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Tianjin Goldin Finance 117 Tower: The Solution to a Slender Geometry

天津高银117大厦：细长体型的结构解决方案

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Abstract

With an architectural height of 597m, the Tianjin Goldin Finance 117 tower will have the highest structural roof of any building under construction in China, and will have a structural height-width ratio of approximately 9.5, making it very slender. To satisfy earthquake and wind-resisting requirements, the structure consists of a perimeter mega-braced frame and reinforced concrete core with composite steel plates. Based on the new requirements from the latest Chinese building seismic design codes, the design includes a number of new features and solutions in overall stiffness control, material and component type selection which are further evaluated by seismic performance-based design, mega-column design, robustness analysis as well as elastic-plastic time-history analysis. The design overcomes various structural challenges and satisfies the requirement of the architect and the client.

Keywords: Mega Frame, Seismic Zone, Mega Column, Composite Steel Plate Shear Wall

摘要

天津高银117大厦建筑高597米，是中国在建的屋顶高度最高的建筑物，结构高宽比达到约9.5，使其形态非常纤细。为满足抗震与抗风的技术要求，结构采用了含有巨型组合柱的外框架以及含有组合钢板混凝土混合的结构体系。结合新的抗震规范要求，在整体刚度控制、材料与构件选型、性能化设计、巨型柱设计、防倒塌及稳健性分析、弹塑性动力时程分析等方面均体现了许多新的特点和设计要求。在克服了结构设计种种挑战的同时，成功实现了建筑师及业主方的设计意图。

关键词：巨型框架、地震带、巨型柱、组合钢板剪力墙

Engineering Background

Measured from the structural roof level, Tianjin Goldin 117 will be the tallest building in China. The tower is located in Tianjin, China, and will include class-A office spaces, a 6-star hotel and ancillary facilities with a gross floor area of approximately 370,000m^2. The architectural height is approximately 597m with 117 stories (total of 126 structural stories). The project is financed by Goldin Properties Holdings Limited.

The mega tower has a square plan which reduces in size throughout the height and follows a tapered shape in elevation. The plan dimension is approximately 65m x 65m at the ground level and gradually reduces to 45m x 45m at the roof level (see Figure 1).

Design Challenges

The structural height of the tower is 584 meters and the height-to-width ratio is approximately 9.5, significantly exceeding the limit of 7.0 imposed by Chinese seismic code. Accordingly, approval from a National Expert Review Panel is required.

工程学背景

高银117大厦从结构屋顶高度测量将会成为中国第一高楼。此项目位于天津市，为一幢写字楼为主附有六星级酒店及相关设施的大型超高层建筑。总建筑面积约37万平方米，建筑高度约为597米，共117层（地上结构楼层共126层），由天津海泰新星房地产开发有限公司投资开发。

巨塔平面为正方形，外形随高度变化，各层周边建筑轮廓随着斜外立面逐渐变小，塔楼首层建筑平面尺寸约65米×65米，渐变至顶层时平面尺寸约45米×45米（见图1）。

设计挑战

塔楼结构高度为584米，高宽比约9.5，大大超过中国抗震规范7.0的限值要求，因此必须通过国家专家审查。

由于天津处于中国北方地震高烈度区（7度0.15g），且所在场地覆盖层较松软，根据中国规范要求必须采取更为严格的控制标准，因此结构抗震设计面临更为严峻的技术要求和条件。

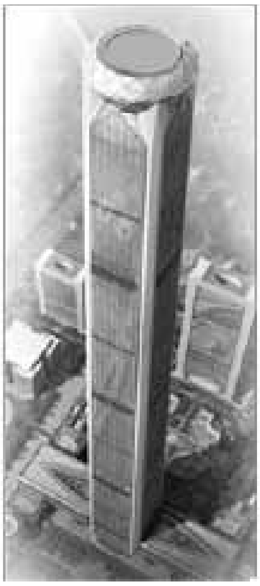

Figure 1. Perspective view (Courtesy of P&T Group)
图1. 整体效果图（巴马丹拿集团提供）

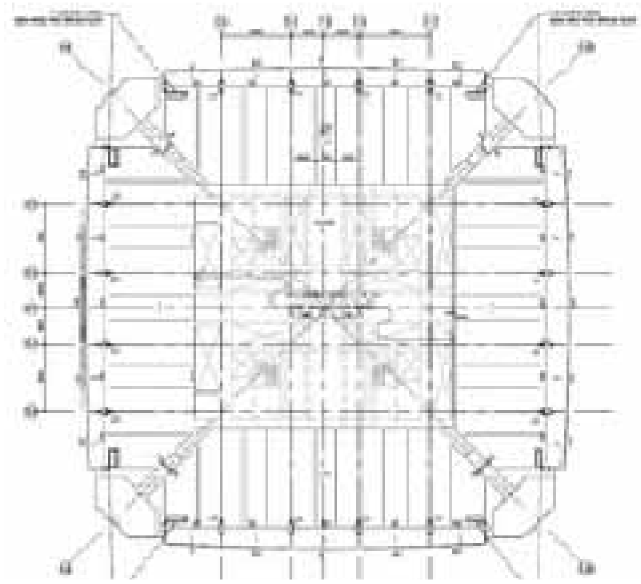

Figure 2. Structural Arrangement Plan of Ground Floor
图2. 首层结构平面布置图

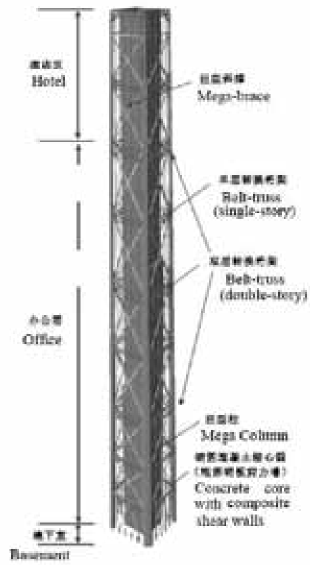

Figure 3. 3D Illustration of Structural System
图3. 结构体系三维示意图

Tianjin is located in Northern China with high seismic intensity (Intensity 7, 0.15g) with soft ground conditions. 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 the seismic design of the structure.

Development of Structural Systems and Member Design

Building Plan

By coordinating with the architect and the service engineer throughout the design process, an almost bi-axially symmetrical arrangement has been achieved for the structural plan and core, as shown in Figure 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 requirements imposed on the mega tower by its height-to-width ratio led 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 a series of technical structural requirements for seismic and wind loading.

Since the stiffness of the perimeter mega frame exceeds that of the core in most of the stories, outrigger trusses were determined to be inefficient in improving overall structural stiffness. Thus, outrigger trusses were not adopted in the final design.

The final structural stability system, as shown in Figure 3, comprises a reinforced concrete core and a perimeter mega braced frame to form a dual structural stability system (see Figure 4). This system provides superior lateral stiffness and safely resists earthquake and wind loading.

The urban planning authority and the client preferred to reduce the visual impact of the cross-bracing. The braces were thus offset inboard from the perimeter beam-column sub-frame which also added the benefit of simplifying the gravity load path (see Figure 5).

结构体系与构件设计开发

建筑平面

设计过程中通过与建筑师和机电专业工程师的不懈协调，最终实现了结构平面及核心筒几乎双轴对称的格局，如图2所示。

周边结构

为实现建筑布局并确保结构安全，结合工程经济性充分发挥钢与混凝土两种材料的优势。塔楼高宽比对巨塔的要求迫使采用更为高效的支撑布置形式。经过与业主及建筑师协调，除底部节间考虑建筑主入口的要求为人字支撑外，其余节间采用交叉撑的形式，此举明显提高了结构整体刚度，最大程度地发挥了构件效率，从而满足了结构抗震及抗风的一系列技术要求。

由于外框架刚度在大部分楼层超过了钢筋混凝土内筒，因此伸臂桁架对于提高结构整体刚度的作用不明显，最终予以取消。

最终的结构稳定体系，如图3所示，是分别由钢筋混凝土核心筒，带有巨型支撑筒、巨型框架构成的周边结构构成的多重结构稳定体系（见图4），提供了强大的侧向刚度，共同抵抗水平地震及风荷载。

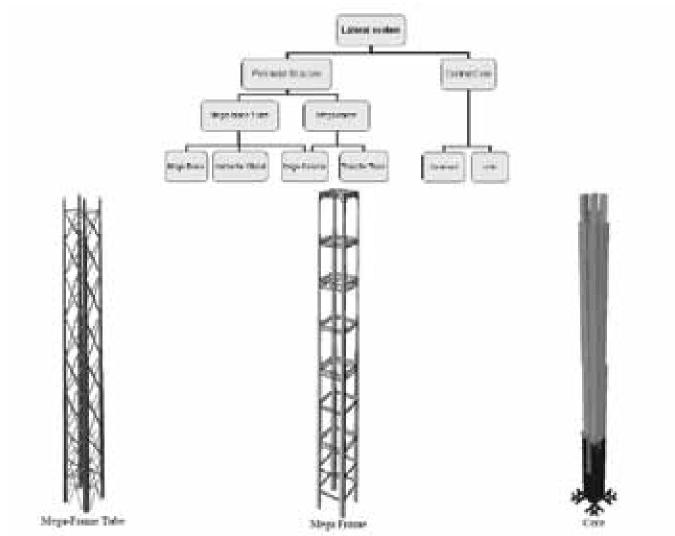

Figure 4. Dual Lateral Stability System
图4. 多重抗侧力稳定体系

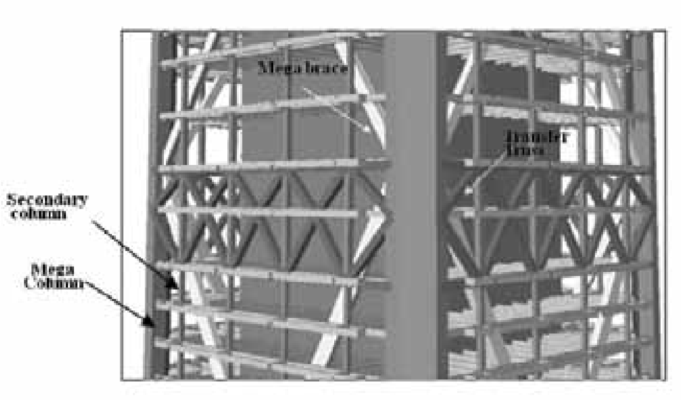

Figure 5. Illustration of Mega Frame and Mega Brace connection – double-story truss
图5. 巨型框架与巨型支撑连接示意图 – 双层桁架

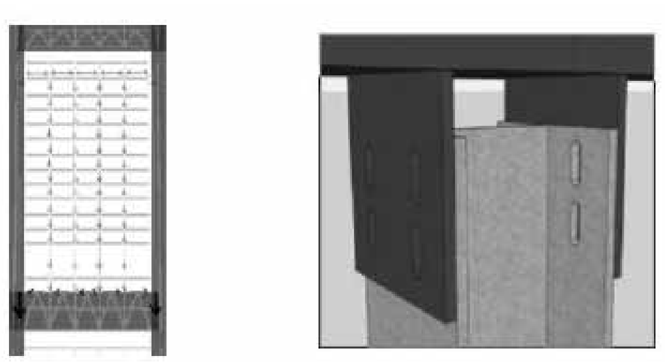

Figure 6. Elevation of perimeter frame and slotted joint at the top
图6. 周边框架立面布置图和顶部滑动连接节点

To prevent potential progressive collapse if the lower part of the columns of the sub-frame is damaged, 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 will be activated if a lower column were ever to fail (see Figure 6). The belt truss is designed for high horizontal/vertical seismic load combination demand, and this simple structural detail enhances the robustness and safety of the gravity system without extra cost.

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 with dimensions of about 34m x 32m at the base, and an approximate square on Level 67 up to the roof.

The core adopts steel-reinforced concrete shear walls with embedded steel sections. At the low zone, alongside 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, as shown in Figure 7. This system has been widely adopted for supertall buildings in China since its first introduction in the China World Trade Centre Phase 3A project. Once the wall is proved to be strong in shear, the ductility of a reinforced concrete shear wall can be guaranteed. At the same time, this kind of C-SPW will not induce any sound during oscillation like a pure steel plate shear wall. The use of this type of composite wall increases the compressive and shear capacity of the element, efficiently reducing the self-weight of the structure and hence the mass.

The core wall thickness gradually reduces from 1400mm thick at the base of the tower to 300mm at the top. Steel plate arrangements within the wall panels vary from two 35mm thick steel plates at the base to a single 25mm steel plate at about Level 32.

Mega-Columns

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. The plan shape of the mega columns satisfies the architectural profile and structural connection requirements, resembling a six-sided polygon with a cross sectional area of about $45m^2$ at the bottom (see Figure 8). The mega columns reduce in size at zones along the height of the tower with the exterior face of the columns held flush.

The mega columns are connected to the transfer trusses and mega braces. Initial designs of the columns were envisioned to be steel reinforced columns (SRC), however it was determined that polygonal concrete filled tubes would ultimately be required. The normal practice

由于城市规划部门和业主弱化交叉支撑视觉效果的要求，因此设计中采取了将斜撑与周边次框架在平面上错开的方案，这还可简化重力荷载路径（见图5）。

为防止可能的连续倒塌，一旦副架的下层柱遭到破坏，上部需有另一条传力路径，通过长圆孔节点来连接柱体和带状桁架，一旦下层柱失效，节点则会被激活（见图6）。带状桁架设计用于横向/竖向高地震荷载的综合要求。而且这一简单的构造在不增加额外成本的同时，还提升了重力系统的鲁棒性与安全性。

核心筒

核心筒从承台面向上伸延至大厦顶层，贯通建筑物全高，其平面基本呈长方形。底部尺寸约为34米×32米，直至核心筒于层67完全呈现正方形。

塔楼核心筒采用内含钢骨的型钢混凝土剪力墙结构，并在下部采用内嵌钢板的组合钢板剪力墙结构，以防止大震下的剪切破坏，如图7所示。此体系自北京国贸三期在国内首次采用后在超高层建筑中得到了广泛的应用。一旦证明墙体的抗剪强度很高，可以改善普通混凝土墙的延性，同时组合墙体也不像纯钢板墙在受力时可能发出声音。组合墙体的采用提高了构件抗压、抗剪承载力，有效降低结构自重及质量。

核心筒周边墙体厚度由1400mm从下至上逐步均匀收进至顶部300mm；墙体内的钢板布置由底部的两块35mm厚钢板到约层32处的单块25mm钢板。

巨型柱

巨型柱位于建筑物平面四角并贯通至结构顶部，在各区段分别与水平杆、转换桁架及巨型斜撑连接。其平面轮廓结合建筑及结构构造连接要求，呈六边菱形，底部截面约为$45m^2$（见图8），沿高度并配合建筑要求分多段内收，柱体外侧平齐。

巨型角柱与转换桁架及巨型斜撑连接，其设计也经过了不断的演化，特别是对型钢混凝土柱和钢管混凝土柱之间进行了各方面的比对，权衡利弊。根据相关试验结论，柱内各孤立的钢骨间必须采取全高的强连接的方式，避免出现类似格构柱的分离式的布置，确保柱的整体延性。最终考虑将钢板在周边外置，内部钢板根据构造要求相互连接，独立分割，如图所示，形成了多腔体的6边形钢管混凝土组合构件，获得巨大的拉压弯及抗剪扭承载力，以抵抗竖向荷载及风、地震产生的侧向荷载。

巨形柱非节点区整体含钢率约为4%~6%，钢材采用Q345GJ(或Q390GJ)。由底至顶内填高强混凝土，强度C70~C50。各腔体内布设钢筋，在提高构件强度的同时，有效降低混凝土收缩徐变产生的不利影响。在各腔体内侧对称布设纵向内肋板，并用水平拉结钢筋连接，约束钢板面外屈曲。

巨型柱结构设计，综合平衡了建筑布局、结构整体刚度、构件受力性能、节点连接、工程造价、制作加工、施工可行性等各方面

of discrete steel sections in SRC columns was 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. The final design is an external steel plate enclosure with internal inter-connected plates forming separate chambers in accordance with the detailing requirements (as shown). The six-sided polygonal concrete filled tube composite member has sufficient capacity to resist axial, bending and shear forces generated by earthquake and wind loads.

The overall steel percentage of the mega columns outside the connection zone is about 4~6%, using Q345GJ (or Q390GJ) grade steel with high-strength concrete infill of grade C70~C50. Reinforcing steel is distributed within each compartment, enhancing the strength of the member while 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 restrain out-of-plane buckling of the steel plates.

The structural design of the mega columns was a compromise between various factors including architectural arrangement, overall structural stiffness, element performance under loading, connection design, construction cost, production and constructability, and achieving the best overall economic and technical performance.

Mega Brace and Transfer Issues

The mega braces are arranged on the four elevations of the tower, using welded steel box sections and are connected to the mega columns. Since the mega braces are separated from the perimeter beam-column frame, lateral support was provided for in the floor system to restrain out of plane buckling of the mega braces.

Transfer trusses were coordinated with architectural and services requirements and were located at mechanical and refuge floors. There are nine sets of trusses, distributed evenly at approximately every 12 to 15 floors. The transfer trusses resist the gravity loading from each zone and transfer the loads to the corner mega columns. The transfer trusses also create a frame with the corner mega columns, enhancing the torsional stiffness of the tower. Under severe seismic activity, the transfer trusses are a vital component of the structural system in preventing progressive collapse of the floors and ensuring safety. Vertical earthquake action has also been considered for the long-span trusses and its performance criteria has been increased to prevent yielding under severe seismic activity.

Floor System

Outside the core of the tower, a composite floor system is adopted with simply supported steel beams that are spaced at 3 meters on center with spans ranging from 6 meters at the top of the tower to 13 meters at the bottom of the tower. The office slab floor is 120mm thick and the hotel floor is 130mm thick including the metal decking.

To ensure reliable transfer of diaphragm forces between the core and external frame, the main tower and the podium wings, and the main structure and the basement, specific floors were strengthened by thickening to 200~300mm and by introducing in-plane bracing.

Foundation System

Underneath the 4-story 26-meter deep basement, the tower is supported by a 6.5m thick raft which is in turn supported by 941 cast-in-situ bored piles. The raft is 86m x 86m in plan with C50 concrete strength. The piles are one meter in diameter and are founded at 100 meters below ground, while the effective length is approximately 76 meters. Post pressure grouting for pile shaft and toe was used to increase the pile capacity and reduce the settlement. Piles were zoned

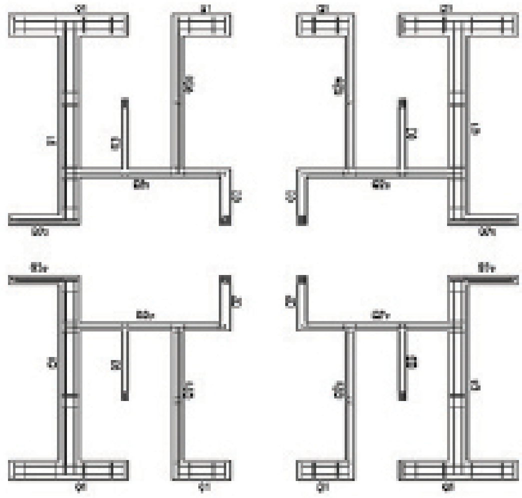

Figure 7. Diagram of composite steel plate shear walls.
图7. 复合钢板剪力墙图表

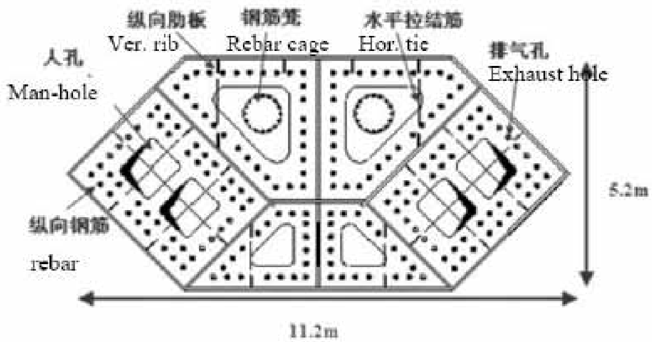

Figure 8. Typical low zone 45m² Mega Column section detail
图8. 底部典型楼层巨型角柱45m²截面构造示意图

的要求，达到最优的综合经济技术性能。

巨型支撑和转换桁架

巨型支撑设置于大厦四边的垂直立面上，采用焊接箱形钢截面并与巨型柱连接。巨型斜撑与边梁柱相互脱开，为楼面系统提供了侧向支持以控制巨型支撑平面外的屈曲。

转换桁架配合建筑及机电专业要求，设置于避难及设备层。由9组沿塔楼每12~15层均匀分布。转换桁架承担其间隔楼层竖向荷载并将其转换至角柱，并与四角的巨型柱共同作用，提供部分抗侧刚度，增加大厦的抗扭性能。在罕遇地震下，转换桁架将成为防止楼面局部倒塌，确保安全的重要构件，还考虑了大跨结构竖向地震作用，提高其性能化设计水准至大震不屈服。

楼板体系

塔楼核心筒外，楼面梁采用了常见的组合楼板体系，间距为三米的简单支撑钢梁，跨度由高至低约为6～13m，两端铰接，钢梁典型间距为3m，包括金属面板在内的总楼板厚度办公楼层为120mm、酒店楼层为130mm。

为确保水平剪力在核心筒与外框架、主塔楼与裙楼间以及主体结构和结构大底盘之间的可靠传递，楼板固层厚度为200～300mm。

基础体系

结构共有4层地下室,埋深约26米.塔楼采用钻孔灌注桩-平板式筏板基础,筏板尺寸为86m*86m,厚度6.5m,混凝土强度C50.塔楼下总桩数941根,有效桩长约76m, 桩径1m,混凝土强度C50.为进一步提高单桩承载力并控制沉降,桩侧桩底采用了后压浆,结合塔楼下不均匀桩反力分布,单桩设计承载力采用分区设计,巨型柱下最大单桩承载力特征值达到1650吨以满足抗震和抗风的设计要求。

with different design capacities in which the maximum characteristic value of single pile capacity is 16500kN.

Elastic Analysis of Overall Performance

Basic Parameters

The design reference period and working life recurrence interval is 50-years, and the durability design recurrence interval is 100-years. The seismic fortification intensity is 7.0 with a design peak acceleration of ground motion at 0.15g.

The wind loading for the main tower is determined by wind tunnel testing with wind speeds of 50-year and 100-year return periods for displacement and strength checking, respectively.

Seismic Performance-Based Design Requirement

Since the structure exceeds prescriptive code requirements 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. Elasto-plastic time-history analysis was used to confirm that the seismic performance objectives were met for severe earthquakes.

Elastic 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 review engineer.

The first three modal periods of the structure are 9.06s, 8.97s and 3.46s, with the first two modes being translational modes and the third being a torsional mode.

Overturning Moment, Story Shear and Shear-Gravity Ratio

The distribution of shear under frequently occurring earthquakes and wind is shown below. The seismic story drift, scaled according to the minimum base shear, was found to be larger than the wind load. Accordingly, it was determined that seismic loads controlled the design (see Figure 9).

Displacement

As shown in Figure 10, the maximum inter-story drift is 1/667 for a 50-year wind event, while inter-story drift under frequently occurring earthquakes, magnified per the allowable shear-gravity ratio and considering vertical earthquake response, is 1/516 (1/614 before magnification) at Level 97, which is within the 1/500 limit of the Chinese code.

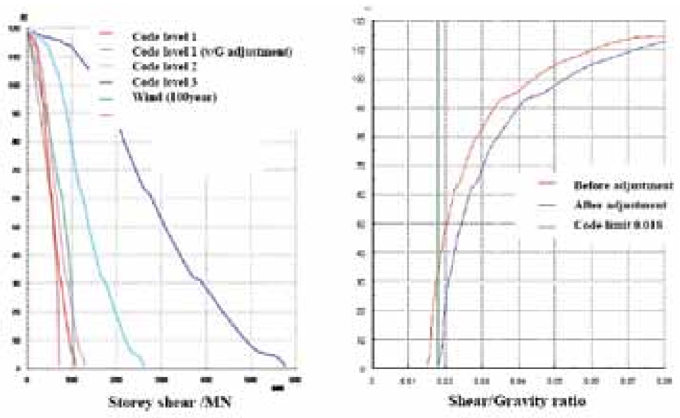

Figure 9. Story shear and frequent earthquake shear-gravity ratio distribution
图9. 楼层剪力和小震剪重比分布图

整体性能弹性分析

基本参数

塔楼结构设计基准期及设计使用年限为50年，耐久性为100年，建筑抗震设防烈度7度，地震加速度为0.15g。

塔楼主体结构风荷载的确定，按照“强度控制按100年规范风速风洞试验，位移控制按50年规范风速风洞试验荷载”原则进行。

抗震性能化设计要求

由于结构超限较多，按照性能化设计的思想，经过与超限审查专家组的多次讨论，明确了结构整体和各构件抗震目标。对于弹塑性时程分析明确了大震下的抗震性能目标。

弹性分析

弹性分析采用了ETABS和MIDAS。弹塑性分析除采用LS-DYNA，同时应用ABAQUS进行了第三方的比对分析。

结构前三个自振周期分别为9.06，8.97，3.46s，前两振型均为平动，第三振型为扭转。

倾覆弯矩、层剪力和剪重比

小震和风荷载作用下底部剪力如下所示。结构地震下的位移根据最小底部剪力进行了调整，大于风荷载下的位移。因此结构的侧向刚度主要受地震荷载控制（见图9）。

位移

如图10所示，采用50年一遇的风荷载，最大层间位移角为1/667，小震下层间位移角计算考虑了剪重比和竖向地震放大的影响，最大层间位移角为1/516（放大前为1/614），均处于建筑层97，在中国设计规范限制的1/500以内。

小震弹性时程分析

小震分析采用了七组强震加速度记录作为动力时程分析，其中两组为人工波，其余五组为天然波。七条时程曲线的基底剪力均大于反应谱法的65%，平均值大于反应谱法的80%，满足规范要求。结构设计取时程波在各层的平均值，在反应谱基础上将内力值进行放大调整，进行构件的补充验算。

内外框架剪力与倾覆力矩

外框筒承担了标准楼层约70%以上的剪力，明显大于核心筒（见图11）。在结构加强层，由于外框筒刚度在该楼层的显著增大，导致外框筒吸收的地震剪力出现突变，同时伴随水平力在内外筒间进行传递。

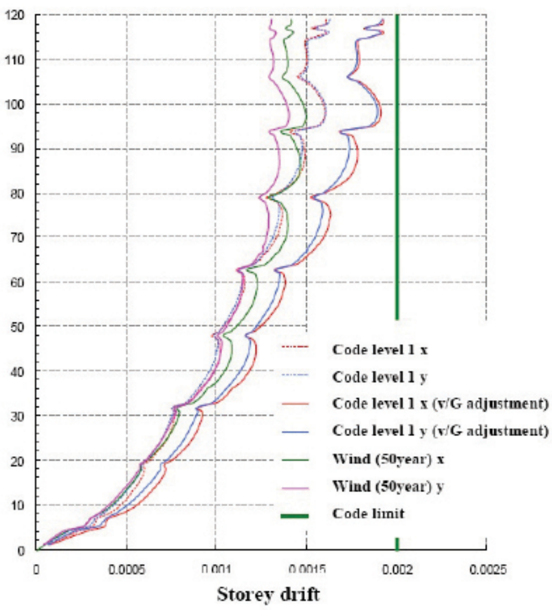

Figure 10. Inter-story drift under earthquake and wind
图10. 地震及风作用下层间位移角

Elastic Time-History Analysis of Level 1 Seismic

Seven sets of earthquake acceleration records have been studied in the frequent seismic dynamic time-history analysis, in which two sets are artificial records and five sets are natural records. All seven sets of records have base shears greater than 65% of that obtained from the response spectrum analysis, with an average value greater than 80%, which satisfies Code requirements. In the structural design, an average value of the story shear has been used to magnify the story shear obtained from the response spectrum analysis.

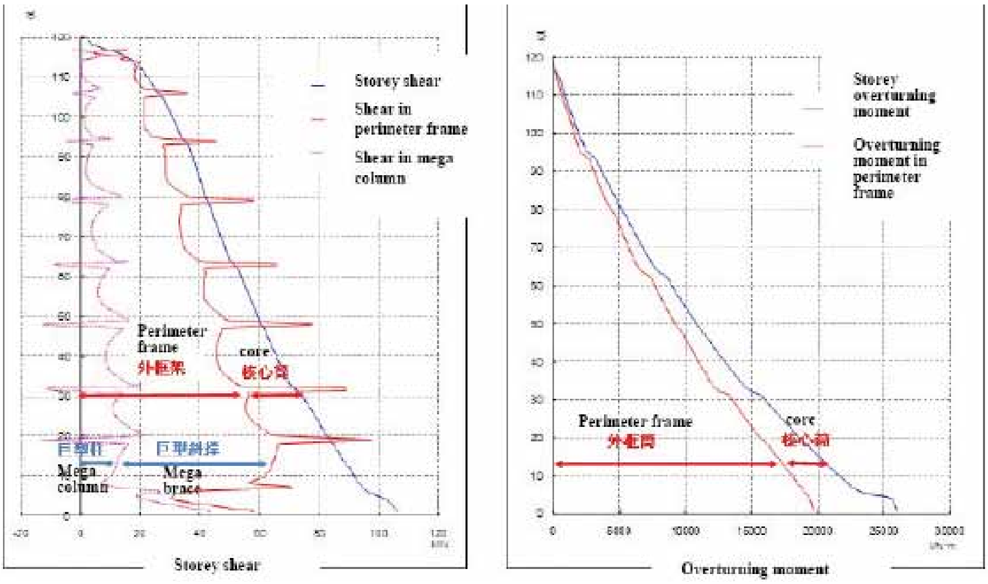

Figure 11. Distribution of shear and overturning moment between internal and external frame
图11.　剪力及倾覆弯矩在内外框架分布示意图

Shear and Overturning Moment distribution between Internal and External Frame

The external tube carries over 70% of the story shear on typical floors, which is considerably greater than that taken by the core (see Figure 11). At the strengthened floors, 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 tube.

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

These distributions of shear and overturning moments between the internal and external tube indicates that the perimeter mega-structure provides the majority of stiffness while the internal core becomes a "secondary" system. The advantage of this arrangement is that the core can be designed for a relatively low demand.

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 consistent and indicate an estimated peak acceleration of 20.3mill-g at the highest occupied level, which satisfies the national code requirements.

Axial Shortening Analysis

Tall buildings will shorten under the gravity load, elastically as well as under the effect of shrinkage and creep. An initial construction program simulation analysis was carried out to estimate the amount of the axial shortening and evaluate the additional forces incurred. The internal forces of the mega braces were found to be increased due to the shortening of the mega columns which has been allowed properly in the member capacity checking.

Elastic-Plastic Time-History Analysis of Level 3 Seismic

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 FEMA356 and ATC40. 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.

As for the composite steel plate shear wall, the steel plates and reinforced concrete shear walls are separately modelled as non-linear

外框筒分担了各层约80%的倾覆力矩。

从层剪力和倾覆力矩内外筒分担比例分析看，带有巨型支撑的巨型框架结构体系提供了大部分刚度，而内部核心筒则成为“次级”体系。这种布置的好处是对内部核心筒的抗震性能要求可以降低。

风舒适度分析

塔楼风洞试验分别由英国BMT公司和汕头大学风洞试验室完成，两个试验结果是一致的，显示本工程的最高住人楼层加速度为20.3mill-g，满足国家规范要求。

轴向缩短分析

高层建筑在重力荷载下高度会有所缩短，并且在收缩和徐变效应下也会弹性化的变化。初步的建设方案模拟分析被用于估算轴向缩短量并评估额外产生的受力。其结果显示，巨型框架的内力由于巨型柱的缩短而增大，而这些巨型柱已被进行适当的受力能力检查。

罕遇地震弹塑性时程分析

为实现在罕遇地震作用下防倒塌的抗震设计目标，工程采用了美国FEMA356（建筑抗震修复预标准及其说明)和ATC40 所提供的结构构件弹塑性变形可接受限值，以及所建议的结构非线性地震分析方法与步骤。结构的非线性地震反应分析采用了通用非线性动力有限元分析软件LS-DYNA 进行计算，考虑了几何非线性与材料的非线性。

对于组合钢板剪力墙，把钢板和混凝土剪力墙分别建成共节点的非线性壳单元，保证钢板与混凝土协同作用。大震对弹塑性整体分析模型的质量及周期振型等信息与ETABS模型相比较，确保了弹性兼容性的动力特性。

图12显示了罕遇地震下，7条时程波在弹塑性动力分析中，结构整体在X和Y方向上层间位移角的分布情况，均满足规范1/100的要求。

设计研究和建议

整体刚度控制与结构选型

塔楼结构高近600m，在风及地震作用下，整体刚度合理定量控制成为结构设计最重要的内容之一。因而刚重比、剪重比、结构层间位移角及风载顶点加速度（舒适度）这4项指标同时成为结构整体刚度的控制性因素，特别是超高层建筑“剪重比不能超越规

shell elements in space with common nodes, ensuring compatibility between the steel plates and reinforced concrete. Parameters such as overall mass and period of the elastic-plastic model under severe seismic activity are calibrated with the ETABS results to ensure the elastic compatibility of the dynamic properties.

Figure 12 indicates the inter-story drift response in the X and Y directions for the seven sets of time-history records under severe seismic activity, and each response satisfies the code requirement of 1/100.

Design Study and Recommendations

Overall Stiffness Control and Selection of Structural Form

The structural height of the tower reaches almost 600 meters. The overall stiffness of the tower, therefore, was key to resisting the demanding wind and earthquake loads, and was driven by the stiffness-gravity ratio, shear-gravity ratio, inter-story drift, and peak acceleration of the top level under wind (comfort). The major design effort was focused on reducing the structure self-weight and to improve the structural efficiency. The steel-reinforced concrete composite structural system adopted for the tower maximizes the technical advantages of a steel structure, provides superior structural stiffness and fire proofing resistance, while taking advantage of the relatively low cost of concrete. All of these characteristics are advantageous when compared to a pure steel structure, and ultimately provides an economical and reasonable structural form.

Material Selection

Analysis revealed that 30% of the seismic mass originates from the concrete core, and that the shear wall is controlled by axial stress throughout the height of the building. Considering ductility requirements, the concrete grade was kept no higher than C60 for the shear wall. As a result, steel plates were added to provide composite action and to increase overall stiffness, shear resistance and ductility.

Selection of Structural Form for External Frame

The overall stiffness of the external frame is significantly greater than the internal tube due to the cross mega brace configuration.

Similar flexural deformations for both the perimeter structure and the internal core occurred because of the increase of the external frame's stiffness, which provided further confirmation that the external frame dominated the overall tower stiffness. Accordingly, outriggers were determined to be unnecessary due to this deformation consistency.

Performance-Based Design

For seismic design of code-exceeding buildings in China, the code-based prescriptive method, or "seismic concept design", is still the major design method used which is supplemented by the verification from a nonlinear elastic-plastic analysis and shake table test for the severe earthquake event. Important members were identified and designed for the elevated design criteria. For example, different criteria in severe earthquakes have been considered for various components of the mega columns, transfer trusses, core walls and mega braces were designed to remain elastic under the fortification level earthquake. In the axial-moment analysis of mega columns under various internal force combinations, mega columns are in compression for the fortification earthquake event (1 in 475 years), the frequent earthquake event (1 in 50 years), and the 100-year return period wind event. Under severe earthquake, compression in the mega columns is overcome by axial tension induced by lateral action, and the mega columns are under a tension-bending state for most height. This phenomenon happens in high-rise buildings in high earthquake intensity regions

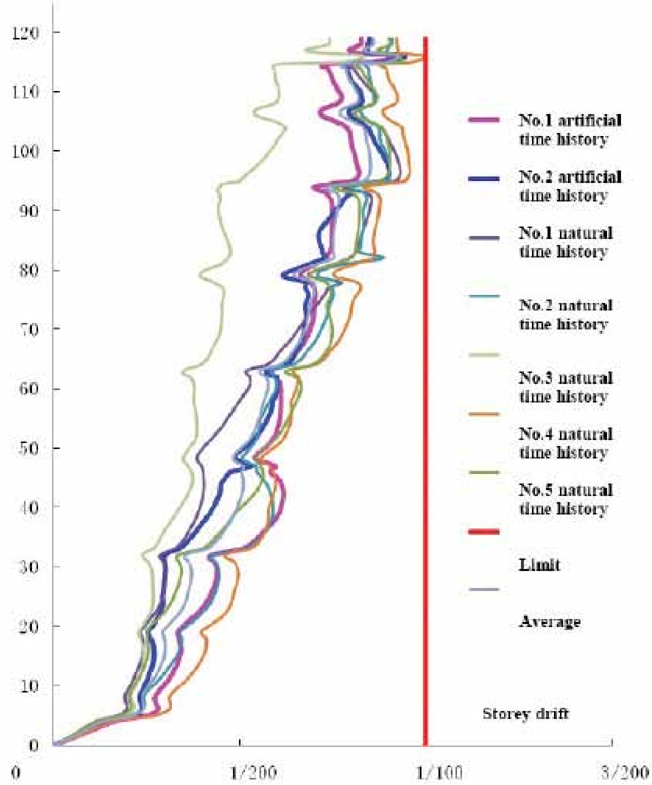

Figure 12. Overall inter-story drift under severe earthquake elastic-plastic time-history analysis
图12. 罕遇地震弹塑性动力时程分析结构整体层间位移角

范限值太多”的要求下，控制结构整体重量，采用更高效的抗侧力体系和构件布局成为设计的关键性因素。结构整体采用钢-混凝土混合结构，有效地将钢及混凝土进行组合，既具有钢结构的技术优势又具备混凝土造价相对低廉的特点，结构刚度大，防火性能好，与纯钢结构相比具有明显的优势，成为了塔楼最为经济合理的结构形式。

材料选择

经统计，结构地震质量的30%来源于混凝土核心筒，剪力墙轴压比成为核心筒全高的控制性指标。考虑一定的延性要求，混凝土材料等级被控制在C60，因此嵌入钢板的这种构造，提高了结构整体刚度、抗剪及延性等性能。

外框架结构形态选择

由于塔楼采用巨型框架+交叉支撑的结构形式，外框架整体刚度明显大于内筒。

外筒刚度的增大，使得内外筒均呈现整体弯曲变形的特征，两者变形相对协调，伸臂桁架对协调两者变形的效能降低，对结构整体刚度几乎无贡献。

性能化设计

在规范体系框架下，采用抗震概念设计并辅以罕遇地震下弹塑性时程分析和整体模型振动台试验仍旧是中国对于复杂超限建筑抗震设计主要的方法，对于重要构件的强度要求则根据特点相应采用更高的抗震性能目标。例如本塔楼巨型柱、转换桁架要求满足大震性能，而核心筒（拉压弯）和巨型斜撑则按照中震性能进行复核。对于巨型柱的拉压弯构件校核，则分别考虑了小震、中震以及百年风作用不同的内力组合，而在大震下，结构自重不能平衡地震作用产生的轴拉力，因而使巨型柱在大部分高度范围内出现受拉工况。随着地震设计内力水准的显著提高，高震区超高层建筑中拉弯作用有可能成为相关构件设计的控制性因素，因而巨型柱的型钢和钢筋同时根据校核结果设计抵御此轴拉作用。此外

after the seismic force magnification and would possibly become the controlling criteria for element design. The steel section and rebar have been checked to resist this tension force and relevant stiffness degradation is modelled in a non-linear analysis.

在非线性分析中，同时考虑了巨型柱内由于混凝土受拉开裂构件刚度退化产生的相应影响。

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 entire design process. Extensive linear and non-linear spectrum-based and time-history-based analyses have been carried out for different levels of earthquake events as well as wind events to ensure that the structure meets performance objectives. There are still obstacles within the code system and within the industry before a true performance-based design can be performed and accepted. However, we understand that the current approach is pragmatic in the current environment in China of fast-track construction of supertall buildings.

总结

对于天津高银117大厦这样一个高纤细建筑的结构设计，在中国规范基础上采用相应的抗震性能化设计贯穿了整个设计过程。基于大量的线性与非线性反应谱和动力时程进行的抗震分析与抗风工况分析确保了整个结构满足相应的性能目标。在现有中国规范体系下，实施并接受真正的性能化设计仍旧存在一些障碍。但考虑当前中国建造超高层建筑急速发展的环境，目前的设计方法是在现行规范下较为实际和稳妥的。

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