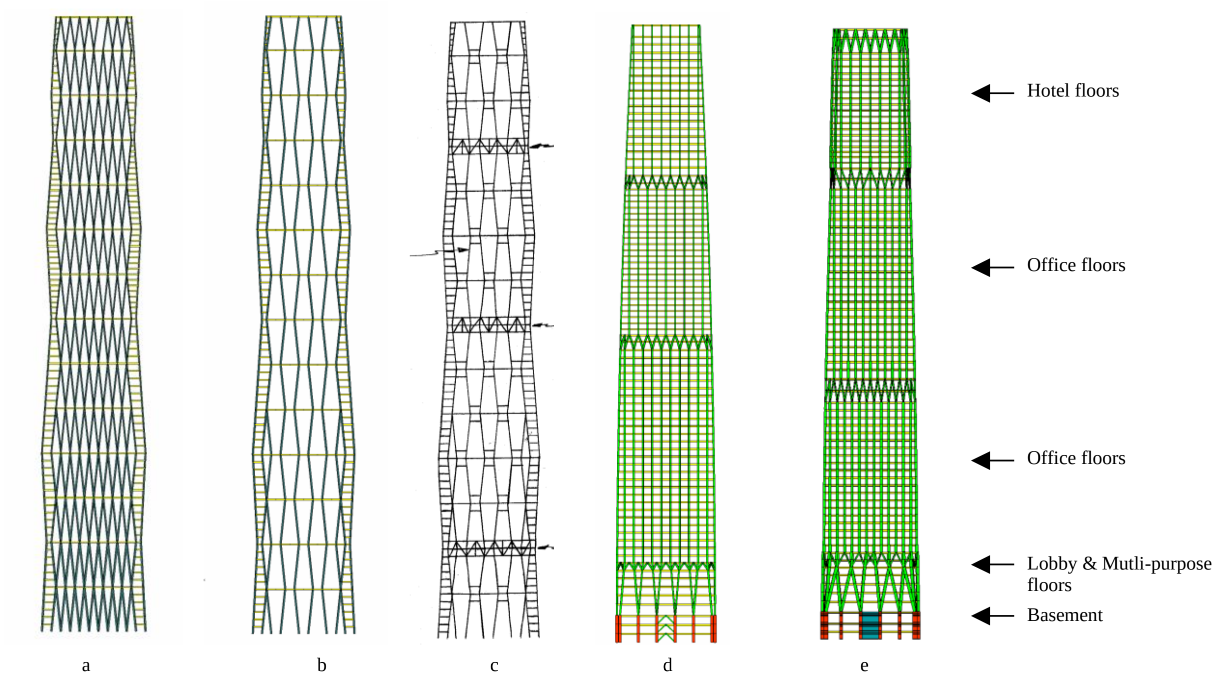


Figure 1: Schemes for the perimeter tube

the limits for which general design approaches are provided in Chinese design code. Hence both the applicable design criteria and the consequent structural preliminary design have to be reviewed by an expert panel organised by the National Seismic Design Review Committee for Buildings Exceeding Limits to get the consent for further work. This was accomplished for the structural preliminary design of Mega Tower, CWTC3 in September 2004.

Wind tunnel tests were carried out in Rowan Williams Davies & Irwin Inc, Ontario Canada. The 100 year return period wind load was sought for the structural design, which by Chinese code is equivalent to 10 minute mean wind speed at 10m height of 28.2m/s.

3. Structural System Design History

The Mega Tower, with a mixed usage for hotel and office, has undergone several major changes in terms of floor area and space division while the exterior look with an undulating profile has been basically kept unchanged. Various structural systems have been proposed and studied to be commensurate with the architectural layout, with the attempt to make the most structurally safe and cost-effective use of different materials. The schemes for the perimeter tube and the central core were effectively developed separately and then combined to a dual system. For the perimeter structure, several concepts with variations have been studied:

1. Steel diagrid frame (Fig 1a). This may have been the most structurally efficient option by combining the lateral and vertical load transfer system together. Different shapes of triangular unit were explored for best compromise between structural efficiency, build-ability and architectural design intent. The corner members were removed and replaced by flexural link beams to avoid fabrication problems of the large and shallow interconnection.
2. Steel hexigrid frame (Fig 1b, 1c). In order to improve buildability and views from the building, a diagonal hexigrid frame solution was derived from the diagrid option by removing pairs of diagrid frame members. This dramatically improved the open window space and also reduced the complex cross connections thus improving build-ability. The advantage of this system is that it was a hybrid system and achieved strength and stiffness from the axial stiffness of the inclined members and the flexural stiffness of the joints between the inclined members and the edge beam members.

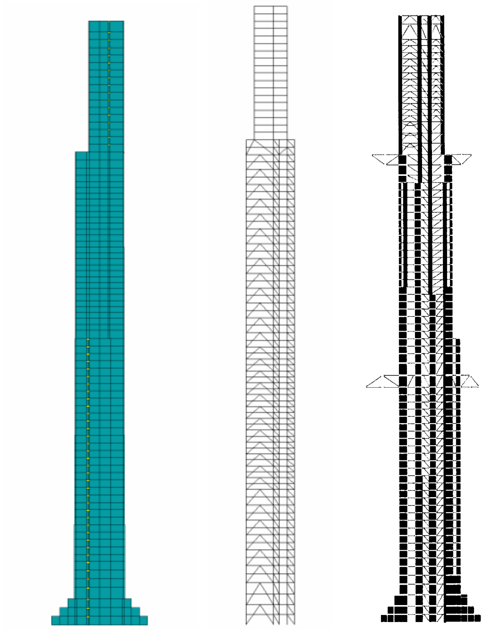


Figure 2: Schemes for the central core

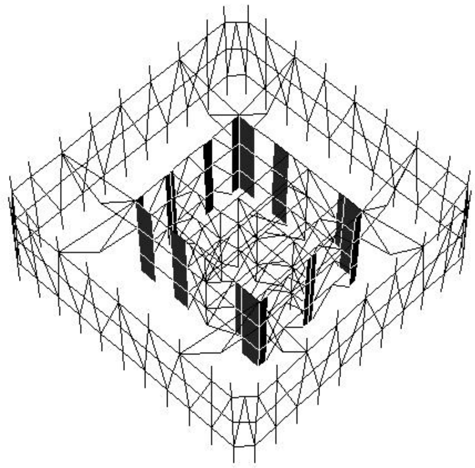
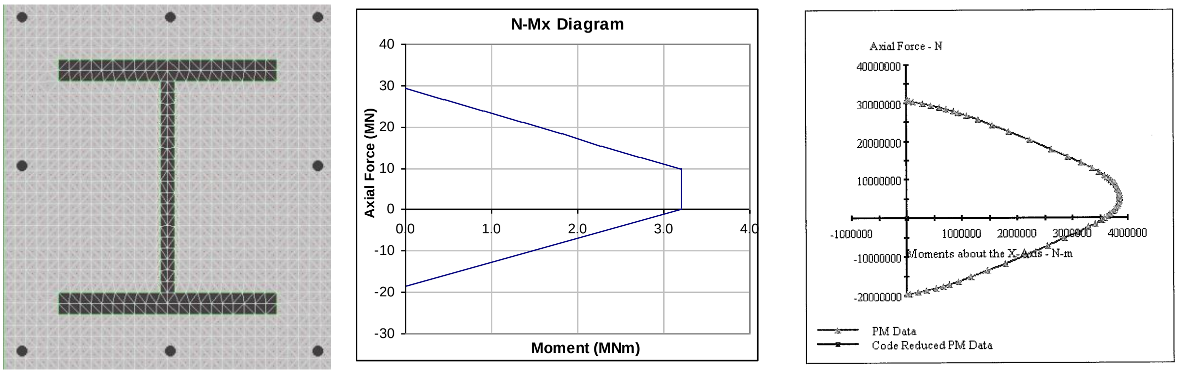


Figure 3: Isometric view of outriggers and the belt truss

3. Moment frame. Among all the schemes, moment frame provided the best exterior view for all profitable floors. The column grid was initially 5m (Fig 1d) for all levels but finally 5.6m spacing in hotel zone and 4.2m in office zones were adopted (Fig 1e). Belt trusses located in the mechanical levels of 6-8, 28-30 and 55-57 facilitated column shifts to cater for the reduction in the number of columns resulting from the gradual tapering of the building shape. Coupling with the outriggers, the belt trusses improved the shear lag effect of the perimeter tube and mobilised the perimeter columns more efficiently. At the bottom beneath the two-storey transfer belt truss V-shape braced perimeter columns achieved double column spacing for the entrance lobby level, providing a strengthened base for the perimeter frame instead of weakening it as many transfer structures..

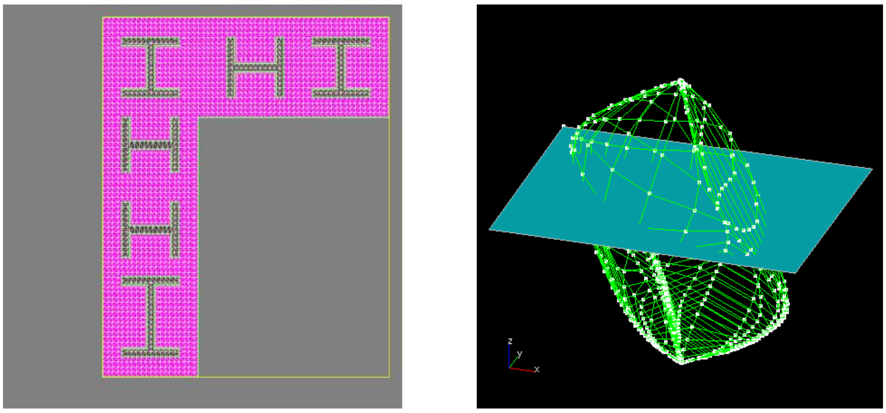
The central core options, which were largely confined by the architectural layout and building service requirements, differentiated themselves primarily by the usage of the material. During the development history, three possible schemes, the concrete shear wall (Fig 2a), the pure steel braced frame (Fig 2b) and the composite braced frame (Fig 2c) have been studied in depth. While all structurally feasible they are, the key issues were how structural redundancy and safety margin could be adequately provided under earthquake events in a cost-effective way. The composite braced frame, which is composed of composite columns and steel beams and braces, was deemed to be the most appropriate. Both concentric bracing and eccentric bracing were used not only to accommodate openings and ducts, but also to supply more ductile mechanism within the stringent requirement on the lateral stiffness.

The final design is then the combination of the composite moment frame perimeter tube shown in Fig 1e and the composite braced frame core shown in Fig 2c. They are further linked and enhanced by two sets of single-storey outriggers between Level 28-29 and level 56-57, each set of 8 outrigger arms (Fig 3). The tips of outriggers arms connected to the junctions between the belt truss and the moment frame result that one outrigger arm would actually mobilise two adjacent columns. Studies have proved that such arrangement, in conjunction with the belt trusses, improves the shear lag effect of the perimeter tube significantly because the central few columns on four faces of the perimeter frame, which are originally inefficient in the framed tube action due to their locations far away from the corners of the frame and perimeter beams with insufficient stiffness, are hence efficiently mobilised by outriggers.



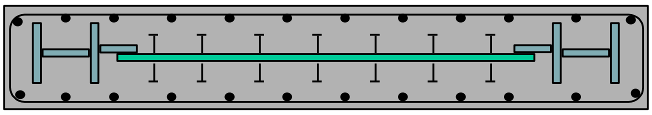
a. Sample section b.Results of BS5400:Part5 c. Results of finite element analysis

Figure 4: N-M interaction diagrams for standard rectangular SRC column

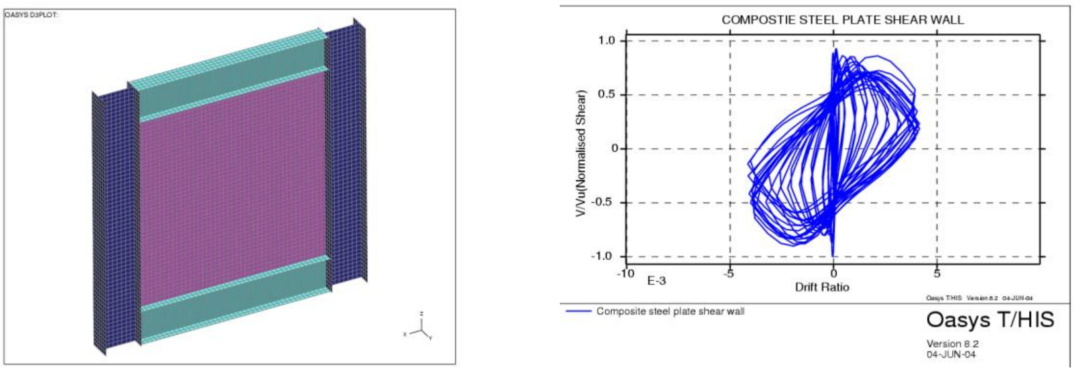


a. Finite element model b. Three-dimensional N-Mx-My interaction surface

Figure 5: Modelling and Capacity Curves for complicated L shape SRC columns



a. Typical configuration



b. FE model for non-linear verification analysis c. Hysteresis curves of C-SPW by numerical analysis

Figure 6: The composite steel plate wall (C-SPW)

Unlike the regular rectangular shape of the composite columns in perimeter frame, the composite columns in the core are of L shape or rectangle with multiple (nos 2 to 7) steel stanchions cast in (Fig. 5). Steel beams and braces are connected to these steel sections directly. The design complexity of such columns has been fully envisaged and verified with both international codes and finite element analysis, which will be presented later.

The internal core is further enhanced at the lower levels by the composite steel plate walls (C-SPW) which provide great lateral shear resistance under severe earthquake events and contribute stiffness to resisting wind load and frequently occurred earthquakes (seismic fortification level 1). The C-SPWs, which are placed in the internal partition areas of the core and between the composite columns, would actually integrate with those columns via steel connections, concrete and reinforcement (Fig 6a and 6b). In basement levels C-SPWs are introduced on the perimeter of the core and further project out at corners in each side to act as stiffeners for the pile cap.

The structural steel composite floor system is adopted. The typical floor beam span varies from 8m to 16m due to the tapered profile of the building. Human comfort due to the vibration of the floor system has been carefully checked especially for the hotel zone where the depth of the concrete slab is increased accordingly.

As the bearing capacity of the soil under the Mega Tower is not sufficient to support the superstructure, pile foundation was proposed for the Mega Tower. The piles are 1200mm in diameter and 55m in length and the toe level is 75m in depth from the ground level. In addition to the walls projecting out at core corners, the pile cap, 4.5m deep generally and 4m deep at some perimeter area locally, distributes the loading from the columns and walls in the basement to the pipes evenly.

4. Elastic Analysis

Three-dimensional computer models utilising two different analysis software according to the requirement of Chinese design code have been set up with all structural elements of the superstructure and the basement modeled explicitly. It is assumed in the model that the Mega Tower is fixed at the top of pile cap with the lateral restraint effect arising from basement floors outside the footprint of the tower considered properly. In the model, monolithic sections with equivalent sectional properties sensibly represent the composite elements. Rigid diaphragms may reduce the degree of freedom significantly, resulting in use of elastic slab elements at the area where this assumption would unnecessarily stiffen the structures, for example the outriggers and the braces.

Response spectrum analyses according to Chinese seismic design code (GB50011-2001) have been carried out, with the analysis results being verified by the average results of multiple elastic time history analyses using one artificial TH curves and two natural seismic acceleration records, as shown in Fig. 7.

The periods of the primary translational modes in two principal axes are 6.82s and 6.55s respectively while the period of the first torsional mode is 3.66s. The base shear forces in X and Y direction of the response spectrum analysis are 50.3MN and 51.0 MN respectively, which have been rectified to 67.6MN in compliance with minimum base shear requirement for specific element capacity checking. The seismic overturning moment in X and Y direction are 10297MNm and 10087MNm, becoming 13828MNm and 13360MNm respectively for the same reason. The base shear and overturning moment of the resultant wind load is 33MNm and 6666MNm, which is far less than the counterparts of the seismic effect.

The maximum inter-storey drift ratio resulting from the seismic and wind effect is 1/559 and 1/833 respectively, as shown in Fig. 8.

The critical members such as belt trusses and the V-shaped columns at bottom have been designed to remain elastic at seismic fortification level 2.

5. Composite Design

The composite columns, especially those located at the core corners, are subjected to the combined effect of axial force and bi-axial bending moment. As being not covered in Chinese code, the design methodology in BS5400 Part 5 has been employed with proper modifications to incorporate the material properties and the like in Chinese code. Accurate finite element sectional analyses assuming each mesh as a fiber to deform on the basis of plane section remaining plane has been carried out to verify the code-based results. It is proved that, in comparison with the FEA results, the BS5400 methodology would give relatively

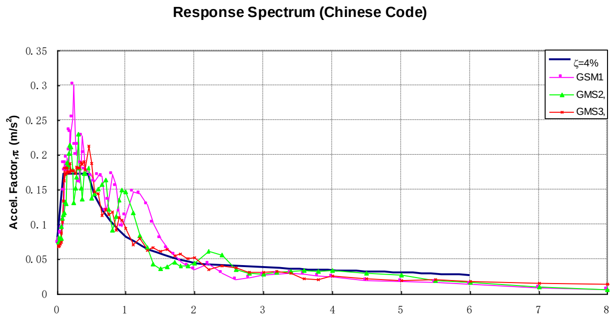
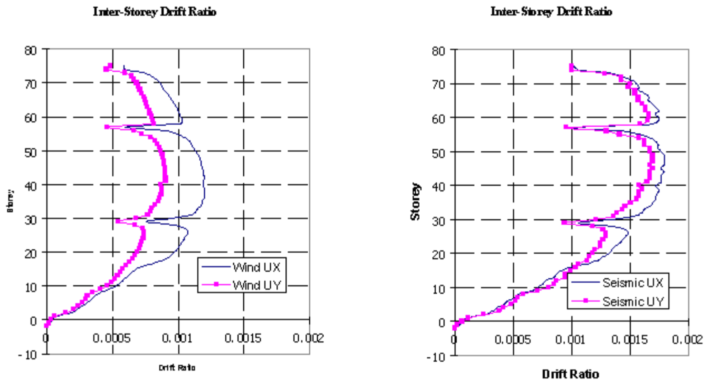


Figure 7: Design Response Spectrum and Time History Records



wind Seismic

Figure 8: Inter-storey Drift Ratio under Seismic and Wind Load

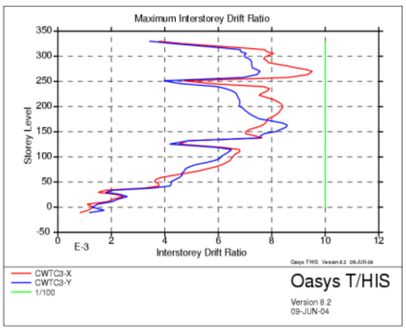
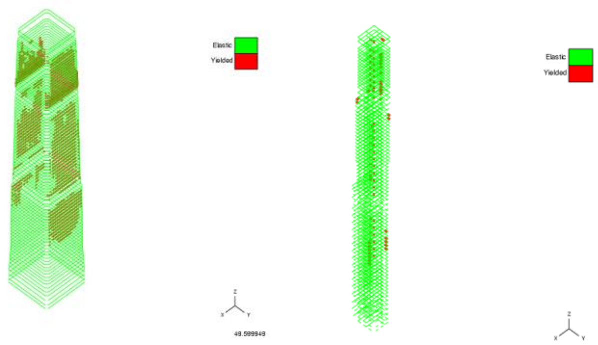


Figure 9: Max Elasto-plastic Inter-story Drift Ratio under Severe Earthquake (Artificial Time History Record)



a. perimeter frame beam b. core beams

Figure 10: Beam Plasticity under Severe Earthquake (Artificial Time History Record)

conservative results for the capacity of rectangular SRC columns.

However the L-shaped composite columns are beyond the coverage of the codified design as these columns are of unsymmetrical sections and the finite element analysis indicates that the N-Mx-My interaction curving surface is distorted spatially (Fig. 7). Therefore, the capacity curves produced by FEA are used in design, which will be further verified by specimen tests.

The composite steel plate wall (C-SPW) has been utilised in seismic zones of North America and Japan for a period and relevant provisions have been included in IBC/AISC[2,4] and Japanese codes. It is defined as[a] structural walls consisting of steel plates with reinforced concrete encasement on one or both side of the plate and structural steel or composite boundary members. With the restraint of the reinforced concrete, the shear capacity of C-SPW is able to be up to the shear yield capacity of the whole steel plate. This is significantly greater than the shear capacity of pure steel plate wall, where only the diagonal tension field is effective[3]. This composite mechanism of C-SPW provides sound ductility capacity (Fig. 6c) in addition to lateral stiffness. Fire protection is not necessary if the reinforced concrete is on both sides of steel plate.

As such elements are introduced in China possibly for the first time, the design criteria have been carefully considered. The member force adjustment factors for the composite walls follow those designated to RC walls originally and the shear capacity of steel plate is designed accordingly. On the other hand to comply with the design philosophy of “no damage in minor earthquake”, the concrete encasement is reinforced to resist the seismic fortification level 1 shear force distributed by its stiffness proportion in the composite section.

6. Non-linear Elasto-plastic Time History Analysis

In an event of a rarely occurred earthquake, the structural element is allowed to deform plastically but P-Delta effect due to the gravity load acting on the building geometry with excessive lateral deformation would result in collapse of the entire building, which should be prevented. Therefore, an analysis with the structural model subjected to according ground motion excitation history, considering both material non-linearity and geometric non-linearity, has been carried out using LS-DYNA to evaluate the behaviour of the Mega Tower in such circumstance. The maximum elasto-plastic inter-storey drift and the plastic deformation of various structural elements are the criteria adopted for such evaluation. As suggested by the expert panel during the review, the limit for this inter-storey drift ratio has been set as 1/100, which should be used for RC frame-core system. For limits of plastic deformation of single element, FEMA356[5] is referred to as Chinese code does not provide such provisions.

The results of the 3-D elasto-plastic time history analysis using LS-DYNA reveal that plastic deformation occurs mostly at beams (Fig. 10) while the critical area such as transfer trusses and bottom V-shaped columns remains elastic. The plastic deformations of all elements are within their corresponding limits and the maximum inter-story drift ratio is 1/105, less than the preset limit.

7. Conclusion

As from making use of innovative structural systems with eccentric braces and composite steel plate walls to performing accurate finite element analysis and advanced non-linear elasto-plastic time history analysis for ensuring both structural behaviour and safety under different seismic levels, the structural design of the CWTC Phase 3 Mega Tower has not only been a comprehensive and detailed exploration of the innovative structural schemes, but also demonstrated that the probably best balance between the economy and the structural safety for a super high-rise building located in a highly seismic zone in China is capable of achieving by extensive use of concrete and steel as composite material for structures with sufficient ductility.

References

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