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# The Structural Design of Tianjin Goldin Finance 117 Tower

**Peng Liu[1†], Goman Ho[1], Alexis Lee[1], Chao Yin[1], Kevin Lee[1], Guang-lei Liu[1], and Xiao-yun Huang[1]**

[1]*Arup, Beijing 100020, China*

**Abstract**

Tianjin Goldin Finance 117 tower has an architectural height of 597 m, total of 117 stories, and the coronation of having the highest structural roof of all the buildings under construction in China. Structural height-width ratio is approximately 9.5, exceeding the existing regulation code significantly. In order to satisfy earthquake and wind-resisting requirements, a structure consisting of a perimeter frame composed of mega composite columns, mega braces and transfer trusses and reinforced concrete core containing composite steel plate wall is adopted. Complemented by some of the new requirements from the latest Chinese building seismic design codes, design of the super high-rise building in high-intensity seismic area exhibits a number of new features and solutions to professional requirements in response spectrum selection, overall stiffness control, material and component type selection, seismic performance based design, mega-column design, anti-collapse and stability analysis as well as elastic-plastic time-history analysis. Furthermore, under the prerequisite of economic viability and a series of technical requirements prescribed by the expert review panel for high-rise buildings exceeding code limits, the design manages to overcome various structural challenges and realizes the intentions of the architect and the client.

**Keywords:** Super high-rise building, Mega frame, High seismic zone, Mega column, Composite steel plate shear wall

## 1. Engineering Background

Measured from the structural roof level, Tianjin Goldin Finance 117 is the tallest building under construction in China. The scheme development of the structural system of this building has taken nearly two years, and many different schemes have been studied. The design consideration of super high-rise buildings is nowadays not only the function and cost for the space inside the building. It is naturally the symbol of a city, taking responsibility for elevating the land value and economic activity of the proximity, as well as the outcome of a huge amount of initial and maintenance cost. The structural design was therefore, unavoidably, to be affected by many aspects other than the structural safety. Aside from these, there is the challenge of designing the slim tower in the high seismic zone of Northern China. The high aspect ratio of the building dominates the structural design and the seemingly simple structure was actually the result of several runs of scheme development.

The tower is located in the Central Business District of Gaoxin District in Tianjin, China, accommodating grade-A office spaces, a 6-star hotel and ancillary facilities with a gross floor area of approximately 370,000 $m^2$. The architectural height is about 597 m with 117 stories (total of 126 structural stories). The project is financed by Goldin Properties Holdings Limited, and is the tallest in terms of roof height among the buildings under construction in China.

The mega tower has a square plan with dimension reduced along the height, which follows a tapered shape on elevation. The plan dimension is approximately 65 m by 65 m on ground level (at edge of façade), reduced gradually to 45 m by 45 m at the roof level. The central reinforced concrete core is a rectangle, containing shuttle lifts, plant rooms and services.

**Figure 1.** Perspective view (Courtesy of P&T Group).

†Corresponding author: Peng Liu
Tel: +852-2268-3154
E-mail: Peng.Liu@arup.com

## 2. Design Challenges

The structural height of the tower is 584 m (to main roof) and the structural width at ground level is 61.24 m (external edge of mega column). The height to width ratio is over 9.5, significantly exceeding the limit of 7 imposed by the seismic code for buildings of Class B height at Seismic Fortification Intensity 7 region (GB50011-2001, 2008; GB50011-2011, 2010; JGJ-2002, 2002). Approval from a National Expert Panel is thus required (Jianzhi, 2010. 109).

Tianjin is located in Northern China with high seismic intensity (Intensity 7, 0.15 g) with soft ground condition. It is 120 km away from Tangshan, where the disastrous earthquake in 1976 took away the life of around 240,000 people. The thick soft soil layer amplifies the seismic force further to be close to Intensity 8 or as in Beijing. According to Chinese codes and practice, a more stringent set of controlling criteria had to be adopted, leading to more challenging technical requirements and conditions for seismic design of the structure.

Continuous exploration and optimization has been carried out during the schematic and preliminary design stage, from overall structural scheme to general arrangement of elements, to ensure adequate structural stiffness and an economical engineering solution.

## 3. Development of Structural Systems and Member Design

### 3.1. Building plan

Through coordination with the architect and service engineer throughout the design process, an almost bi-axially symmetrical arrangement has been achieved for the structural plan and core. The reinforced concrete core is regular and symmetric, offering good earthquake and wind resistance. At the same time, structural openings are reasonably arranged in both directions to form lintel beams, creating a ductility and energy dissipating mechanism while satisfying stiffness requirements.

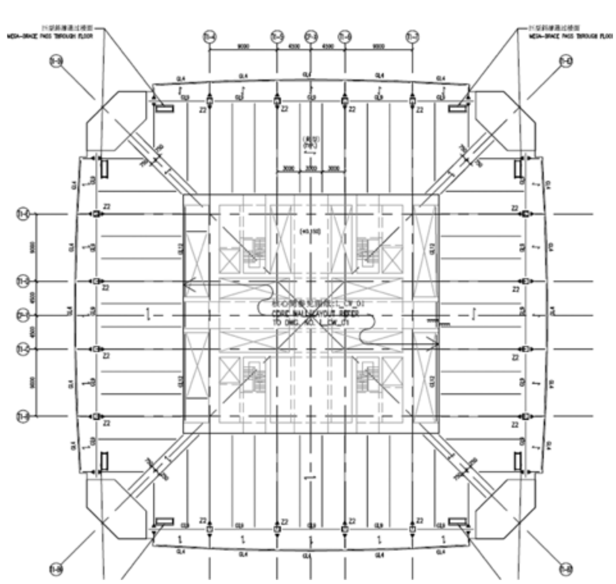

**Figure 2.** Structural Arrangement Plan of Ground Floor.

### 3.2. Perimeter structure

To achieve the building arrangement and ensure structural safety, the advantages of structural steel and reinforced concrete were maximized for the best engineering value. The following schemes were considered for the perimeter frame.

#### *3.2.1. Frame-tube structure (with outrigger truss and belt truss levels)*

The major issue with this scheme for tall buildings over 400 m high is achieving the required stiffness. Chinese codes require the perimeter frame, as the second line of defence, to meet specific stiffness and strength criteria, which leads to a large perimeter column section and close column spacing against the architectural requirements for space and view. The overall steel ratio is high, while multiple outrigger trusses add complexity to its connection with the core, increase construction cost and lead to unfavourable abrupt stiffness changes.

#### *3.2.2. Combined mega-brace and moment frame (inverted K brace or diamond brace)*

This scheme utilizes the high stiffness characteristic of mega brace, but the force of gravity is transferred to the ground through multiple columns with a large cross-section meanwhile the mega columns are in tension under lateral load combination. The building space requirements are not ideal.

#### *3.2.3. Mega-brace frame (with transfer truss)*

This scheme diverts the force of gravity from the perimeter columns through mega transfer trusses at the mechanical and refuge floors to the corner mega columns on both sides, thereby greatly reducing the size of the perimeter sub-columns and facilitating a spacious building environment while enhancing lateral structural stiffness. Regarding the brace pattern, both inverted K brace and diamond brace has a relatively large angle to the horizontal and thus is less efficient in contributing to the stiffness required.

The requirements imposed on the mega tower by its height to width ratio lead to the adoption of a more efficient brace arrangement. Through coordination with the client and architect, a cross brace pattern was finally adopted for all zones, except the lowest zone which adopts an inverted K brace arrangement to suit the main entrance requirements. This significantly enhances the overall stiffness of the mega tower and maximizes element efficiency to satisfy series of technical structural requirements against seismic and wind action.

Since the stiffness of the perimeter mega frame exceeds

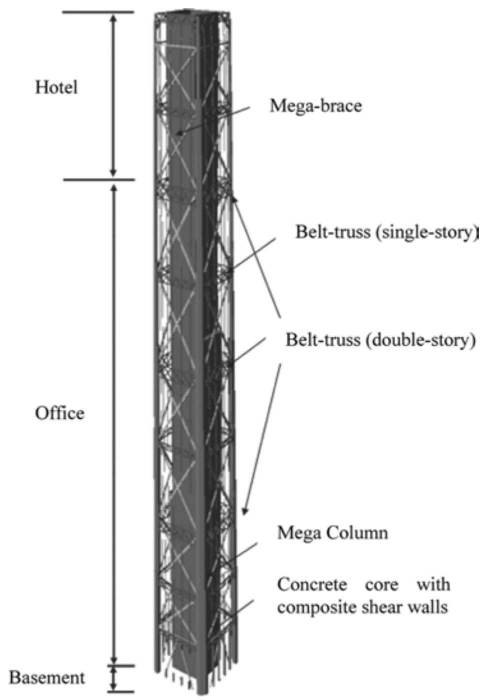


**Figure 3.** 3D Illustration of Structural System.

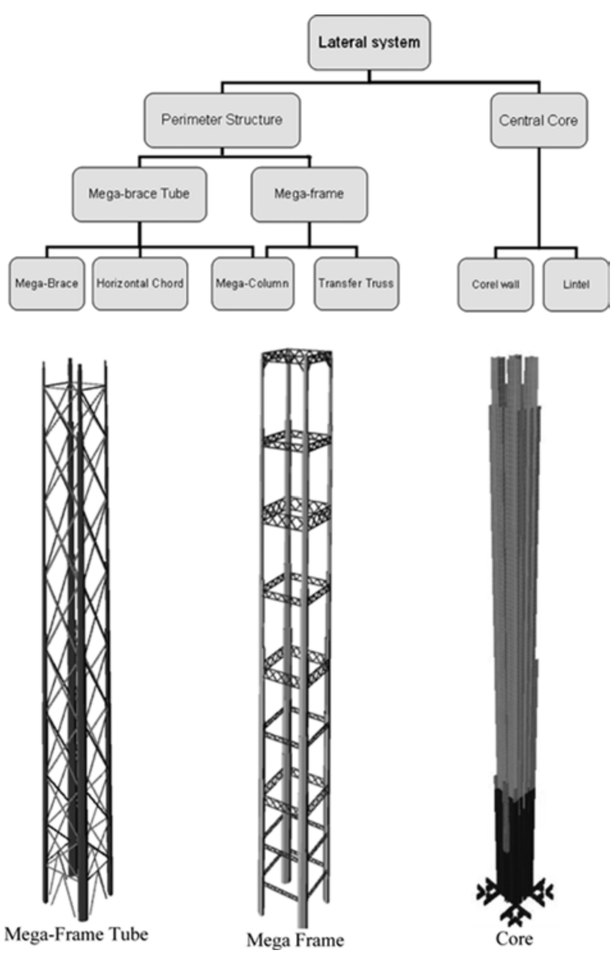


**Figure 4.** Multi-Tier Lateral Stability System.

that of the core in most of the stories, outrigger trusses do not perform much in improving overall structural stiffness. Thus, outrigger trusses are not adopted in the final design.

The final structural stability system, as shown in Fig. 3, comprises a reinforced concrete core and a perimeter structure with mega-braced tube and mega frame to form a multi-tier structural stability system, providing magnificent lateral stiffness, at the same time as resisting lateral earthquake and wind loading.

The relationship between mega brace and perimeter sub-frame in each zone is another issue involving not only structural design. The urban planning authority and the client would like to weaken the visual impact of the cross-bracing. The braces were thus offset backwards from the perimeter beam-column sub-frame, as a side result also simplifying the gravity load path.

Each sub-frame zone is about 15 stories, in which a beam-column frame passes the gravity load to the belt truss below, which in turn transfer it to the mega columns. To prevent the potential progressive collapse when the lower part of the sub-column is damaged (in an extreme scenario, for example under blast or external collision), an alternative load path for the upper portion is provided by connecting the columns with the belt truss above via a long slotted joint which was triggered like a "fuse" when the lower column fails. As the belt truss is designed to a high demand in the horizontal/vertical seismic load combination, this simple structural detail enhances the robustness and safety of the gravity system without extra cost.

### 3.3. Core

The core extends from the top of the pile cap up to the roof of the mega tower, passing through the full height of the building. The core is rectangular in plan and centred. The plan dimensions of the core are about 34 m × 32 m at the base, with the wall gradually being set back on both sides of the core, the core becomes an approximate square on Level 67, with a bi-symmetric arrangement up to the roof. Typical lintel beam depth between the core wall panels is 700 mm, while openings are arranged regularly on

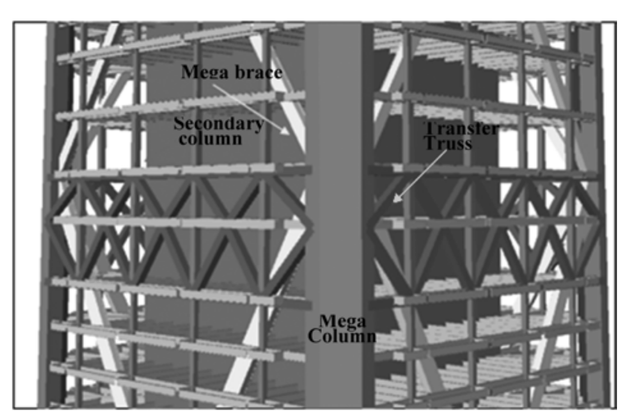


**Figure 5.** Illustration of Mega Frame and Mega Brace connection – double-storey truss.

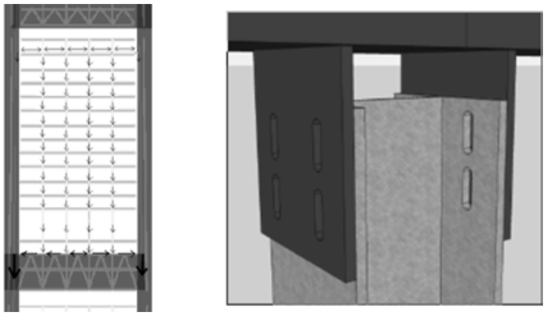

**Figure 6.** Elevation of perimeter frame and slotted joint at the top.

plan, wall panels are also arranged equally, openings are arranged regularly and continuously without offset in the vertical direction.

The core adopts steel-reinforced concrete shear walls with embedded steel sections. At the low zone, alongisde steel sections, steel plates are added to form composite steel plate walls (C-SPW) to prevent shear failure of the concrete walls in case of a severe earthquake. This system has been widely adopted (Liu et al., 2008) for super-high-rise buildings in the country since its first introduction in the China World Trade Centre Phase 3A project. The embedded steel plate increases the shear capacity of core. Once the wall is proved to be strong in shear, the ductility of reinforced concrete shear wall can be guaranteed. The axial stress ratio requirements (Sun et al., 2008) could be reduced as typical shear walls. At the same time, this kind of C-SPW will be induced any sound during oscillation like pure steel plate shear wall. Besides that, reinforced concrete has the benefit of substantial stiffness, good sound insulation and sound fire resisting performance which also reduce the construction and maintenance cost. The use of composite wall increases the compressive and shear capacity of the element, efficiently reducing the self weight of the structure and hence the mass.

In the schematic design stage, a braced steel core scheme was considered for the upper portion of the tower. Because the tower is governed by the overall stiffness criteria and the peak acceleration for wind comfort is approaching its limit, it is difficult to achieve the overall performance using a steel braced core even if it can reduce some of the structural self weight.

The outline face of the core wall gradually reduces from 1400 mm thick at the bottom to 300 mm at the top, while the majority of the internal wall reduces from 600 mm to 300 mm. Steel plate arrangements within the wall panels vary from two 35 mm thick steel plates at the bottom to a single 25 mm steel plate at about Level 32. At the top of the tower above the highest belt truss, the internal core is subject to whipping effect and thus enhanced with embedded steel plates and braces as well.

The core contributes more significantly in self weight than in stiffness. However special care should be taken in

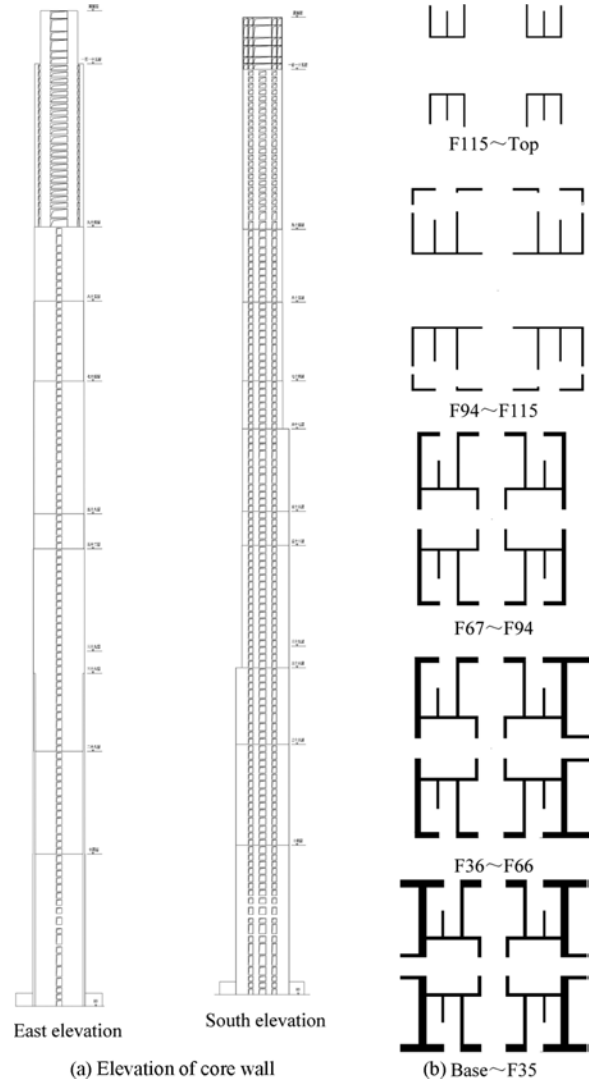


**Figure 7.** Composite Steel Plate Wall plan arrangement.

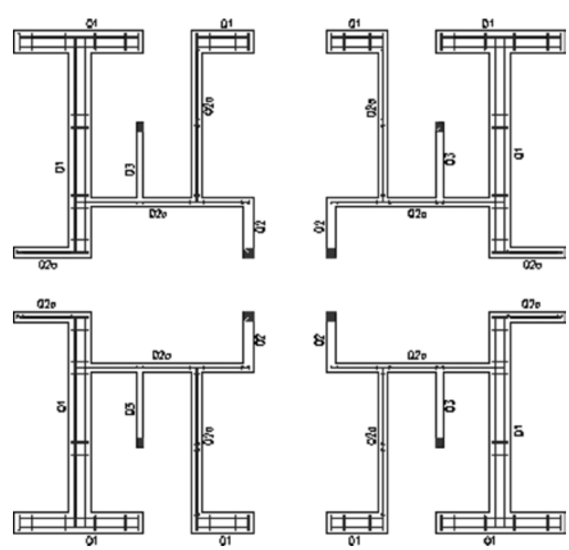

**Figure 8.** Ground Floor Composite Steel Plate Wall and Cast-in Steel Post arrangement.

the reduction of core walls to maintain necessary stiffness to control the top wind-induced acceleration. At mid and upper zone of the core, steel posts are strategically introduced at critical locations like wall corner and edge to enhance the capacity and ductility of the structural member.

To reduce the self weight of the structure and ease construction of the connection, composite floor system is adopted within the core with a typical slab thickness of 120 mm for general use.

### 3.4. Mega column

Mega columns are strategically located at the four corners of the building plan and extend to the top of the tower, connecting beams, transfer trusses and mega braces at each zone. Its plan shape satisfies the architectural profile and structural connection requirements, resembling a six-sided polygon with cross sectional area of about 45 $m^2$ at the bottom. The mega column reduces its size in zones along the height while satisfying the architectural spacing requirements, with external face flushed. The topmost section is about 5.4 $m^2$ in area.

There was, certainly, the concern of corner columns blocking the invaluable corner view. However it was found to be the only solution to provide enough lateral stiffness in the context of Chinese code and review system, similar to the adoption of cross-bracing.

The mega column connects with transfer truss and mega brace. The sectional design started with the steel reinforced concrete column but ended as the polygonal concrete filled tube. The normal practice of discrete steel sections in SRC columns had been considered to have insufficient ductility based on results of previous tests in other projects. The requirement of a full-height inter-connection of all steel sections resulted in a closed continuous steel section which is virtually a concrete filled tube section. (Yan et al., 2010; Rong, 2007), the final design is an external steel plate enclosure with internal inter-connected plates forming separable chambers in accordance with the detailing requirements (as shown). The six-sided polygonal concrete filled tube composite member has great capacity against axial, bending and shear under lateral action from earthquake and wind.

The overall steel content of the mega column outside the connection zone is about 4~6%, using Q345GJ (or Q390GJ) grade steel with high-strength concrete infill of grade C70~C50. Each compartment is distributed with reinforcement of 0.5~0.8% ratio, enhancing the strength of the member while at the same time minimizing the undesirable effects of creep and shrinkage of concrete. Vertical stiffeners are arranged symmetrically on the inner face of the compartments, and linked with reinforcement ties to restraint out-of-plane buckling of the steel plates (Zhong, 2003).

The structural design of the mega columns reaches a balanced compromise between various factors including architectural arrangement, overall structural stiffness, element performance under loading, connection design, construction cost, production and constructability, achieving the best overall economic and technical performance. The design will be verified by lab tests to check its axial capacity and ductile behaviour. The construction of the mega column is another challenge which shall be overcome by engineers at next stage.

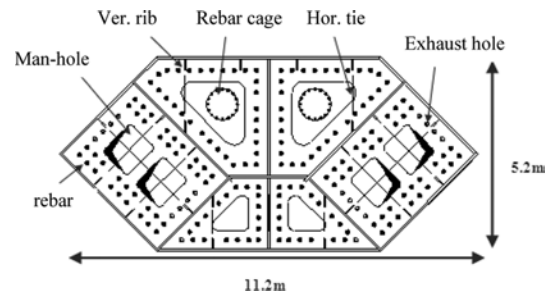


**Figure 9.** Typical low zone 45m$^2$ Mega Column section detail.

### 3.5. Mega brace and transfer trusses

The mega braces are arranged on the 4 elevations of the tower, using welded steel box sections and connecting with mega columns. Since mega brace is separated from the perimeter beam-column frame, lateral support was carefully arranged in the floor system to restrain the mega brace in out-of-plane directions.

Transfer trusses (belt trusses) suit with architectural and services requirements and locate at mechanical and refuge floors. There are 5 sets of single-storey trusses and 3 sets of double-storey trusses, distributing evenly at approximately every 12 to 15 floors. The transfer trusses resist the gravity loading from the zone and transfer it to the corner mega columns, at the same time interacting with the mega columns at the corners, enhancing the torsional stiffness of the tower. Under severe seismic activity, transfer truss will become an important element in preventing progressive collapse of the floors and ensuring safety. In the design, vertical earthquake action has been considered for the long span structure and its performance criteria have been raised to not yield under severe seismic activity.

### 3.6. Floor system

Outside the core of the tower, a composite floor system is adopted with simply supported beams spanning from 6~13 m from top to bottom of the tower at 3 m typical spacing. The office slab floor is 120 mm thick and the hotel floor is 130 mm thick.

To ensure reliable transfer of horizontal shear force between the core and external frame, main tower and podium wings as well as main structure and basement, slab stress analysis has been carried out and the floor structure has been strengthened accordingly through thickening and introduction of plan bracing. Slab thickness of strengthening floor (where the transfer truss is located), main tower floors near podium wing roof, roof of podium

**Table 1.** Seimic performance objectives

| Seismic intensity (reference level) | | | Frequent seismic (Level 1) | Fortification level (Level 2) | Severe seismic (Level 3) |
|---|---|---|---|---|---|
| Qualitative performance level | | | No damage | Repairable damage | No collapse |
| Inter-storey drift limit | | | h/500 | - | h/100 |
| element performance | Core | Combined axial load and bending | Code design requirements, elastic | Elastic (strengthening zone at bottom) | Allow plastic hinges, control plastic deformation |
| | | | | No yield (other floors and secondary wall panels) | |
| | | Shear | | Plasticity allowed | No yielding |
| | Coupling beam | | | Plasticity allowed | First to enter into plastic stage |
| | Mega-column | | | Elastic | No yielding[1] |
| | Sub-frame Perimeter beam and sub-column | | | Elastic | Allow plastic hinges, control plastic deformation |
| | Mega-brace | | | Elastic | |
| | Transfer truss | | | Elastic | No yielding[1] |
| | Other structural elements | | | Plasticity allowed | Allow plastic hinges, control plastic deformation |
| | Connection | | No damage prior to element damage | | |

[1]consider vertical earthquake under Level 3 earthquake

wings, first floor and restraint floor are 200~300 mm.

To prevent complicated connection detail between reinforced concrete beam and cast-in steel sections inside the core, the floors inside the core and basement floors also adopt a composite floor system.

### 3.7. Seismic performance based design requirement

As the structure exceeds code limit considerably, seismic performance objectives have been established for the overall structural behaviour and element performance according to performance based design principles and numerous discussions with the expert review panel as shown in Table 1. Elasto-plastic time-history analysis was used to confirm the seismic performance objectives for severe earthquakes (return period 1 in 2475 years). Lintel beams of the core develop plastic behaviour fully first, and the critical members, such as mega column and transfer trusses were carefully controlled to avoid excessive plastic deformation.

## 4. Elastic Analysis of Overall Performance

### 4.1. Basic parameters

The design reference period and design working life is 50-year, and for durability design 100-year is adopted. The seismic fortification intensity is 7 with design peak acceleration of ground motion at 0.15 g (return period 1 in 475 years). The building seismic fortification category is class B, requiring a more stringent seismic detail requirement;

The wind loading for the main tower is determined by wind tunnel testing with wind speed of 100-yr and 50-yr return period for displacement and strength checking respectively (Arup, 2010).

### 4.2. Analysis

For the elastic analysis, ETABS and MIDAS were used. For elastic-plastic analysis, LS-DYNA was adopted while ABAQUS was selected by the independent checking engineer.

Composite steel plate wall and multi-cellular concrete filled tube column are modelled while dimensional effects have been considered in setting out the geometry of mega brace and transfer truss. The second-order (P-Delta) effect is also considered in the analysis.

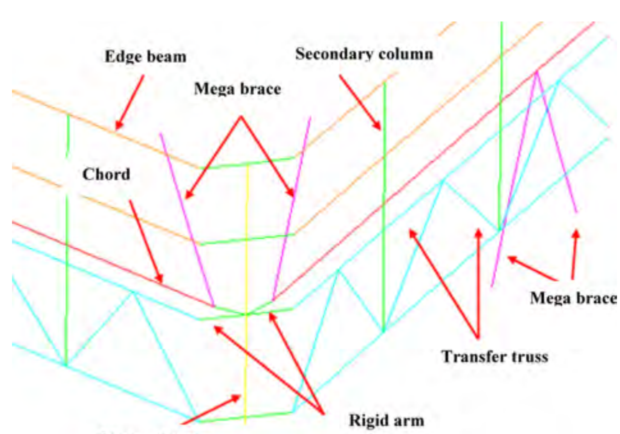


**Figure 10.** Introduction of rigid arm in analysis model to model dimensional effects and eccentricity.

The first to third mode of the structure has a period of 9.06s, 8.97s and 3.46s, with the first two modes being translational the the third being torsional mode. The ratio of the first torsional mode to first translational mode is 0.38, which is significantly smaller than the Chinese code limit.

### 4.3. Overturning moment, storey shear and shear-gravity ratio

The distribution of shear under frequent occuring earthquakes and wind is shown below. Seismic load case is found to be the primary control action. One reason for this is that the seismic story drift has to be scaled according to the minimum base shear, which is finally larger than the wind load.

### 4.4. Displacement

As shown in Fig. 12, the maximum inter-storey drift is 1/667 in 50-year return period wind loading, while inter-storey drift under frequent occurring earthquakes, magnified per allowable shear-gravity ratio and considering vertical earthquake response, is 1/516 (1/614 before magnification) at Level 97. The inter-storey drift limit in Chinese code for this type of building is 1/500.

As seen from the displacement curve under earthquake and wind, as in Fig. 13, the tower exhibits obvious flexural deformation behaviour. The maximum top displacement is 600 mm under frequent earthquake (prior to shear-gravity ratio magnification), and that under 50-year return period wind is approximately the same at 650 mm.

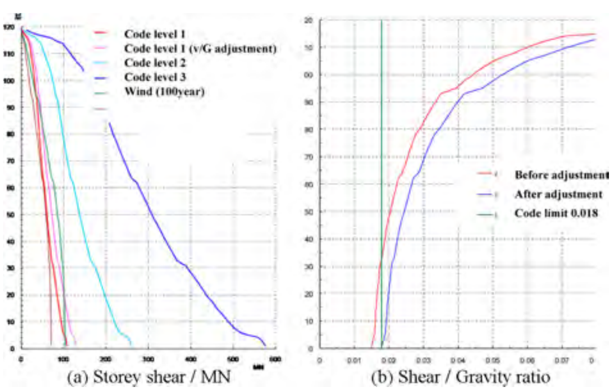


**Figure 11.** Storey shear and frequent earthquake shear-gravity ratio distribution.

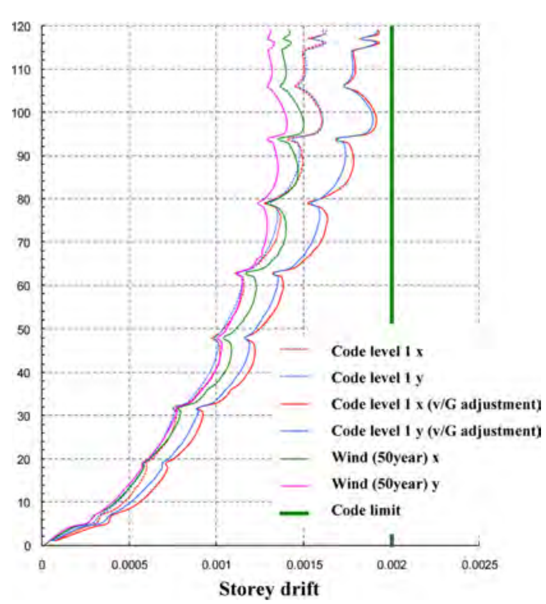


**Figure 12.** Inter-storey drift under earthquake and wind.

### 4.5. Stiffness-gravity ratio analysis

The ratio of stiffness and gravity loading of the structure (stiffness-gravity ratio) is a crucial factor influencing gravity P-$\Delta$. That means satisfying the Chinese code requirement of (JGJ-2002, 2002) $EJ_d \geq 1.4H^2\sum_{i=1}^{n} G_i$. As to the determination of $EJ_d$, according to the recommended method in the supplementary notes to the code, an inverted triangularly distributed load is applied along the full height of the tower $H$. Results indicate that $EJ_d/H^2 \sum_{i=1}^{n} G_i = 1.44$, which is within the [1.4,2.7] range, satisfying the lower limit of stiffness-gravity ratio. As the value is less than 2.7, according to the code, undesirable effects of second order gravity response are considered in internal force determination and deformation calculation. Analysis results indicate that stiffness-gravity ratio and overall deformation are the key controlling factors to the stiffness of the tower.

### 4.6. Elastic time-history analysis of level 1 seismic

Seven sets of strong earthquake acceleration records have been adopted in the frequent seismic dynamic time-


**Figure 13.** Displacement under earthquake and wind.

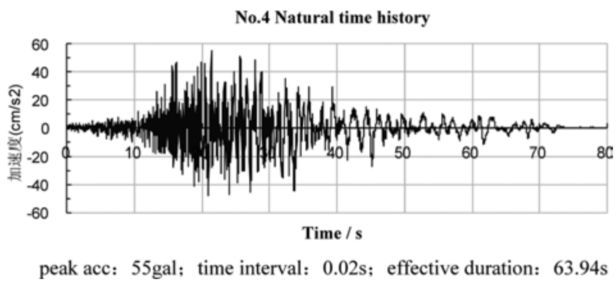


**Figure 14.** Frequent earthquake elastic time-history natural wave 4 (major direction).

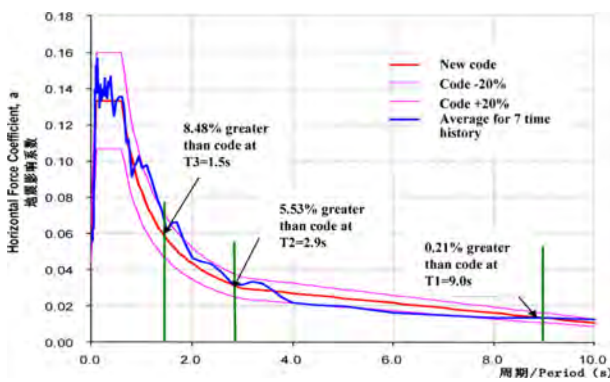


**Figure 15.** Corresponding horizontal force coefficient of frequent earthquake time-history compared with code response spectrum.

history analysis, in which 2 sets are artificial record and 5 sets are natural record. In the selection of records, requirements include not only peak acceleration, duration, spectral characteristics, but also those associated with base shear and high mode responses.

Storey shear obtained from elastic time-history analysis in both X and Y direction is as shown in Fig. 16. All 7 sets of records have base shear greater than 65% of that obtained from response spectrum analysis, with an average value greater than 80% of that from response spectrum, satisfying the code requirements. In the structural design an average value of the storey shear has been used to magnify the story shear obtained by response spectrum analysis.

### 4.7. Shear and overturning moment distribution between internal and external frame

The external tube is carrying over 70% of the storey shear on typical floors, considerably greater than that taken by the core (about 30%). At the strengthening floor (where transfer truss locates), the sudden increase in stiffness of the external tube results in a sharp increase in shear force taken by the external frame with a horizontal force transfer between the internal and the external. Shear force taken by mega columns thus shows a reverse sign.

About 80% of the storey overturning moment is taken by the perimeter structure.

These distributions of shear and overturning moment

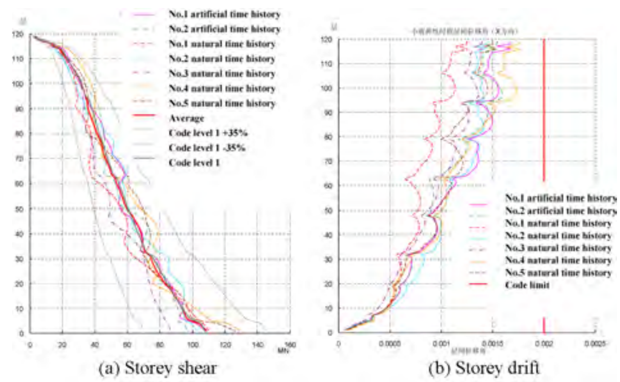


**Figure 16.** Elastic time-history and response spectrum shear distribution comparison.

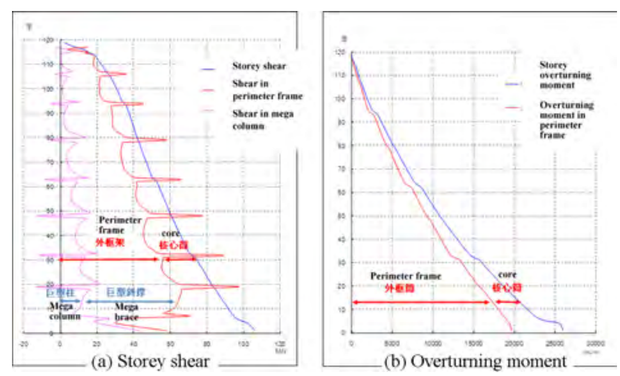


**Figure 17.** Distribution of shear and overturning moment between internal and external frame.

between internal and external tube indicates that the perimeter mega-structure provides major stiffness while the internal core becomes a "secondary" system. The advantage of such arrangement is that the core can be designed for a rather lower demand.

Based on the characteristics of the project, C-SPW and cast-in steel posts have been introduced to the lower part of the core and critical location on other levels, significantly enhancing the ductility of the reinforced concrete core. Regarding the perimeter structure, elements have been designed for higher-than-code demand level.

### 4.8. Wind comfort analysis

Wind tunnel tests for the tower were independently completed by BMT Fluid Mechanics and Shantou University Wind Tunnel Laboratory. Results from both laboratories are in line and indicate an estimated peak acceleration of 0.203 m/s² at the highest occupied level, which satisfies the national code requirements.

## 5. Elastic-Plastic Time-History Analysis of Level 3 Seismic

To achieve the seismic performance objective of no collapse under severe earthquake, the design adopts the member plasticity development limits and analysis methods and procedures suggested in FEMA356 (FEMA 356, 2000) and ATC40 (ATC40, 1996).The non-linear seismic analysis was carried out with the general non-linear dynamic finite element analysis software LS-DYNA considering geometric non-linearity as well as material non-linearity.

Elements like beams and columns adopt finite element models for section capacity, bending-curvature curve and deformation limit calculations. Shear wall is modelled by non-linear shell element, with non-linear beam elements at two ends of wall to model the hidden-columns. As for the composite steel plate wall, the steel plate and reinforced concrete shear walls are separately modelled as non-linear shell elements in space with common nodes, ensuring steel plates and reinforced concrete work in coordination. Parameters like overall mass and period of the elastic-plastic model under severe seismic activity are calibrated with ETABS results to ensure the elastic compatibility of the dynamic properties.

Seismic time-history records adopt 5 sets of natural records and 2 sets of artificial records. The overall structural performance is assessed by

- elastic-plastic inter-storey drift,
- shear-gravity ratio,
- top displacement of the structure
- time-history of base shear,
- development of plastic hinges
- location of plastic hinges.

Assessment of element is done by the comparison of the plastic deformation and its deformation capacity, as well as condition of plastic deformation of critical element at critical locations. Figure 18 indicates the inter-storey drift response in X and Y direction of the 7 sets of time-history records under severe seismic activity, with all of them satisfying the code requirement of 1/100.

## 6. Design Study and Recommendations

### 6.1. Overall stiffness control and selection of structural form

Structural height of the tower reaches almost 600 m. The overall stiffness under wind and earthquake becomes one of the most important issues in structural design. The stiffness-gravity ratio, shear-gravity ratio, inter-storey drift and peak acceleration of top level under wind (comfort) govern the overall structural stiffness. Major design effort has been put in to reducing the structure self-weight and improve the structural efficiency. Steel-reinforced concrete composite structure is adopted for the overall structure, effectively putting steel and concrete in composite action, maximising the technical advantages of steel structure while enjoying the relatively low cost of concrete, with high structural stiffness and good fire proofing resistance. All these characteristics are advantageous when compared with pure steel structure, allowing the best economical and reasonable structural form.

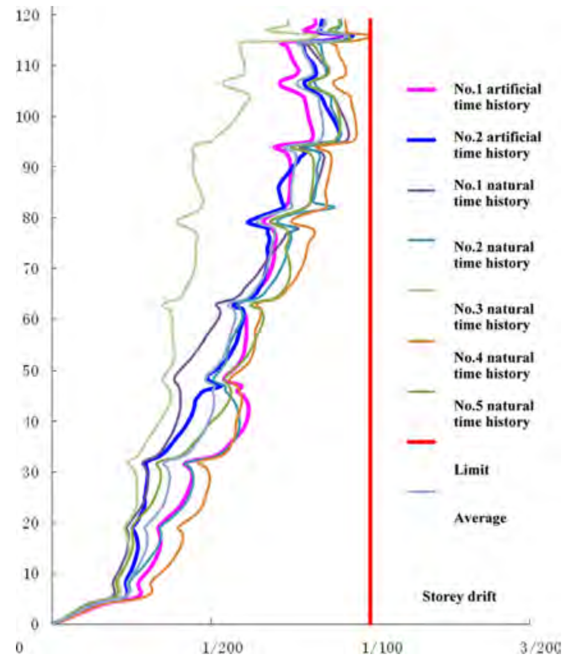


**Figure 18.** Overall inter-storey drift under severe earthquake elastic-plastic time-history analysis.

### 6.2. Selection of response spectrum

Velocity and displacement of ground motion may introduce greater damage to structures with long period (Yu, 2002; Zhao and Wang, 2003). With this kind of long period structures, acceleration response spectrum for seismic analysis is difficult to achieve design requirements. However, considering the existing technology available to design practice and approval process, elastic acceleration response spectrum is still the commonly adopted method for seismic design. The current code does not specify clearly the descending curve of the seismic response coefficient in the acceleration response spectrum after 6s. Following an extensive review on the behaviour of long period response spectrum, with the support from Chinese experts, a new extended version of up to 10s response spectrum were proposed for this project. However, high-rise building with extra-long fundamental period is not sensitive to ground acceleration, but the frequency content of the selected time-history records and the displacement. Therefore, the major focus on the calculations should be the time history analysis instead of response spectrum analysis.

Based on discussions with expert panel, the codified base shear-gravity ratio limitation was slightly adjusted and used in the design.

### 6.3. Load path and progressive collapse analysis of mega structure

Mega-structures have a smaller number of columns and relatively less redundancy comparing with traditional structural systems, requiring a clear load path in structural design and robustness under earthquake and other accidental incidents (for example blast, fire, collision) to avoid progressive collapse and inaccuracy in analysis introduced by an unclear load path.

In extreme condition, transfer trusses can offer vertical support from below as well as hanging support from above the gravity load of sub-frames in 2 zones. This method adds an alternative load path without obvious cost increase, improving the safety of structure under extreme conditions.

The impact of local damage of mega brace and effects of wind and accidental load during installation to progressive collapse have been considered. Design checks have been carried out making reference to relevant requirements in American DOD (Department of Defense) and GSA (General Services Administration) as well as the new national code for high-rise building design.

Currently there is no specific technical guidance for the prevention of progressive collapse in China. It is worth discussing and further studying the potential problems and suggested solutions for mega frame structural systems.

### 6.4. Material selection

Analysis revealed that 30% of seismic mass originates from reinforced concrete core, and axial stress ratio becomes the controlling criteria for shear wall along the height of the building. Considering ductility requirements the concrete grade is not higher than C60 for the shear wall, as a result steel plates were added to act as a composite wall which has an increased stiffness, shear resistance and ductility. Consequently the C-SPW is becoming popular for high-rise buildings above 300 m tall in China, since its first adoption in China World Trade Centre Phase 3A, Beijing.

### 6.5. Selection of structural form for external frame

As the mega tower adopts a mega-frame plus cross mega brace structural form, the overall stiffness of the external frame is significantly greater than the internal tube, leading to a weak internal tube, strong external tube structural system.

Increase of the external frame stiffness lead to similar flexural deformation pattern for both the perimeter structure and the internal core. Outriggers are thus eliminated due to this deformation consistence.

### 6.6. Performance-based design

For seismic design of code-exceeding buildings in China, code-based prescriptive method, or "seismic concept design", is still the major design method which is supplemented by the verification from the nonlinear elastic-plastic analysis and shaking table test for the severe earthquake (1 in 2475 yrs) scenario. Important members were identified and designed for elevated design criteria. For example, different criteria in severe earthquakes have been considered for mega columns and transfer trusses, core walls and mega braces were designed to be elastic in fortification level earthquake. In the axial-moment analysis of mega column under various internal force combinations, mega column is under compression in fortification (1 in 475 yrs) and frequent earthquake (1 in 50 yrs), as well as 100-year return period wind. Under severe earthquake, gravity force of mega column cannot balance the axial tension induced by lateral action, mega column is under tension-bending state for most height. This phenomenon happens in high-rise buildings in high earthquake intensity region after the seismic force magnification and would possibly become the controlling criteria for element design.

To minimize the adverse effects of such a response, when designing buildings with a height of over 400 m in high intensity earthquake region, reasonable load path for gravity load should be considered. As the mega frame can transfer gravity force of each zone through transfer truss to mega column, earthquake component has been largely balanced. This becomes a necessary guarantee for selection of structural form during design development stage.

In frequent earthquake design, multiplication of element internal force adjustment factors including strong column weak beam, time-history analysis, shear-gravity ratio, soft storey, external frame shear ratio ($0.2Q_0$), internal force combination factors, combined with wind combinations may results in design force greater than fortification level earthquake design in normal circumstance. This reveals the inherent inadequacy of adjustment using parameters based on conceptual design.

## 7. Conclusion

For the structural design of the slender tower of Tianjin Goldin Finance 117, the Chinese codes together with the prescriptive performance-based design principles guided the whole design process. Extensive linear and non-linear spectrum-based or time-history-based analyses have been carried out for different levels of earthquake as well as wind loading levels to justify the performance of the structure against those criteria. There is still distance towards an actual performance-based design which asks for many changes in the code system and also a broader understanding and acceptance of the advanced analysis in the industry. However, we understand it as a pragmatic approach in current environment in China which is in the fast-track of the construction of super high-rise buildings.

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