Title: **Research and Design of a Complex Connected Structure Consisting of Three Super High-Rise Towers**

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Research and Design of a Complex Connected Structure Consisting of Three Super High-Rise Towers | 超高层三塔连体结构研究与设计

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Abstract | 摘要

Currently under construction, Golden Eagle Plaza is slated to be the highest rigid-connected building in the world. The building consists of three supertall towers of over 300 meters, which are connected at the height of approximately 200 meters by a six-story sky lobby. The longest span of the sky lobby exceeds 70 meters. The dynamic properties and coupling effects brought on by the six-story sky lobby and the individual towers are discussed in this paper. A method based on sinusoidal excitation was applied to detect the development's seismic weakness. Characteristics of the wind load and key points on the design of the sky lobby are expounded. Research concludes that the seismic performance of this connected structure is quite different from that of a single tower. Some regulations in present codes are inapplicable to this particular structure. Results of the shake-table test confirm that the seismic capacity of the connected structure is reliable and appropriate, and it is consistent with the established design objective.

Keywords: Connected Structure, Seismic, Shake-Table Test, Structure Dynamic, Supertall, Wind Loads

南京金鹰天地广场是目前全世界在建的高度最高、连体跨度最大的非对称刚性连体结构，该项目由三栋高度均超过300米的超高层塔楼在200米左右的高空经6层空中大堂连接形成，空中大堂最大跨度超过70米。文章对超高层连体结构的动力特性进行了探讨，分析了连接体和塔楼耦合关系对整体结构抗震性能的影响，建议了一种基于正弦波激励的动力时程法用于超高层连接结构分析。对连体结构的风荷载特性进行了探讨，并对空中连体大堂的设计要点进行了阐述。研究表明，超高层连体结构的动力特性和抗震性能和普通超高层建筑有较大的区别，现行规范的部分相关规定对于这种特殊的结构形式并不适用。模拟地震振动台试验验证了相关研究结论，表明结构的整体抗震能力是合理、可靠的，结构的实际抗震性能与性能设计目标相吻合。

关键词：连体结构、地震、振动台试验、结构动力学、超高层建筑、风荷载

Background

Connected super high-rise structures have inspired architects to expand their creativity on aspects of both elevation and layout. Currently, connected buildings with super high-rise towers completed and under construction are mostly twin-tower developments, such as the Petronas Twin Towers, the new CCTV Building (Dasui 2008), and Oriental Gate (Min 2012). Related research is also concentrated on the twin-tower structures. In recent years, connected structures have had a tendency towards becoming multi-tower formations. Due to the complex mechanical properties caused by rigid connections between towers, multi-tower developments in general can be designed in forms of moveable connections by seismic bearings to simplify structural systems. Examples of these types of towers include the Singapore Marina Bay Sands, Hangzhou Citizen Center (Yimin 2009), and Beijing MOMA (Ziguo 2008).

背景

超高层连体结构给予了建筑师在立面和平面上充分的创造空间，近年来得到了越来越多的应用。目前已经建成和在建的超高层连体结构多为双塔连体结构，如吉隆坡彼得罗纳斯大厦、北京CCTV主楼（Dasui 2008)、苏州东方之门（Min 2012)等，相关的研究工作也主要集中在双塔连体结构领域。近年来，连体结构已有向多塔连体方向发展的趋势，但由于多塔刚性连体结构力学特性更为复杂，因此这类结构往往采用滑动或隔震支座的连接方案，简化受力体系，如新加坡金沙酒店、杭州市民广场（Yimin)、北京当代MOMA[4]等。

位于中国南京的金鹰天地广场项目由一个超高层三塔连体主楼、9~10层裙房以及15层地上停车库组成，地下室4层。主塔楼与裙房间设置抗震缝，形成各自独立的结构单元。主楼由三栋高度分别为368m（76层）、328m（68层）、300m（60层）的超高层塔楼A、B、C在200m左右的高空连接而形成，连接体共6层（43~48层），高度超过40m，最大跨度超过70m，是目前世界在建的高度最高、连体跨度最大的

Figure 1 shows the Golden Eagle Plaza structure, which consists of a super high-rise connected building, a podium, and a parking garage (Figure 1). The main connected building and podiums are divided into separate units. The main building involves three diverse towers named Tower A (368 meters), Tower B (328 meters), and Tower C (300 meters). This trio of towers is connected by a six-story sky lobby at the elevation of approximately 200 meters. The height of the sky lobby is about 40 meters and spans about 70 meters, making the building the highest and longest rigid-connected triple tower in the world. Figure 2 presents a typical plan for the connecting floor (Figure 2). The three towers are different not only in height, but also in their directional orientation. The complex structural form brought great challenges to the designing phase. Using this project as a case study, this paper seeks to identify the project's static and dynamic properties, analyze its methods, and provide some key focuses on the design of similar structures.

Two-Structure Arrangement

Golden Eagle Tiandi Plaza is located in Nanjing City, of which the seismic precautionary intensity degree is seven. The site is categorized as class III and the design earthquake is classified to group one, according to Chinese Code GB50011-2010. The three towers are hybrid structures of SRC columns combined with interior concrete tubing. Accessional belt trusses and outrigger trusses are set up in equipment layers to enhance the lateral stiffness of the towers. The bottom of the concrete tubing was reinforced with steel plates to improve ductility and reduce the thickness of the wall. Maximum thickness of the shear wall is 1,300 millimeters (with a steel plating of 35 millimeters thick). In this project, the highest concrete level of exterior column is C70 and C60 of the concrete tubing (with a steel plating of Q345B).

The three towers are connected through five-story-high (from the 43rd to 47th floor) trusses. The main truss surrounds the sky lobby and the three towers, serving as a deformation coordinator as well as the lateral stiffener. The bottom story of the sky lobby is equipped with transfer trusses to bear the vertical loads of the whole platform. The highest material level of the surrounding trusses and transfer trusses is Q390GJC. The structure arrangement can be referred as shown in Figure 3 (Figure 3). The damping of the hybrid structure under frequent

超高层三塔连体结构，建筑效果图（图1）。图2为空中连体部分的平面图，可以看到，主塔楼不仅高度各异，平面尺寸、布置以及主方向角度也各不相同，复杂的结构形式给设计工作带来了巨大的挑战（图2）。本文以金鹰天地广场为依托，对该项目在力学特性、分析方法与设计上的关键问题进行研究和归纳总结，相关成果可为此类结构体系的进一步研究和应用提供参考资料。

结构体系

金鹰天地广场位于南京市河西区，抗震设防烈度为7度，设计地震分组第一组，场地类别III 类，整体结构体系如图3所示。三栋超高层塔楼采用型钢混凝土框架柱加混凝土核心筒的混合结构体系，结合设备层设置环带桁架和伸臂桁架对塔楼抗侧刚度进行加强。为了提高剪力墙延性并减小剪力墙厚度，核心筒底部外墙采用钢板混凝土剪力墙，最大剪力墙截面厚度为1300mm（钢板厚度35mm）。本项目框架柱混凝土最高强度等级为C70，核心筒最高为C60（钢板Q345B）。

空中连接体为钢结构，周边通过五层（43~47层）高的主桁架将各塔楼两两相连，主桁架环绕贯通三栋塔楼，在协调三栋塔楼变形的同时并提高各塔楼的抗侧刚度。连体最下层（43层）设置转换桁架，承担空中平台内部的竖向荷载。连接体主桁架与转换桁架的材料强度为Q390GJC，结构布置见（图3）。根据《高层建筑混凝土结构技术规程》（JGJ3-2010），混合结构在罕遇地震作用下的阻尼比取为0.04。

Figure 1. Golden Eagle Tiandi Plaza (Source: ECADI)
图1. 金鹰天地广场效果图（来源：华东建筑设计研究总院）

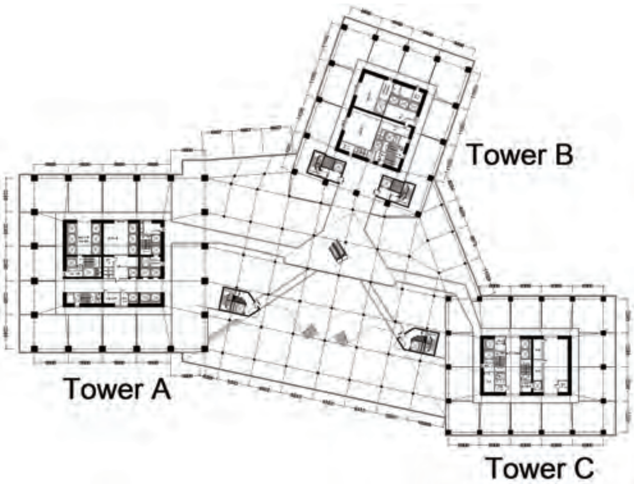

Figure 2. Plan of the connecting stories (Source: ECADI)
图2. 连体楼层平面图（来源：华东建筑设计研究总院）

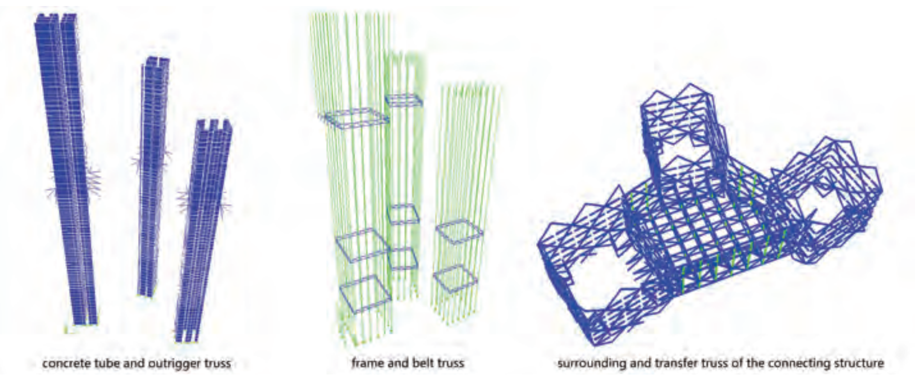

Figure 3. Structure arrangement of the Golden Eagle Tiandi Plaza (Source: ECADI)
图3. 连体结构布置（来源：华东建筑设计研究总院）

Mode	Period(s)	UX (%)	UY (%)	RZ (%)
1	6.84	6.46	54.15	6.34
2	6.52	62.33	7.15	2.08
3	5.86	1.16	6.99	59.99

Figure 4. Modal characteristic of connected structure (Source: ECADI)
图4. 连体结构模态特性（来源：华东建筑设计研究总院）

Mode	Period(s)	UX (%)	UY (%)	RZ (%)
1	7.72	61.15	6.81	0.08
2	7.45	6.62	57.95	0.06
3	4.73	0.09	0.03	74.09

Figure 5. Modal characteristic of Tower A (Source: ECADI)
图5. 独立塔楼A模态特性（来源：华东建筑设计研究总院）

earthquakes was four percent, according to Technical Specification for Concrete Structures of Tall Building (JGJ3-2010).

Structure Analysis and Design

Dynamic Properties

It is observed that there are fixed modes of a symmetrical connected structure. Translational motions on x and y axes are independent to each other, and there is no coupling between translational motion and torsional motion (Chunfeng 2008). Nonetheless, in this project, there is no symmetrical relationship between any two towers and the local coordinates of the three towers are not uniform. In consequence, there are no fixed basic modes for this structure. Meanwhile, the coupling effects of three degrees of freedom make all modes translation-torsion hybrid. The accurate solution of modes cannot be deduced by solving the matrix formula, but obtained approximately by numerical simulation.

The information on the first three modes of the connected structure and Tower A is listed in Figures 4 and 5 (Figure 4 & 5). It reveals that the torsion proportion in the translational motion of the connected structure is higher than that of a single tower, so the first mode shape is mixed with certain torsion (Figure 6).

For the rigid-connected structures, the coupling response depends on the eccentricity degree between the stiffness and mass centers. In this project, three separate towers' stiffness was optimized to reduce the lateral deformation difference under earthquakes at the range of the sky lobby. The adjustment weakened the coupling response of translation and torsion accordingly and, as a result, the torsional mass participation coefficients of the main translation modes on the y and x directions are decreased to 6.34 and 2.08 percent.

Torsional Vibration

Because of the previously mentioned coupling effect, connected structures are more likely to behave with torsion movement under the dynamic load. The seismic indices, such as the torsional period ratio (the first torsion period Tt to the fist translation period T1) and the torsional displacement ratio of the whole structure, are difficult to satisfy the design code. The period ratio and displacement ratio of the Golden Eagle Tiandi Plaza are 0.86 and 1.50 (ECADI 2013), and that of Oriental Gate are 0.88 and 1.53 (Min 2012).

However, existing dynamic time history analysis and shake-table tests indicate that the actual damage caused by the torsional response of well-designed, connected structures were not serious (Xiaohan 2011, China Academy of Building Research 2013). Figure 5 shows that the torsion center of the rigid-connected structure is generally located

结构分析与设计

动力特性

研究表明[6]：对称连体结构存在固定的基本模态，其中x、y向平动相互独立，且平动与扭转无耦合。本项目双向均无对称性，且三塔楼主向角度不同，结构的基本模态和对称连体结构有较大不同，同时，x、y向平动与绕z轴扭转的耦合，使得每个模态均为平扭耦合振动，矩阵分析难以确定具体的振动形态，需通过数值计算得到。

表4为金鹰天地广场三连体结构的前三阶模态信息，表5为独立塔楼A的前三阶模态信息，通过对比可以看到（图4、5），连体结构中以平动为主的模态的扭转分量（RZ值）较普通单塔结构有明显增加。即连体结构的第一阶模态表现出一定的整体扭转效应（图6）。

对于刚性连接的连体结构，平动为主模态的扭转分量大小取决于结构刚度中心和质量中心的偏心率。本项目在设计过程中，首先取消连接体，以各独立塔楼在原连体位置处的水平变形差为优化目标，通过调整各塔楼刚度比，减小了这种平扭耦合效应，最终使得连体结构以平动为主的模态扭转质量参与比减小到6.34%和2.08%。

结构扭转

连体结构的平扭耦合效应明显，因此在动力荷载作用下，结构较易发生整体扭转现象。在设计指标上反映为扭转为主的第一周期Tt提前或与平动为主第一周期T1之比难以满足规范要求且整体扭转位移比超限。金鹰天地广场整体扭转周期虽出现在

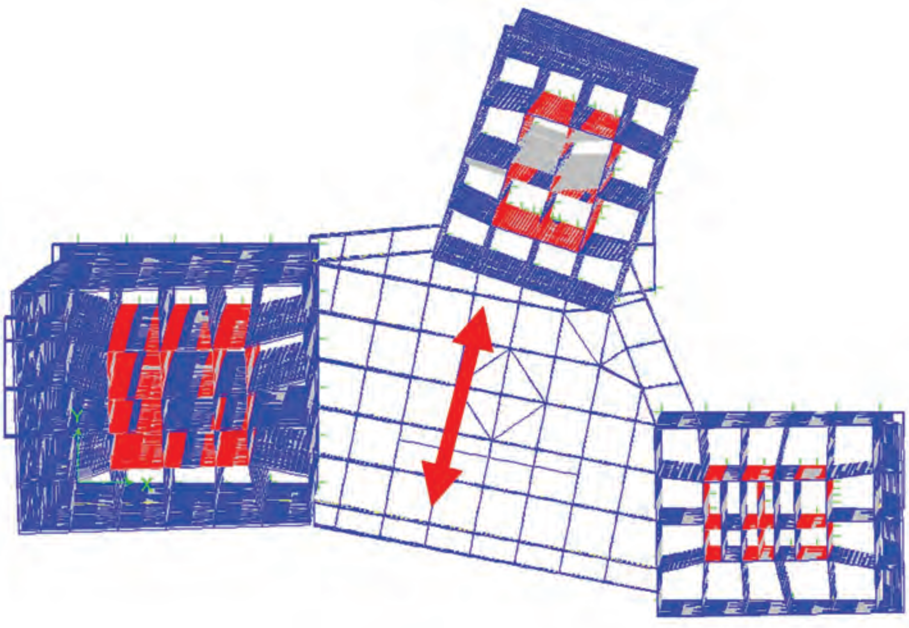

Figure 6. Coupling response of translation and torsion (first mode) (Source: ECADI)
图6. 连体结构平扭耦合效应（第1阶模态）（来源：华东建筑设计研究总院）

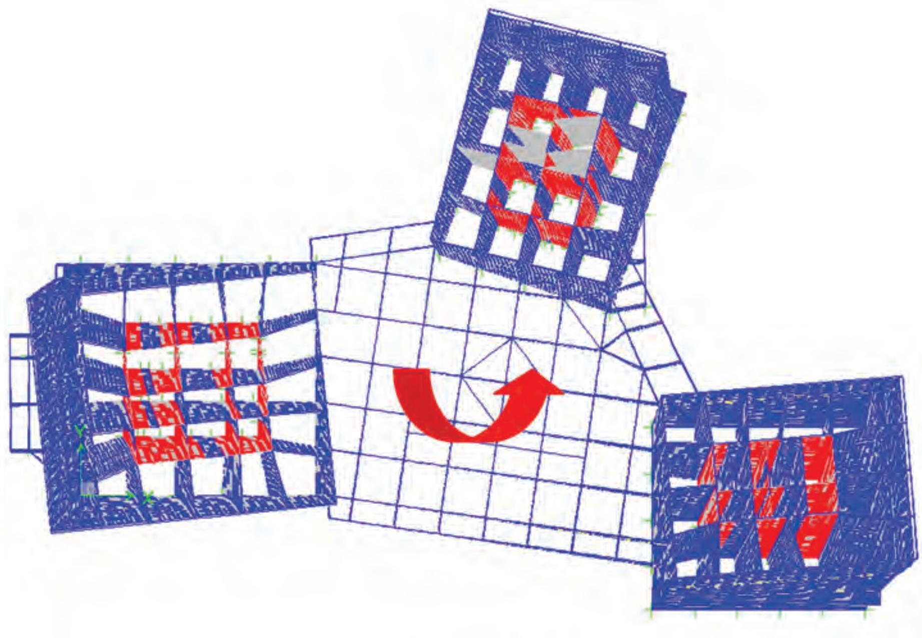

Figure 7. Torsional response of the whole structure (third mode) (Source: ECADI)
图7. 连体结构整体扭转（第3阶模态）（来源：华东建筑设计研究总院）

outside of the towers (Figure 7). It is the phase difference of each tower's translational motion that leads to the torsional response of the whole structure; as for each tower, it is essentially translational movements, but not individual torsion. Meanwhile, the individual torsional mode of each tower is hard to trigger because of the restraining of the rigid sky lobby. In this project, the individual torsional modes of the three towers occur at the sixth, 11th, and 15th mode, respectively. Torsional period ratios are merely 0.32, 0.21, and 0.17, respectively.

Take Tower A for example; Figure 8 shows its torsional displacement ratio (Figure 8). The values of the ratio for most floors are less than 1.1, except for a few floors at the bottom. Each individual tower is strengthened by the rigid connecting structure, which makes the anti-torsion stiffness of the towers in connected structures much larger than that of the single-tower structures. This has also been verified by Oriental Gate – another connected structure with super high-rise towers designed by ECADI (Jiahua 2008).

Relevant provisions in code, such as the torsional period ratio and the torsional displacement ratio on the limitation of a structure's torsion are based on the rigid-floor assumption, and they are suitable for the regular, single-tower structures. Rigid-floor assumption is not, however, suitable for connected structures. In other words, each tower of the connected structures works as an equivalent "column" or "wall" element, and anti-torsion stiffness of a whole tower is much greater than that of a single column or wall. Hence, for designing the connected structures, suggestions are given as follows: firstly, the requirement on the torsional period ratio could be lower, but the torsional mass participation coefficient in the main translation mode should be minimized; secondly, the requirement on the torsional displacement ratio of the whole structure could be lower, but that of each single tower should be strictly limited.

Coupling Effect of Towers and Connecting Structures

Figure 9 lists the ratios of the elevation of the sky lobby to the height of each tower (Figure 9). The location of the lobby is approximately 1/2 to 2/3 the elevation of Tower A, 2/3 of Tower B, and 3/4 of Tower C, respectively; it is, in general, the middle-upper

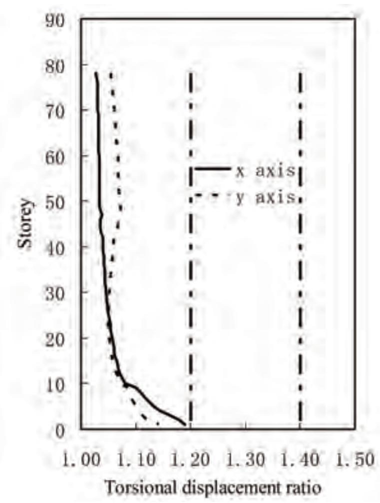

Figure 8. Torsional displacement ratio of Tower A (Source: ECADI)
图8. 塔楼A扭转位移比（来源：华东建筑设计研究总院）

第三阶，但周期比为0.86，整体结构扭转位移比最大为1.50；又如东方之门，其扭转周期出现在第二阶，扭转周期比为0.88 (Min 2012)，整体结构扭转位移比最大为1.53。

然而已有的时程分析和振动台试验均显示（Xiaohan 2011，China Academy of Building Research 2013）：设计合理的连体结构，扭转造成的结构实际损伤并不严重。图5显示整体结构的扭转中心位于各塔楼范围以外，因此刚性连体结构的扭转模态实质是由各塔楼的平动相位差所引起（图7）。对每个塔楼而言，其扭转效应很小。进一步分析发现，由于刚性连接体的约束作用，连体结构中各塔楼的扭转模态难以发生，塔楼A、B、C的扭转周期分别出现在第8、11、15阶，与第一平动周期之比分别仅为0.32、0.21、0.17。

图8为连体结构中塔楼A的扭转位移比（图8），除底部几层外，绝大部分楼层的扭转位移比均小于1.1。以上结果均表明刚性连体结构中的各塔楼扭转效应较独立单塔结构有了较大改善，因此刚性连接体增强了各塔楼的抗扭刚度。此规律在本院的另一个超高层连体结构—东方之门的设计中也得到证明（Jiahua 2008）。

现行规范对于结构扭转效应的限制条件（扭转周期比和扭转位移比）对于满足刚性楼板假定的单塔结构是适用的。但在连体结构中，整体刚性楼板的假定不成立，同时各组成塔楼相当于单塔结构中的“柱”和“墙”，其抗扭刚度要远远大于后者。因此在连体结构的设计中，我们建议：1适当放松整体结构的扭转周期比限制条件，尽量减小平动模态的扭转质量参与比；2降低整体结构的扭转位移比要求，重点控制连体结构各塔楼自身的扭转位移比。

连接体与塔楼的耦合效应

图9为空中连体标高与各塔楼总高度之比（图9），可以看到，连接体的位置分别接近塔楼B、C高度的2/3和3/4，对于塔

	Tower A 塔楼A	Tower B 塔楼B	Tower C 塔楼C
Position of Sky Lobby	368m	328m	300m
Top (225.6m) 主桁架中心(标高 208.6m)	0.613	0.688	0.752
Middle (208.6m) 主桁架中心(标高 208.6m)	0.567	0.636	0.695
Bottom (191.6m) 主桁架底面(标高 191.6m)	0.521	0.584	0.639

Figure 9. Ratio of the elevation of the sky lobby to the height of the tower (Source: ECADI)
图9. 连体标高与各塔楼高度比（来源：华东建筑设计研究总院）

Direction 方向	Single Tower A 独立 塔楼A	Single Tower B 独立 塔楼B	Single Tower C 独立 塔楼C	Connected Structure 三塔连体
x	1.58	1.76	1.74	3.9
y	1.62	1.58	1.8	3.23

Figure 10. Ratio of equivalent lateral stiffness to gravity (Source: ECADI)
图10. 结构刚重比（来源：华东建筑设计研究总院）

Unit: 10^7 kN-m 单位：10^7 kN–m	Total Overturing Moment 总倾覆弯矩	Load Overturning Moment 局部抗倾覆弯矩	Whole Overturning Moment 整体抗倾覆弯矩	Proportion of the Whole Overturning Moment 整体抗倾覆弯矩比
x	1.986	1.500	0.486	24.40%
y	1.806	1.327	0.479	26.50%

Figure 11. Analysis on basement overturning moment (Source: ECADI)
图11. 结构倾覆弯矩分析（来源：华东建筑设计研究总院）

level of each tower. Analysis on position of the connected stories revealed that (ECADI 2013) the lower they placed, the weaker the lateral stiffness of the connected structure. At the same time, the structure's torsional response enhanced and the whiplash effect of the stories above the connected part intensified, meaning that the seismic performance of the connected structure is optimal when the connecting structure is placed at the upper elevation of the tower.

Figure 10 presents the parameter of single towers and connected structures to reflect the structural stability of each (Figure 10). The parameter is the ratio of equivalent lateral stiffness to the gravity of the structure. It indicates that the structure's stability can be significantly improved by the informed, decisive arrangement of the connecting structure. Utilizing this advantage with the efficiency analysis of the outrigger truss, each tower is eventually reinforced with the outrigger truss in just the fist story of the sky lobby. The outrigger truss can be seen in Figure 3 (a).

According to the analysis on the overturning moment of the connected structure under earthquakes (Figure 11), the local overturning moment borne by each single tower serving as "columns" of the connected building decreased, while the proportion of the whole overturning moment borne by the axial forces of the towers increased up to 24.4 and 26.5 percent on the x and y directions, respectively. The local and whole overturning moment are defined as shown in Figure 12 (Figure 12). The result shows that the connected structure of three towers works effectively as a whole structure. The proportion of the whole overturning moment is a key index for the evaluation of the entirety of the connected structure. Owing to the higher efficiency of the axial-force-resisting components than the moment-resisting ones, the bending moment of most columns and walls in the connected structure decreased when compared to what it would have been if separated into three towers.

Modal Sinusoidal Excitation Method

The Code for the Seismic Design of Buildings (GB50011-2010) provides that three to seven sets of earthquake waves should be brought in when performing a dynamic time history analysis; however, due to the randomness of the actual earthquakes and complicated dynamic properties of the connected structure, tests of the limited earthquake waves may not be able to reflect the structural seismic performance comprehensively (Yayong 1991). To obtain profound understandings on the dynamic properties of connected structures, a method based on sinusoidal excitation can be applied. Sinusoidal excitation will amplify the resonance reaction of the structure exited

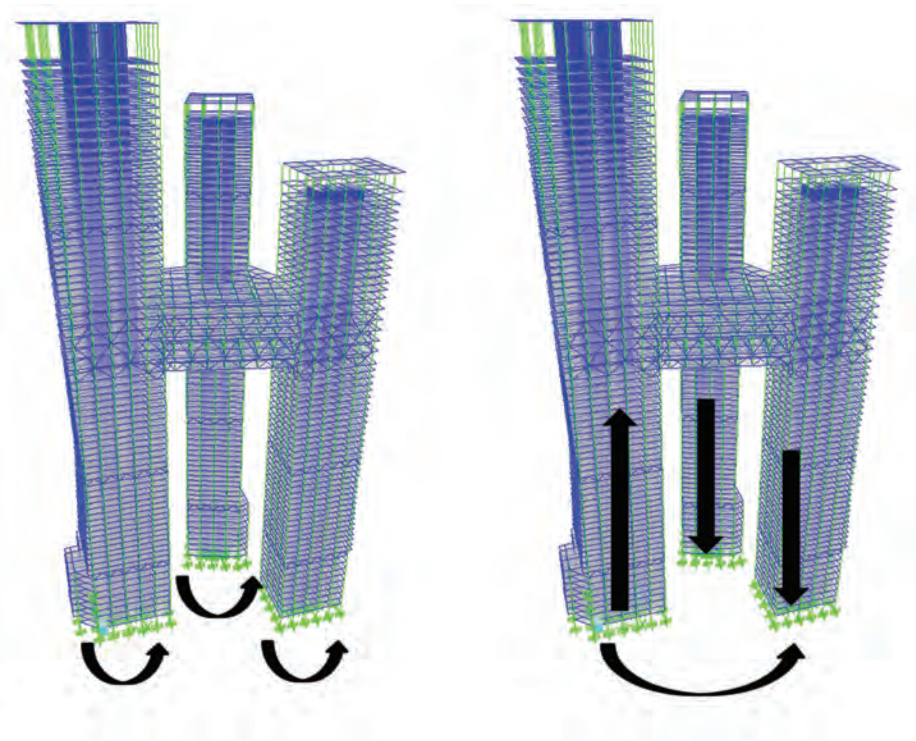

Figure 12. Definition of local and whole overturning moment (Source: ECADI)
图12. 局部倾覆弯矩和整体倾覆弯矩（来源：华东建筑设计研究总院）

楼A则在高度的1/2~2/3之间，总体上均位于各塔楼的中上部位。通过对刚性连体结构中的连接体位置的参数化分析后发现(ECADI 2013)：随着连接体位置的降低，连体结构整体抗侧刚度降低，塔楼的扭转效应增加，且连体以上部分的鞭稍效应增强，因此当连接体处于各塔楼的中上部位时，结构的整体抗震性能较好。

图10为单塔与连体结构的刚重比，合理的连接体位置显著提高了结构的整体稳定性。利用这一有利条件，通过伸臂桁架的效率分析，各塔楼最终仅在连体首层设置了一道伸臂桁架（图3(a)），即可满足结构的刚度需求。

通过对单塔以及连体结构基底倾覆弯矩的对比分析发现（图11），在水平地震作用下，各塔楼作为连体结构的"柱子"所承担的局部倾覆弯矩减小，三塔楼轴力承担的整体倾覆弯矩在X、Y向分别达到总倾覆弯矩的24.4%和26.5%（图12）。表明连体后的结构整体效应明显，这是判断连体结构连接强弱的重要指标。进一步分析还表明，由于在承担水平荷载时，轴向受力构件的效率大于受弯构件，因此连体后各塔楼竖向构件的内力也较独立塔楼有所减小。

多模态正弦波激励法

《建筑抗震设计规范》（GB50011–2010）规定，在进行动力时程分析时，可取三组或七组地震波，并对计算结果取包络或均值进行设计。但实际地震动的随机性较大，且超高层连体结构的动力特性复杂，有限数量的地震波输入往往难以全面

by each mode, thus the weakness of the structure can be thoroughly detected. This is helpful to improve the structure arrangement and reinforcement. The method is elaborated as follows:

- Carry out the modal analysis on the structure and acquire the periods of modes;
- Establish functions on multi-modal sinusoidal waves as the periods of sinusoidal waves are equal to the corresponding mode periods, the peak ground accelerations are determined by the seismic precautionary intensity, and the wavelengths are as long as five to 10 periods;
- Calculate the translational angle of the sinusoidal wave of each mode (Wilson 2006) as follows (Where $p_j = -\phi_j^T M$ X_{jm} and Y_{jm} are the mass participation ratios on the x and y directions of the j[th] mode);

$$\theta_{jx} = \tan^{-1}(p_{jy} / p_{jx}) = \tan^{-1}\sqrt{Y_{jm} / X_{jm}}$$

- Perform a dynamic history analysis with the established functions of sinusoidal waves in the direction of the translational angle θ_{jx}.

Figure 13 presents the story drift of three towers on the x direction through the analysis of the first 10 sinusoidal waves for the outrigger and belt trusses that are simultaneously placed in the first story of the sky lobby, while the other strengthened stories are equipped with just belt trusses, greatly improving the stiffness catastrophe (Figure 13).

The rigid connecting structure is located at the 2/3 to 3/4 elevations of Towers B and C, allowing the two towers be restrained more reliable; therefore, the towers' dynamic response is mainly contributed by the first two modes. At the same time, the stiffness of all stories (except the connected ones) is well-distributed and there are no serious vulnerabilities. For Tower A whose connected position is relatively lower, the structure not only reacts to the first two mode contributions, but also performs with an obvious whiplash-effect above the sky lobby excited by high-order modes.

Based on the results of the analysis, some improvements were made to the structure:

- For the floors above the sky lobby of Tower A, the story shear forces are moderately amplified at the basis of the results of response spectrum analysis;
- The beams of the outside frame above the sky lobby are strengthened and the corresponding seismic performance objective is upgraded to be elastic under medium earthquakes;
- In order to moderate critical changes of story lateral stiffness, additional belt trusses are added to adjacent stories close to both the upward and downward connections of the structure; and
- The reinforcements in the concrete tubing neighboring to the sky lobby are strengthened to improve the ductility capacity.

Wind Load

反映结构的抗震性能（Yayong 1991）。为了对复杂连体结构的振动特性进行深入的分析，本文建议可采用多模态正弦波激励法进行补充分析。正弦波激励可强化结构在各阶模态下的共振响应，能够全面揭示结构的薄弱部位，为结构布置与加强提供依据。分析方法如下：

- 对结构进行模态分析，得到结构各阶振动周期；
- 建立多模态正弦波函数。正弦波的周期与各模态周期对应，峰值加速度根据抗震设防烈度确定，持续时间取5～10个周期；
- 确定各阶模态对应的正弦波方向角（Wilson 2006）

$$\theta_{jx} = \tan^{-1}(p_{jy} / p_{jx}) = \tan^{-1}\sqrt{Y_{jm} / X_{jm}}$$

$$p_j = -\phi_j^T M \quad X_{jm} \quad Y_{jm}$$

  其中，分别为第j阶模态x、y向的质量参与比。
- 利用各阶模态对应的正弦波对结构进行动力时程分析。

图13给出了前13阶正弦波分析所得到的三塔x向层间位移角分布，由于本项目仅在连接体的首层设置伸臂桁架，其他加强层只采用环带桁架，因此楼层刚度突变主要集中在连接体部位，其他加强层的刚度突变得到改善（图13）。

对于塔楼B和C，刚性连接体处于2/3~3/4高度处，对两塔楼的约束较强，结构的动力反应贡献以前三阶模态为主，受高阶模态的影响较小，同时除连接体外的楼层刚度均匀，结构无明显的薄弱部位；对于塔楼A，连接体的位置相对较低，结构的变形除受前二阶模态影响较大外，连体以上结构还在部分高阶模态正弦波的激励下，

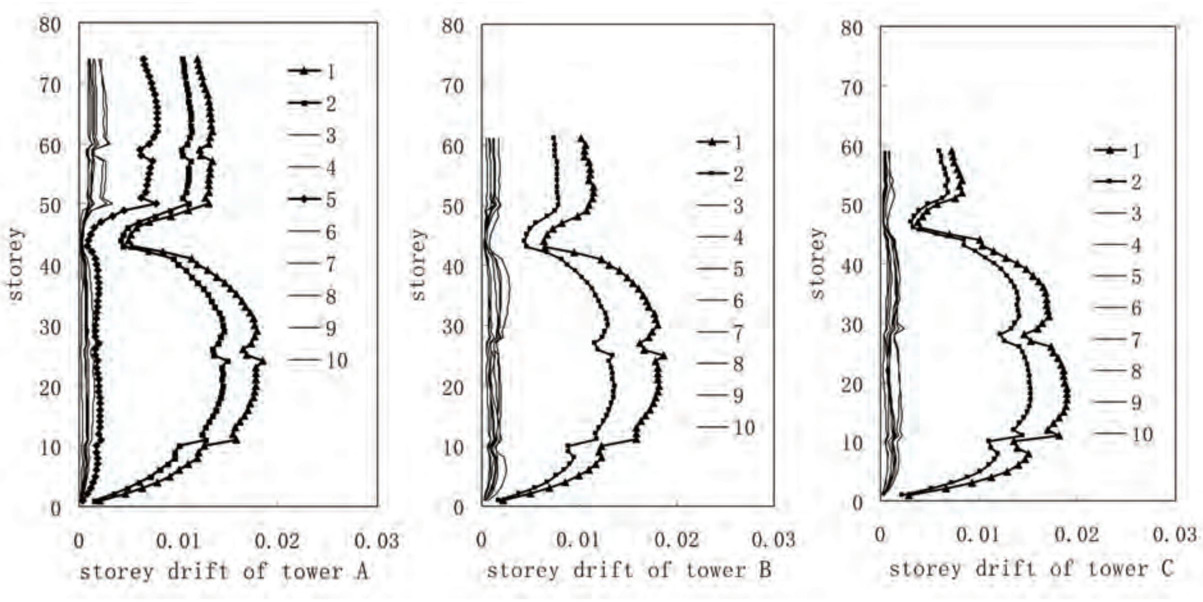

Figure 13. Story drift under sinusoidal excitation (Source: ECADI)
图13. 正弦波激励下的各塔楼层间位移角分布（来源：华东建筑设计研究总院）

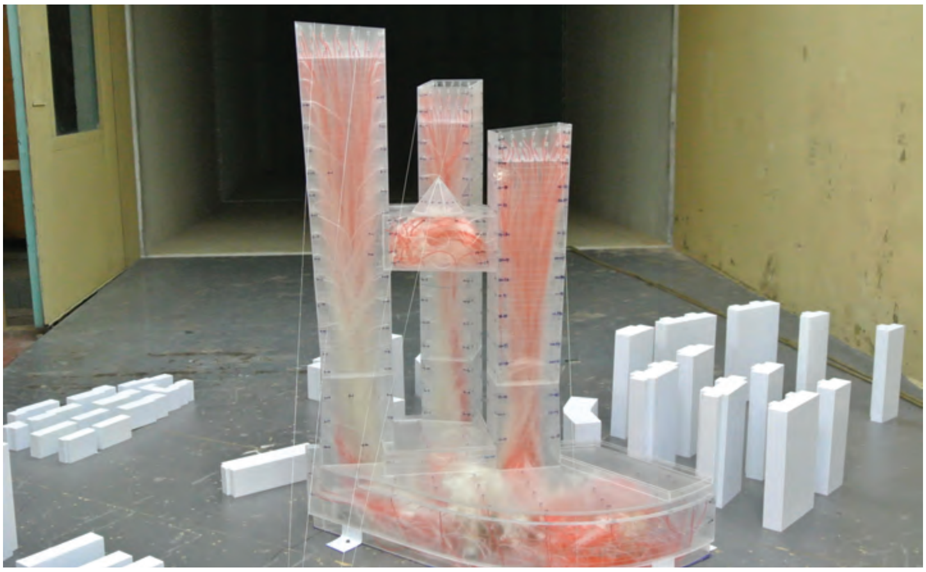

Figure 14. Wind tunnel test of Golden Eagle Tiandi Plaza (Source: ECADI)
图14. 风洞试验模型（来源：华东建筑设计研究总院）

The wind load of connected structures is influenced by a large number of factors, such as the number, shape, distance, and direction of the towers and the various properties of the connecting structure. Wind tunnel testing of the project was entrusted to the State Key Laboratory of disaster prevention for civil engineering in Tongji University. The model's scale is 1:350 (Figure 14). The terrain roughness is categorized as class B, and the basic wind stresses are 0.40kN/m2 and 0.45 kN/m2, in respect to a 50-year and 100-year return period.

Based on the results of the wind tunnel testing, the story shear forces of the three towers before and after they are connected to each other were analyzed in detail. Take the y-direction for example (Figure 15), the basement shear forces of a connected Tower C increased by 13095 kN as compared to that of the separate Tower C structure. In the same situation, the shear forces of Towers A and B decreased by 264 kN and 7468 kN, respectively. In addition, there was a transfer of about 5363 kN from the range of the sky lobby to Tower C. All of the above data indicates that the wind loads were redistributed among towers owing to the influence of the sky lobby.

The same analysis was also carried out based on the wind loads in accordance with the load code (GB50009-2012). The results are illustrated in Figure 16 (Figure 16), showing that the method presented in code does not take the coupling effect of the sky lobby and towers into consideration, and consequently failed to reflect the redistribution of wind loads.

表现出显著的鞭稍效应，形成抗震设计的相对薄弱部位。

根据多模态正弦波激励分析结果，设计时采用了如下对应加强措施：

- 塔楼A连体以上的楼层剪力在反应谱分析结果的基础上适当放大，用以进行主要抗侧力构件的承载力设计；
- 对连接体以上楼层的结构外框架梁进行加强，提高其性能目标至“中震弹性”；
- 三塔楼在连接体上下各两层的范围内增设环带桁架，减轻连接体与上下相邻楼层的刚度突变；
- 对连接体楼层及上下层相邻层的核心筒配筋和型钢予以加强，提高构件延性；

风荷载

连体结构的风荷载受塔楼刚度、形状、距离、角度以及连体形状、位置、与塔楼耦合关系等因素的综合影响，其作用机理较单塔结构有显著不同。项目委托同济大学土木工程防灾国家重点实验室进行了风洞试验，模型缩尺比为1:350（图14），地面粗糙度类别为B类，基本风压分别为0.40kN/m2、0.45 kN/m2（对应50年及100年重现期）。

根据风洞试验结果，对各塔楼连体前后的楼层剪力进行对比分析，以y向为例（图15），结果表明：C塔首层剪力连体后较连体前增加了13095kN，A塔剪力减小264kN，B塔剪力减小7468kN， 同时连体部位传递到C塔风载5363kN，这表明三塔楼刚性连接以后，塔楼A、B通过连接体将部分风荷载传递至C塔，即风载效应在塔楼间存在重分布现象。

此处对现行荷载规范计算得到风载进行同

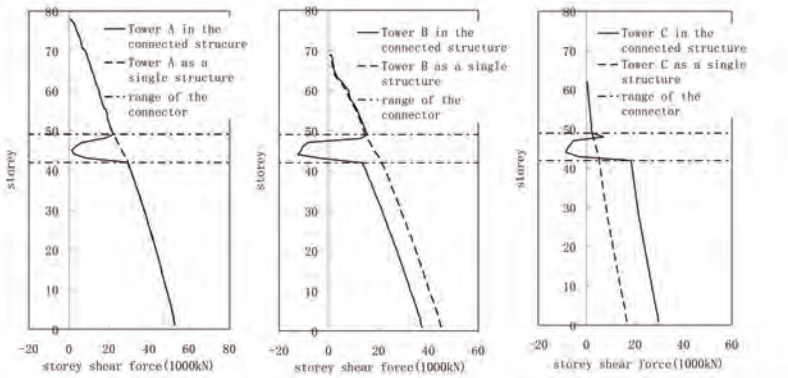

Figure 15. Story shear forces under wind loads based on wind tunnel testing (Source: ECADI)
图15. 风荷载作用下楼层剪力分布规律——基于风洞试验（来源：华东建筑设计研究总院）

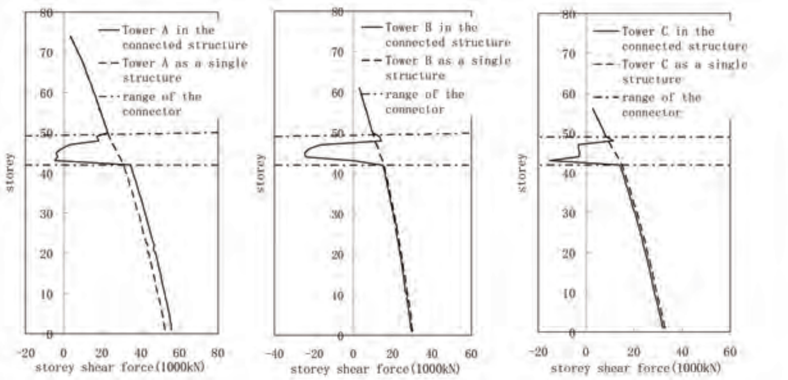

Figure 16. Story shear forces under wind loads based on the load code (Gb50009-2012) (Source: ECADI)
图16. 风荷载作用下楼层剪力分布规律——基于荷载规范（来源：华东建筑设计研究总院）

Component 构件名称	Design Earthquake 设防烈度地震	Rare Earthquake 罕遇地震
Frame Column | 连接体框架柱	Elastic | 中震弹性	—
Outrigger Truss | 伸臂桁架、连体楼板	Un-yield | 中震不屈服	—
Connecting Floor | 连接体环带桁架	Un-yield | 中震弹性	—
Belt Truss | 连接体主桁架	Elastic | 中震弹性	—
Transfer Truss | 连接体转换桁架	Elastic | 中震弹性	Un-yield | 大震不屈服

Figure 17. Seismic performance objective of the components in the sky lobby (Source: ECADI)
图17. 连接体重要构件抗震性能目标（来源：华东建筑设计研究总院）

Design of the Sky Lobby

The sky lobby works not only to bear gravity loads, but also to coordinate the non-synchronous deformation of the three towers. It also enhances the overall structure's lateral stiffness and stability; however, as the heights and directional angles differ from tower to tower, the situation of the sky lobby under wind loads and earthquakes becomes extremely complicated. Additionally, the settlement difference among the towers also generates further internal forces to the sky lobby.

As previously mentioned, the towers' stiffness was adjusted to harmonize the deformation difference of three towers at the height of the connected position, so it is not only the translation-torsion coupling vibration of the connected structure, but also the internal force of the sky lobby can be improved efficiently.

Performance-based seismic design was conducted in this project. The performance objective of the components in the sky lobby was upgraded (Figure 17). The floors of the sky lobby are important in transferring the lateral force among the three towers, so they were strengthened especially. The floors are thickened to 200 millimeters and reinforced according to the stress analysis results under wind loads and earthquakes. Moreover, horizontal X-braces were also arranged in the adjacent range between the towers and the sky lobby.

Finally, in order to reduce the additional internal forces of the sky lobby caused by the settlement difference, a mudstone formation with the better elastic modulus was chosen as the bearing stratum of all the three towers. The depth of piles penetrating into the bearing stratum is not less than seven times the pile diameter, and the post-grouting at bottom of the pile was introduced. Calculation results shows that the maximum settlement difference of the three towers is about 10 millimeters. At the same time, the appropriate arrangement of the construction sequence and post-installation of the members sensitive to the settlement difference will also help to reduce or even eliminate additional internal forces.

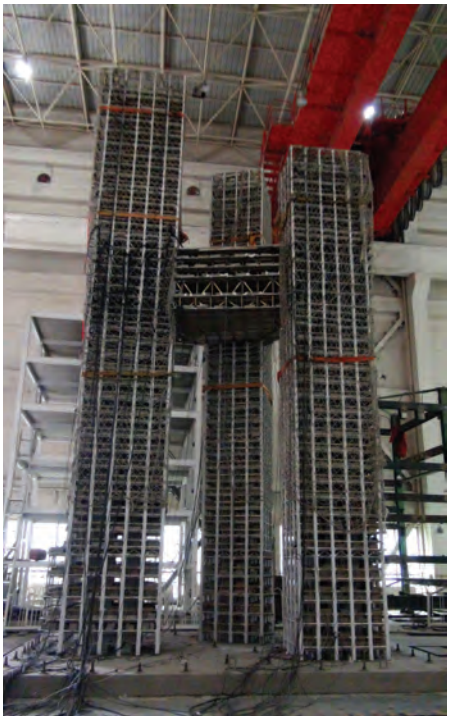

Figure 18. Structure model for shaking-table test (Source: ECADI)
图18. 振动台试验模型（来源：华东建筑设计研究总院）

Shake-Table Testing

A shake-table test was carried out in the State Key Laboratory of Building Safety and Environment to check the seismic capability of the structure. The scale of the model is 1/40, and the height is 9.2 meters (Figure 18).

On a scale of frequent earthquakes, seven groups of earthquake waves (five natural waves and two artificial waves) were used. Each group of earthquake waves was input in three directions simultaneously. In order to reduce the influences of accumulative damages during the following tests, three groups of waves were chosen for the first stage of testing, as the three earthquake waves should motivate comparative large responses of the structure; then the most adverse one from the previous three waves was chosen for testing rarer earthquakes.

样的分析，结果如图16所示（图16），规范方法计算得到的风荷载没有考虑塔楼间的相互影响，无法准确地反映风载重分布现象，因此对于连体结构是不适用的。

连接体设计

本项目中，空中连体不仅承担重力荷载，还起到协调塔楼变形差异以及不同步振动，提高结构整体刚度及整体稳定性的作用。由于三栋塔楼体型与主方向角各异，使得连接体在风载和地震作用下的受力状态较为复杂。此外，塔楼间的不均匀沉降亦会在刚性连接体内产生附加内力。基于这些因素，连接体遵循了以下的设计思路并采取了相应的加强措施。

首先，塔楼的刚度差异越大，连接体的内力也越大，通过调整三塔刚度比，优化各单塔在连体高度处的变形差，不仅能够有效控制连体结构的平扭耦合效应，也会显著改善连接体在协调塔楼不均匀变形时的受力状况。

其次，连接体进行抗震性能化设计，提高重要构件的抗震性能目标（图17），同时对主要桁架体系进行抗连续性倒塌分析，进一步提高主体结构的安全度；对连接体楼层楼板进行加强，楼板厚度取200mm并根据风载和地震作用下的楼板应力分析结果进行配筋，同时在连接体与塔楼相邻跨设置楼板面内水平支撑。

第三，为了减小塔楼不均匀沉降带来的连接体附加内力，三栋塔楼均选择变形模量较大的中风化泥岩层作为桩基持力层，桩端进入持力层不小于7倍桩径，并采用桩底后压浆技术。计算表明，本项目三栋塔楼最大沉降差约为10mm。同时根据施工模拟分析结果确定合理的施工顺序，对沉降差敏感性较高的连接体杆件采用延迟安装方案，从而有效减小或消除附加内力的影响。

振动台试验

为了检验结构的抗震能力，并对相关设计方法进行校核，项目在中国建筑科学研究院振动台实验室进行了模拟地震振动台试验。试验模型长度相似比为1/40，模型总高度为9.2m（图18）。

小震阶段选用7组地震波（5组天然波+2组人工波）进行三向输入；为减小试验过程中损伤累积的影响，中震阶段的试验从7组大震弹塑性分析地震波中，选取位移和基底反力较大的3组波进行三向输入；大震阶段，再从中震试验的3组地震波中，选择反应最大的1组地震波进行三向输入，更多试验信息见文献（China Academy of Building Research 2014）。

More information about the test can be found in document (China Academy of Building Research 2014).

Comprehensive data on the dynamic characteristics of the tests was recorded, such as accelerations, displacement responses, and the stresses of main components. The tests reveal that:

- Under the frequent earthquakes on y-direction, the different amplitudes of the three towers result in the translation-torsion hybrid movement as shown in Figure 4. The maximum story drift is 1/580, meaning the structure is in the elastic state.
- With the increment of earthquake intensity, the phase difference of the three towers' translational movements leads to the torsion of the whole connected structure; but there is no severe damages caused by the structural torsion. The damages concentrate on the concrete coupling beams, shear walls at the bottom, and on part of the columns.
- Under the rarer earthquakes, the structural story drift reached 1/95. The wholeness of the structure is well maintained by the sky lobby. In the meantime, there are no obvious damages in the stories neighboring the sky lobby, meaning the structural arrangement and corresponding reinforcement measures in this part are appropriate.
- The model was subjected to a rare earthquake of 7.5 degrees. The maximum story drift reached 1/66, but the vibration amplitude of the connected structure is far less than a single tower structure of equal height. The major lateral-force resisting members were not seriously damaged. The results indicate the ductility and deformation capacity of the structure are sufficient, and that the seismic capacity reserve is adequate.

Conclusions

The structural characteristics of the Golden Eagle Tiandi Plaza – a complex, connected structure with three super high-rise towers – are discussed in the paper. Some important conclusions are as follows:

- The vibration modes for unsymmetrical connected structures are translation-torsion hybrids. The stiffness of the towers should be optimized to decrease the torsional component of the main translation modes.
- The phase differences of each tower's translational motion make the torsional response of the connected structure obvious; but the torsional mode of each tower is hard to trigger for the restraint of the rigid connecting structure. The results of a shake-table test show that there are no severe damages caused by the structural torsion, so the requirements on some indices for torsion control of the overall structure could be relaxed, but the torsional component in main translation mode and torsion control measures of each single tower should be strictly limited.
- The proportion of the whole overturning moment bourn by the towers' axial forces is a key parameter for the evaluation of the connecting degree. For the rigid-connected structures, when the connected part is at upper stories of the tower, the seismic capacity of the whole structure is optimal.
- Sinusoidal excitation of each mode will amplify the resonance reaction of the structure, and the weakness of the structure can be thoroughly detected. The multi-modal sinusoidal excitation method can be used as a complementary analysis measure. It is helpful to improve the arrangement and reinforcement of the structure.
- The method for calculating wind loads presented in present code does not take the influence of the connecting structure and the towers into consideration, and fails to reflect the redistribution of wind loads accurately. It is not applicable for connected structures.
- The status of the sky lobby under wind loads and earthquakes is complicated. In order to improve the forced condition, the towers' stiffness should be primarily adjusted to reduce the deformation difference of different towers at the elevation of the connected stories. In addition, the settlement difference among the towers should be limited as well, and some strength reserves are necessary for the members that are sensitive to this settlement difference.

试验对结构的自振特性变化、加速度及位移响应、关键构件（核心筒、框架柱、伸臂桁架、连接体主桁架等）的应力进行了全面监测。试验得到以下主要结果:

- 在X向小震作用下，结构整体响应基本为平动；在Y向小震作用下，连体结构的各塔楼的响应以平动为主，各塔楼的振幅差异呈现出图4所示的平扭耦合振动，结构最大平均位移角为1/580；
- 随着地震输入的加大，三塔出现不同相位的平动，造成整体结构的扭转，但整个试验过程，均未出现单塔的扭转现象，结构的损伤主要出现在连梁、底层框架柱和部分剪力墙，扭转造成的结构损伤较小；
- 在7度大震作用下，结构损伤增加，最大层间位移角为1/95，刚性连接体保证了结构较好的整体性，满足抗震设防目标要求。同时，连接体及上下相邻楼层未出现明显的损伤，表明该位置的结构布置及加强措施是合理的；
- 4 模型还进行了超设计设防标准的7.5度大震检验，最大层间位移角达到1/66，但结构的整体变形幅度远小于常规单塔结构，结构保持了较好的整体性，且关键抗侧力构件损伤不严重，说明结构具有良好的变形能力和充足的延性，具有一定的抗震能力储备。

总结

本文对金鹰天地广场项目在结构布置，基本动力特性，抗震性能，风荷载特点、重要构件设计要点等方面进行了分析和探讨，得到了如下结论:

- 非对称刚性连体结构的基本模态影响因素复杂，各阶模态都含有一定的扭转分量，设计时可通过各单体的刚度比优化，减小整体结构的平扭耦合效应。
- 刚性连体结构中，由于刚性连接体的约束作用，各塔楼抗扭刚度远大于单塔结构，且连体结构实际由于扭转效应造成的地震损伤也较小。建议放松连体结构的扭转周期比限制条件，并降低结构的整体扭转比要求，重点控制连体结构各分塔楼扭转位移比。
- 连体结构的连接强弱程度可由各塔楼间产生的整体抗倾覆力矩比进行量化。对于刚性连体结构，当连接体处于各塔楼的中上部位时，结构的整体稳定性较高，抗震性能最优。

- 多模态正弦波激励法强化了结构在各阶模态下的共振响应，能够较全面地揭示结构的薄弱部位，可以作为常规动力时称分析的补充，为结构布置与加强提供参考。

- 由于连体结构各塔楼的相互影响，风荷载效应在塔楼间存在重分布现象，现行规范中的计算方法无法准确地反映这种重分布现象，因此对于连体结构是不适用的。

- 刚性连体结构的连接体实现了各塔楼的共同工作，受力状态较为复杂，在设计时应首先控制塔楼在连体位置的变形差异，减小连接体在水平荷载作用下的内力，并对连接体楼板体系予以加强，保证水平力的有效传递。同时，应采取措施，控制各塔楼的不均匀沉降，提高对不均匀变形敏感构件的安全储备。

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