

Title:	Curtain-Wall System Design
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Subjects:	Building Case Study Façade Design Structural Engineering
Keywords:	Façade Structure
Publication Date:	2014
Original Publication:	Shanghai Tower: In Detail
Paper Type:	1. Book chapter/Part chapter 2. Journal paper 3. Conference proceeding 4. Unpublished conference paper 5. Magazine article 6. Unpublished

Curtain-Wall System Design

幕墙系统设计

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A unique flexible suspended curtain wall support structure (CWSS) was ultimately developed to suit the exterior curtain wall system of Shanghai Tower. Because of the large and varying distance from the exterior curtain wall to the main structural system, the flexible suspension curtain wall support structure system and construction mechanical behavior of CWSS is complicated. So, in the preliminary design, feasibility of construction and structural behavior of CWSS during construction were analyzed in detail and adjustments and optimization were carried out based on the results. Optimized designs of joints, structural systems, and structural behavior of CWSS were introduced. The analysis and design method used in this project could be a reference for the analysis and design of similar projects.

上海中心大厦外幕墙系统采用分区悬挂的柔性幕墙支撑结构，由于主体结构超高，幕墙支撑结构远离主体结构，尺度巨大，且采用悬挂系统，使其施工建造力学行为复杂。因此设计阶段针对幕墙结构系统建造的可行性及施工力学行为进行了详细分析，对幕墙支撑结构进行了优化设计。对幕墙支撑结构体系、节点构造的优化设计，以及施工过程中对幕墙支撑结构的结构行为的影响进行了详细介绍。相关设计和分析方法可为类似工程提供参考和借鉴。

Introduction

Shanghai Tower is one of the first 600m high-rise buildings, which uses a unique “double-skin” curtain wall system. The twisted and tapered outer curtain wall system, which is far away from the main structure, is one of the significant features separating it from other high-rise buildings. In order to provide firm support for the outer curtain wall, and to ensure its visual transparency, a new curtain wall support structure, which is named “flexible suspension curtain wall support system,” is used for achieving the twisted façade of the outer curtain wall.

The unique structural characteristics, and the connected relationship between the main structure and the curtain wall in Shanghai Tower, make its design and construction much more difficult than the curtain wall projects of general high-rise buildings. For this reason, the buildability of curtain wall systems was analyzed and researched during the construction design stage. Optimization of the design of the curtain wall system support structure and details of joints, as well as the effects of the construction process on CWSS, are introduced in this chapter.

Curtain Wall Support Structure System

The Shanghai Tower curtain wall system is divided into nine zones from bottom to top. The flexible suspension curtain support structure system running from zone 2 to zone 8 is the standard structural layout for the exterior curtain walls of Shanghai Tower. The Shanghai Tower standard curtain support system is the spiral structure system, consisting of hoop ring beams and radial struts, as well as high-strength hanger rods (see Figure 3.18).

Hoop ring beams ($\Phi 356 \times 22 \sim 25$) are set vertically every 4.3 to 4.5 meters to fix the curtain plates and convert the gravity and wind loads suffered by the curtain wall. A horizontal radial strut ($\Phi 219 \times 13$, $\Phi 273 \times 13$) is set along the peripheral hoop ring beam every 8-10 meters to link the ring beams to the tower interior floors. While the connection to the ring beam is rigid, the connection to the slab edge beam is hinged to allow the exterior curtain walls to move up and down separately from the floors, and to reduce movement in the radial strut caused by vertical movements of the rods.

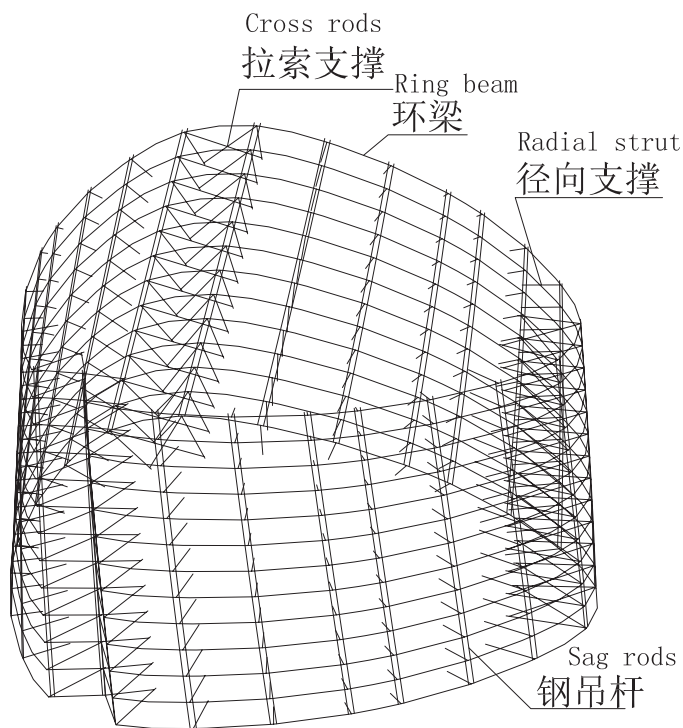
引言

上海中心大厦是中国设计建造的第一幢600m以上的超高层建筑，该项目采用了独特的双层幕墙系统，其中远离主体结构且扭转收缩的分区悬挂的外幕墙系统是这栋建筑区别于其他超高层建筑的显著特点之一。为对远离主体结构且扭曲上升的外幕墙提供坚实的支撑，并保证幕墙在视觉上的通透性，该项目首次在超高层建筑上采用了一种新型的“柔性分区悬挂式的幕墙支撑系统”用于实现外幕墙扭曲的外立面几何形态。

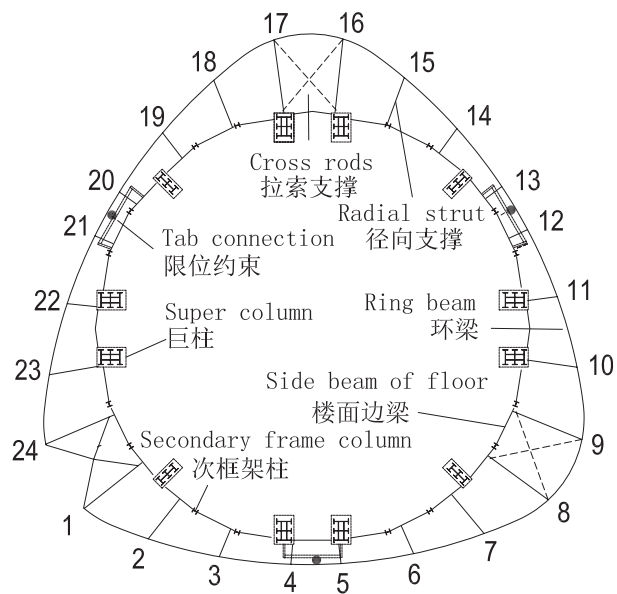
独特的结构特点及其与主体结构的连接关系，使其施工建造的难度远大于一般的超高层幕墙工程。为此，本项目在施工图设计阶段就对幕墙系统的可建造性进行了充分的分析和研究，本文对幕墙支撑结构体系、节点构造的优化设计，以及施工过程中对幕墙支撑结构的结构行为的影响进行了详细介绍。

幕墙支撑结构体系

上海中心大厦外幕墙系统从下到上共分9个区段，其中，2~8区的悬挂式柔性幕墙支撑结构体系，为上海中心大厦外幕墙的标准结构形式。上海中心大厦外幕墙标准的幕墙支撑结构体系为由水平钢管环梁和径



Isometric drawing
轴测图

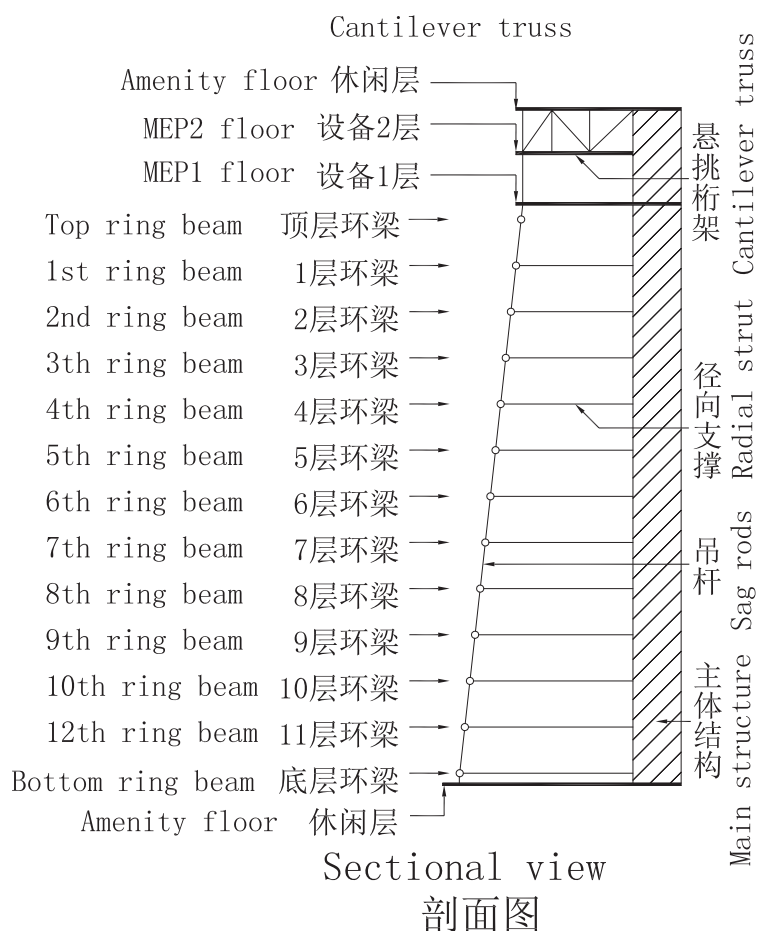


Plan
平面布置图

Two high strength (460MPa) rods ($\Phi 60 \sim 80$) are arranged at the crossing of radial struts and ring beams. All ring beams at every level are suspended from the mechanical floor at the top of CWSS by these rods.

To resist the torsion effect of the curtain wall system, three tab connections are arranged where hoop ring beams are tangential with the cylinder floor in each floor and three cross rods ($\Phi 60$) are arranged at the corners of the ring beams in each floor.

The hoop ring beams at the bottom of each zone are set into vertical bushes (see Figure 3.19) and embedded in the slab of amenity floor through the pole. The connection provides lateral support to the hoop ring pipes, while allowing the structure to move within a certain range of vertical deformation and rotation. The CWSS can move relatively independently from the main structure of the tower. In order to prevent a net compression stress in the rods, the first level hoop ring beams above the amenity floor is filled with concrete. This can provide sufficient weight to keep tension in the rods to avoid compression instability.



Sectional view
剖面图

Figure 3.18. Structure System and position number (Source: TJAD)
图3.18. 结构体系及位置编号 (来源: TJAD)

Optimization Design of Structural System

The CWSS is a closed ring, the longest total perimeter of which is approximately 300m. Its expansion deformation is restricted by radial struts, which will lead to temperature stress in structure.

In the preliminary design stage, some expansion joints were arranged in the ring beam in order to reduce temperature stress (see Figure 3.20). Structural analysis also showed that comparing the ring beam with no expansion joints, the axial forces and moments of ring beam with expansion joints is reduced from 536kN, 249kN moment to 2kN, 8kN moment respectively. The axial forces and moments of radial strut is also reduced from 522kN, 137 kN moment to 3kN, 1kN moment respectively. As a result, it is evident that expansion joints reduced the internal force significantly.

However, some problems are also brought about with expansion joints in the ring beams, namely: 1) The deformation of expansion is 30mm under temperature, which is too large to absorb for the connection detail for glass plates. This would bring about leakage in service. 2) Because of expansion resulting from temperature change, it would be difficult to locate the glass plates during construction of the CWSS. 3) Axial stiffness of ring beams would not be continuous, therefore torsional stiffness is low. 4) Cost of the expansion joints and detailed design and processing would be difficult. These would have a significant effect on construction progress and methodology.

In summary, utilization of expansion joints would bring about several problems which would be difficult to overcome, such as cost increases, detailed design complexity, poor constructability and serviceability of the CWSS. Therefore, the continuous ring beams were adopted as the final structural scheme and no expansion joints were used for the ring beams.

Optimization Design of Vertical Sliding Joint in Bottom Ring Beam (VSJBRB)

VSJBRBs are used to absorb vertical relative deformation between the CWSS and the main structure. These are located between the bottom of the CWSS and the amenity floor. The VSJBRBs prevent glass plates in the bottom of curtain wall from being damaged by vertical stress.

The original design for the VSJBRB was a rigid structure. It carried moment and shear along radial direction and tangential direction of ring beams. The stresses in the bushes, generated by joint moment and shear, forms friction, which can prevent the joint from sliding. That is, when the applied force on the structure is less than the friction, the joint may be self-locking.

The large cantilever structure, and heavy load and deformation of the amenity floor and MEP floor lead to comparatively large additional internal forces in the VSJBRB. Analysis showed that among the additional internal forces, the additional moment along tangential direction is the biggest, which is caused by the installation of the VSJBRB being later than the load application.

Because the deformation of floor after installation of the VSJBRB is the main reason for self-locking of the joint, some measures were adopted to reduce its effect in construction. 1) Pre-loading was applied in order that deformation of floor occurred before VSJBRB installation. 2) In the curtain wall construction process, the bottom ring beam was fixed by temporary measures to release floor deformation during construction. After these floor deformations occur, the VSJBRB was finally installed.

Further analysis showed that, the above method had poor operability, took a long time to construct and was uneconomical. Analysis of the adverse effects of internal forces and the

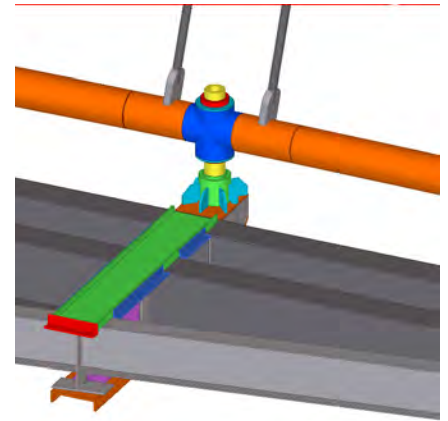


Figure 3.19. Vertical sliding joint in bottom ring beam (Source: TJAD)

图3.19. 底环梁竖向伸缩节点 (来源: TJAD)

向水平钢管支撑以及高强拉杆组成的螺旋上升的结构体系 (见图3.18)。

环梁采用直径356mm钢管, 沿竖向每层(4.3~4.5米) 布置以承受幕墙板块的重量和水平风荷载。沿环梁每8至10米设置一道水平径向钢管支撑 ($\Phi 219 \times 13$ 、 $\Phi 273 \times 13$) 将其与主体楼面结构连接, 径向支撑与环梁刚接连接。楼板边梁的连接采用铰接以允许外幕墙相对于楼板上下运动。

在每个环梁和径向水平钢管支撑相交的位置设置两根屈服强度460Mpa的高强度钢吊杆 ($\Phi 60 \sim 80$) 将每区的所有水平环梁吊至上部机电层辐射桁架的悬挑端。

为约束幕墙相对主体的扭转, 在每层水平曲梁与圆柱体楼面相切的位置布置了三个限位约束, 在角部设置了交叉拉杆支撑 ($\Phi 60$)。

每区最下方的环梁上设置竖向轴衬, 并通过立杆嵌入休闲层楼板中 (图3.19)。这样的连接方式为周边曲梁提供了侧向支撑, 但同时允许结构在一定范围内的竖向变形和转动, 使得幕墙系统与塔楼主体结构可以相对独立的自由变形。为了防止吊杆出现净压力, 位于休闲层以上第一道环梁内灌混凝土, 以保证其有足够的重量使吊杆保持受拉, 从而避免吊杆失稳。 (Ding et al, 2013)

结构体系优化设计

幕墙支撑结构为封闭的环形结构, 环向总周长最长接近300m, 径向支撑的存在限制了环梁的自由伸缩变形, 在结构内部产生较大温度应力。

幕墙结构初步设计阶段, 曾考虑在幕墙环梁中设置伸缩节点 (图3.20), 释放环梁的温度应力。结构分析也表明: 温度作用下, 与无伸缩节点的结构相比, 理论上, 环梁设置伸缩节点后, 环梁轴力和弯矩由

working mechanism of the bottom ring beam showed that the primary adverse internal forces on the VSJBRB was the moment along tangential direction of ring beam but the constraint along this direction is not a requirement of the geometric performance of the bottom ring beam. Therefore, the final solution was the optimizing of the joint structure, by releasing the constraint of joint along tangential direction. This reduced the adverse moment caused by floor deformation and prevented the self-locking of joint, to ensure satisfactory serviceability of the curtain wall system.

Analysis and Control on Deformation of CWSS

Because CWSS is suspended from the MEP floor, the stiffness of which is non-uniform, the weight of CWSS makes the MEP floor deform non-uniformly. Large non-uniform deformations would lead to breakage of glass plates in the curtain wall system, and would make the ring beam offsets predetermined from the elevation too high and too low for satisfactory visual appearance. These would also have an effect on installation and the serviceability of the glass plates. Therefore, a construction simulation analysis was necessary to determine effective control measures that should be adopted.

Construction of the entire curtain wall system is composed of CWSS erection and glass plates installation. The sequence of CWSS erection is from top to bottom and, after CWSS erection is

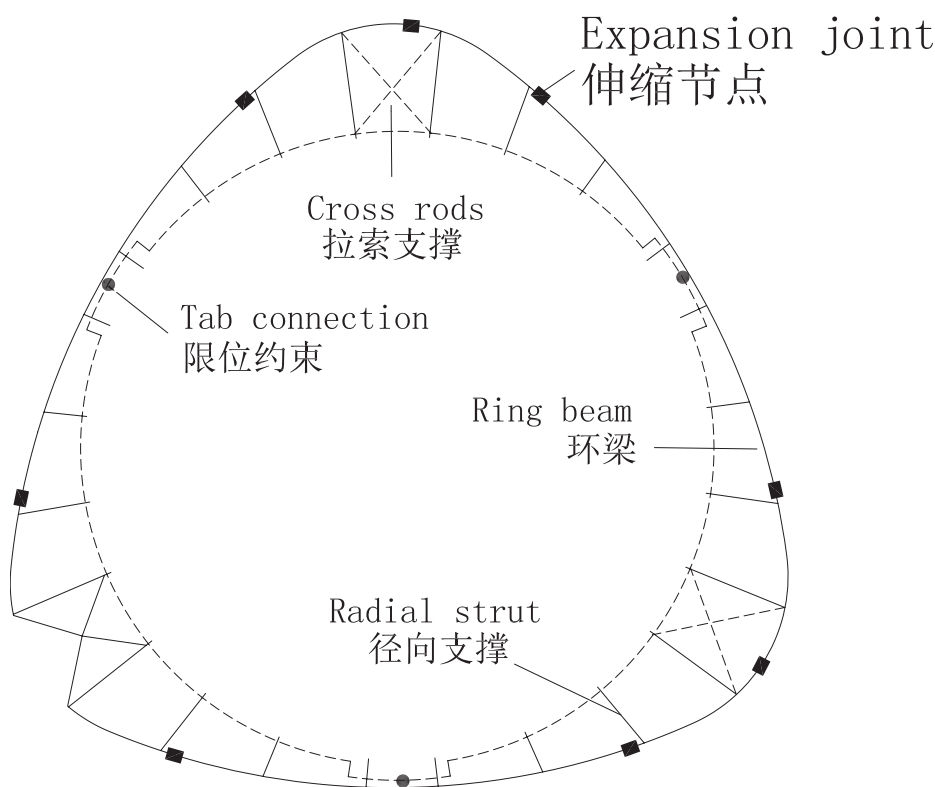


Figure 3.20. Plan of CWSS with some expansion joints (Source: TJAD)
图3.20. 有伸缩节点幕墙支撑结构典型平面布置 (来源: TJAD)

536kN, 249kN·m 下降到2kN, 8kN·m; 径向支撑的轴力和弯矩由522kN, 137 kN·m下降到3kN, 1kN·m, 所有构件的温度作用效应大幅下降至可忽略不计, 伸缩节点对减少构件温度内力作用明显。

但设置伸缩节点也同时带来的一些问题: 1) 温度作用下, 伸缩节点位移达到30mm, 幕墙板块插接构造难以吸收该位移, 从而可能造成幕墙正常使用阶段漏水、漏气; 2) 幕墙支撑结构安装过程中, 由于环境温度变化, 伸缩节点发生的变形, 使幕墙连接件发生位移, 从而导致幕墙板块安装定位困难; 3) 环梁轴向刚度不连续、整体性削弱, 抗扭性能变差; 4) 大量采用的滑动节点造价高, 采购、深化、加工难度大, 其设计制造过程费时费力, 对结构正常的施工过程与进度影响很大。

综上, 由于伸缩节点的采用, 给幕墙支撑系统的采购、制作、安装; 幕墙系统深化设计、正常使用带来了众多难以克服的瓶颈问题。幕墙支撑体系最终取消了水平伸缩节点, 采用连续布置的环梁系统, 并对该体系进行详细的分析与设计。

底环梁竖向伸缩节点构造优化设计

底环梁竖向伸缩节点位于每区底环梁与楼面之间, 用以吸收幕墙支撑结构与主体结构之间的竖向位移差, 防止底层幕墙板块受压破坏。

原竖向伸缩节点采用刚接构造, 同时承受沿环梁径向和环向的双向的弯矩和剪力。在复杂内力作用下, 会引起节点内部立轴和套筒发生挤压, 过大的挤压力将使节点内摩擦阻力过大而发生“自锁”, 无法滑动。

由于竖向伸缩节点位于主体结构休闲层和底环梁之间, 设备层和休闲层为大悬挑结构、荷载重, 楼面变形大, 从而在竖向伸缩节点内引起较大的附加内力, 分析表明, 竖向伸缩节点的弯矩成分中, 比重最大的环向附加弯矩, 即是由于设备层附加、活的施加滞后于节点安装所引起。

由于幕墙底部楼面变形滞后节点安装是引起底部伸缩节点自锁的主要原因, 设计过程中曾考虑在施工中采取一些措施消除其影响: 1) 幕墙伸缩节点安装前, 对休闲层堆载, 使休闲层楼面变形先于伸缩节点安装

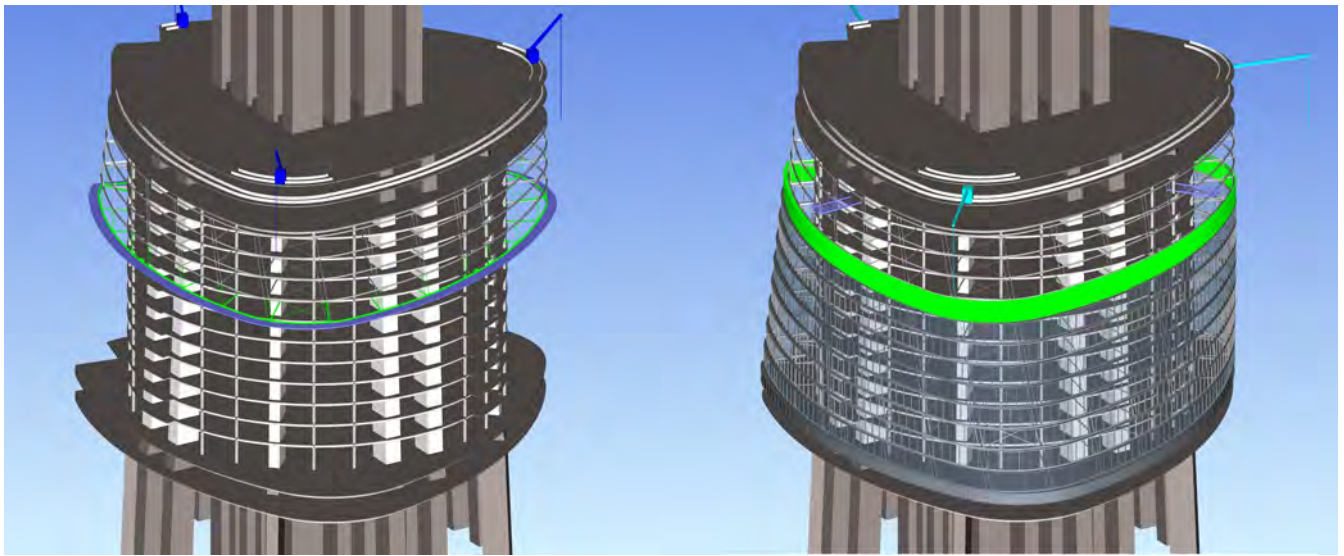


Figure 3.21. Sequence of outer curtain wall (Source: TJAD)
图3.21. 外幕墙施工顺序示意 (来源: TJAD)

finished, the glass plates are installed from bottom to top (see Figure 3.21).

A typical zone of the CWSS (zone 2) was chosen to carry out the construction simulation analysis. The MEP floor was modeled in order to reflect the effect of the stiffness of the MEP floor. The construction simulation was divided to 26 steps. Step1~13 are CWSS erection and the others are glass plates installation.

Point 20&16 were chosen for the introduction of vertical deformation. The cantilevered length of point 20 is smallest and its vertical stiffness is largest. Point 16 is an opposite condition.

If traditional methods of construction, in which the ring beam is calibrated level by level, were used, the elevation of ring beam would not be at the correct level when the CWSS is finished. For example, the max offset value of point 16 is 36mm, but the max offset value of point 20 is only 14mm, due to the non-uniform deformation. Therefore, pre-adjustment of ring beam elevation should be carried out according to the results of the construction simulation (see Figure 3.22a).

Vertical displacements of the ring beam are composed of suspension point displacement caused by the weight of the CWSS and superimposed dead loads, live load of the MEP floor, and prolongation of the sag rod (see Figure 3.22b). Prolongations of the sag rods are almost the same at each position, but suspension point displacements are different at each position. Furthermore, the dominant factor at each position is also different. For the position of small cantilevered length, vertical displacement is mainly caused by prolongation of sag rod. However, for the position of large cantilevered length, vertical

前发生; 2) 幕墙安装过程中底环梁采取临时固定措施, 释放安装过程产生的楼面变形, 待结构加载变形发生后, 对伸缩节点进行终固。

但进一步分析表明, 上述方法可操作性差, 周期长, 且不经济。通过对阻碍滑动的有害内力的构成成分分析和底环梁的工作机理分析表明, 伸缩节点的有害内力主要为环梁环向弯矩 (V 口为双向弯矩), 而伸缩节点对环梁该方向的抗弯约束并非构成底环梁几何不变的必要条件。因此, 最终的解决方案为对伸缩节点构造进行优化, 释放其对环梁环向的抗弯约束 (V 口为双向释放), 从而消除楼面变形引起的节点有害附加弯矩, 改善节点受力, 避免节点“自锁”, 保障幕墙正常使用。(He et al, 2013)

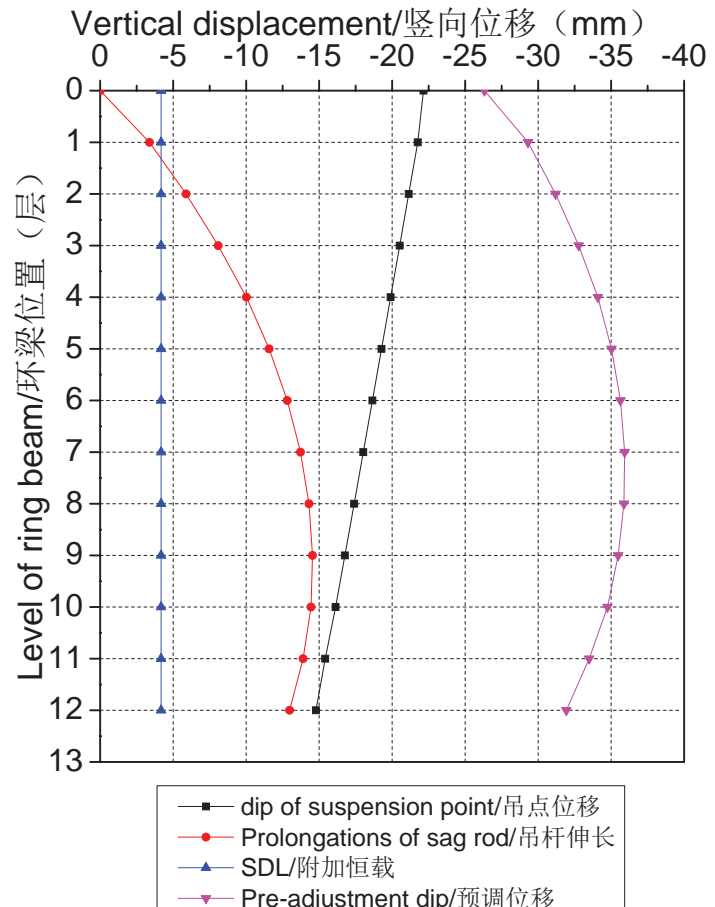


Figure 3.22a. Vertical deformation of ring beam – constitution of vertical deformation of point 16 (Source: TJAD)
图3.22a 环梁竖向变形 – 16号位置竖向位移构成 (来源: TJAD)

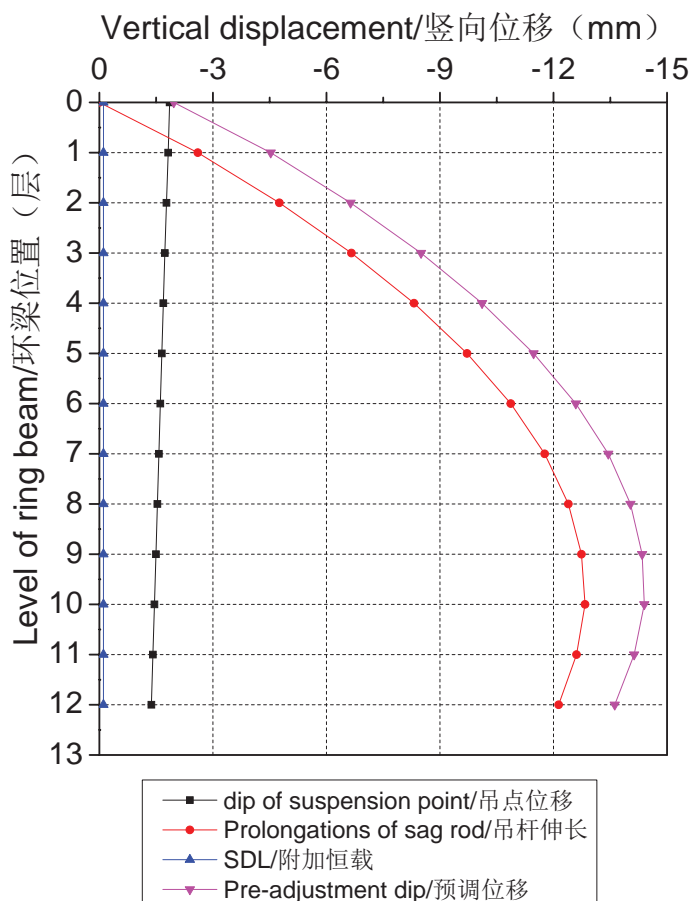


Figure 3.22b. Vertical deformation of ring beam – constitution of vertical deformation of point 20 (Source: TJAD)
图3.22b 环梁竖向变形 – 20号位置竖向位移构成 (来源: TJAD)

displacement is caused by both prolongation of sag rod and suspension point displacement. Therefore, the adjustment of elevation of ring beam is mainly resolved from the following two aspects.

1. Pre-adjustment of the length of sag rod. This method is used to solve the problem of off-levelness of the ring beam caused by prolongation of sag rod. The value of adjustment of sag rods decreases from top to bottom. The value of adjustment at the top is 3mm and less than 0.3mm at the bottom.
2. Pre-raising of suspension point. This method is used to solve the problem of off-levelness of the ring beam and reduce non-uniform deformation of the ring beam caused by the main structure deformation (see Figure 3.23). Figure 3.23 is theoretical pre-raising value of each suspension point. Due to the particularity of concrete, the stiffness contribution of concrete slab is difficult to accurately simulate for the MEP floor. When calculating the pre-raising values, the stiffness of the concrete slab isn't taken into account in order to consider the worst situation. Because the calculation results of the deformation are conservative, 60% simulation value is used for pre-raising in practice.

Considering the importance of displacements of the suspension points for the safety of the glass plates, actual on-site measurements are carried out for displacements of the suspension points. As shown in Figure 3.24, measured values are 43%~ 77% of the theoretical values, and the average is just 60%. Both are in good agreement. (Because the construction has not been completed and superimposed dead loads have not been applied, the displacement caused by superimposed dead loads is not included, but it is only a small proportion in the total displacement.)

Analysis on Effect of Compression of Main Structure on CWSS

The CWSS in each zone is 12~15 floors, 55~66 meters high, which is suspended to the structure of the MEP floor. Large compression of the super column in each zone has an effect on the slide range of the VSJBRB and additional moments in the radial strut caused by relative displacement

幕墙支撑结构变形分析与控制

由于幕墙支撑结构悬挂在主体结构刚度不均匀的弹性设备层上, 幕墙重量会使设备层发生不均匀的竖向变形, 过大的不均匀变形极易引起板块破碎, 同时也将导致幕墙系统各层环梁安装完成后偏离预定的设计标高, 使环梁外观不平整从而影响建筑外观及幕墙板块的安装及使用。因此, 非常有必要对幕墙支撑结构进行施工模拟分析, 并据此采用有效的控制措施。

整个幕墙系统的施工由支撑钢结构吊装及幕墙板块吊装两个阶段组成, 幕墙支撑结构采用由上至下逐层悬挂的方法吊装, 当幕墙支撑结构全部吊装连接完毕后, 再从底层环梁开始由下至上逐层安装幕墙玻璃板块 (图3.21)。

选取典型分区 (2区) 带有幕墙支撑结构的主塔楼整体模型, 对幕墙支撑结构进行施工过程模拟, 如图6所示。对塔楼的设备层准确建模, 以考虑楼面结构刚度对幕墙支撑结构的影响。(计算中未考虑混凝土楼板的刚度贡献) 幕墙的施工过程模拟为26个施工步, 1~13施工步为从上至下安装环梁, 环梁施工时逐层找平, 14~26施工步为从下至上悬挂幕墙板块。

选取位于凸台位置的悬挑长度较小的20号位置 (吊点竖向刚度大) 和位于角部的悬挑长度较大的16号位置 (吊点竖向刚度小) 对环梁的竖向变形特征进行说明, 各层16, 20号位置在环梁上的位置如图7所示。

若环梁采用逐层找平的传统方法施工, 施工完成后, 环梁的竖向位移将使幕墙的几何位置偏离设计标高, 16号位置最大偏移值达到36mm, 而20号位置竖向位移仅为14mm, 沿环向各点位移值分布极不均匀。因此, 需结合分析结果对环梁标高进行预调整 (见图3.22a)。

由图3.22b可以看出, 幕墙环梁的竖向位移主要由幕墙自重引起的吊点位移, 设备层附加恒活引起的吊点位移, 以及吊杆伸长组成。各个位置吊杆伸长相同, 但吊点位移不同, 且各个位置变形的主导因素也并不相同, 对于悬挑较小位置 (20号位置), 变形主要由吊杆伸长引起, 而对于悬挑较大位置 (16号位置), 顶部吊点竖向位移和吊杆伸长对环梁竖向位移均有较大影响。因此, 对环梁标高的调整主要从以下两个方面解决:

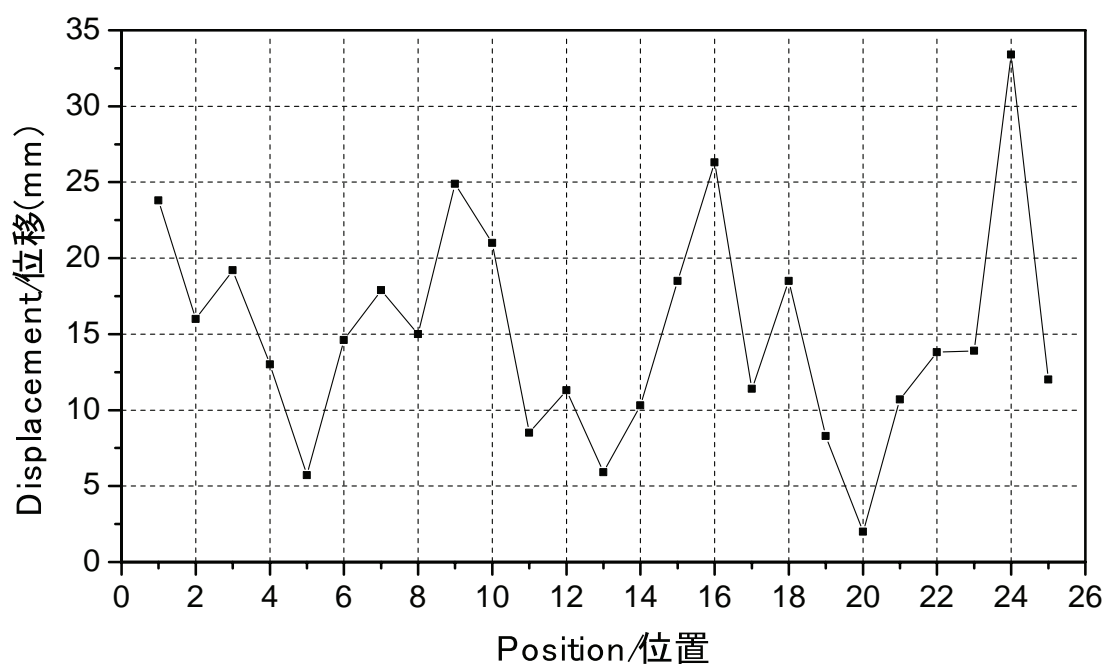


Figure 3.23. Pre-adjustment of suspension point (Source: TJAD)
图3.23. 各吊点标高调整量 (来源: TJAD)

between the main structure and ring beams of the CWSS.

Compression of super columns is composed of two parts. One is elastic deformation caused by gravity and the other is creep and shrinkage of concrete increasing with time. Because of long construction periods, vertical compression will continue to develop after the curtain wall is installed in the low zone. Therefore, the adverse effect of compression of super columns was evaluated in the design stage according to the construction sequence for the main structure.

Calculation of Compression of Main Structure

Compression of the main structure after 50 years is shown in Table 3.7. As shown in table 3.7, non-elastic deformation is about 73%~97% of the total compression deformation. It indicates that creep and shrinkage have a significant effect on the CWSS.

Due to assumptions made in the estimation of creep and shrinkage and error in the calculation of model, the derived value of compression deformation of the main structure will have some error and consequently is an approximate estimate. In design, the coefficient of variation

1. 吊杆长度预调整。通过预调吊杆长度，可以解决由于吊杆自身伸长使环梁偏离设计标高的问题。吊杆调整量由顶部的至下逐渐递增，顶部吊杆调整量约为3mm，在底部不足0.3mm。
2. 吊点预抬高。通过调整吊点标高可以调整由于主体结构变形引起的环梁偏离设计标高的问题，并改善环梁平整度。图3.23为各个吊点的理论预调整量，由于混凝土材料的特殊性，混凝土楼板对于设备层的刚度贡献难以准确模拟，因此在计算吊点调整量时，未考虑混凝土楼板刚度贡献，以考虑最不利的变形情况。由于变形计算结果偏于保守，因此实际操作中，按模拟数值的60%进行吊点预抬高。

考虑到吊点位移对于玻璃安全的重要性，施工中对吊点位移进行了监测，如图3.24

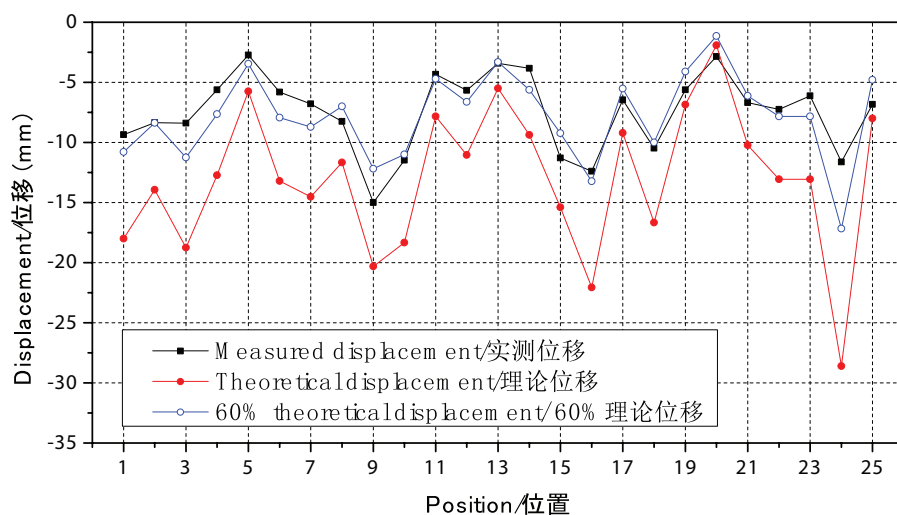


Figure 3.24. The actual measured displacements VS. the theoretical displacements of suspension points
图3.24. 吊点计算位移与实测位移对比 (来源: TJAD)

Zone 区号	Elasticity deformation 弹性变形	Shrinkage 收缩变形	Creep 徐变变形	Total 总计	Total (95% guaranteed rate) 总计 (95%保证率)
8	0.7	15	8.4	24	37.9
7	2.3	12.7	11.9	27	40.8
6	3.7	12.2	14.8	30.6	45.5
5	6.1	11.7	15.6	33.3	48.3
4	8.2	10.4	15.8	34.4	48.5
3	8.9	8.9	17.7	35.5	49.4
2	8.9	7.3	17.1	33.3	45.9

Table 3.7. Relative deformation of two ends of super column in each zone after construction of curtain wall (50 years) /mm
表3.7. 幕墙施工后巨柱两端相对变形 (50年) /mm

of the deformation is considered to adjust the results of design value. The adjusted results of compression of super columns are shown in table 3.7, for 50 years after completion of construction. The maximum value of compression deformation reached 49.4mm, which had a significant effect on detail design of slide joint.

Calculation of Relative Deformation Between Main Structure and CWSS

Compression of the main structure leads to relative displacement between the ring beams and floors. Different displacement distributions are shown at different positions (see Figure 3.25). The maximum displacement difference reaches about 48mm, while the smallest displacement difference is less than 5mm. All larger displacement differences, which are almost the same at different levels, are near the secondary frame column such as points 3, 9, 15(see Figure 3.18) and displacement differences which increase regularly are near the super column such as points 8, 14, 20 (see Figure 3.18).

The reason for the difference in the displacement distributions shown in figure 3.25 is as follows. The side beam at each level is hinged to the secondary frame column and the super column; furthermore, the secondary frame columns do not shorten when the super column is compressed. Therefore, when the ring beams drop 45.9mm caused by super column compression, the 45.9mm relative displacement difference will occur. The 45.9mm is also the value of the super column compression but the inner end anchors of the radial strut near the super column drop with the same value as the super column when it is compressed. However, the vertical displacement of the upper radial strut is larger, due to the longer accumulated length of the super column. Therefore, its relative displacement compared with the ring beam is smaller because the value of compression of the super column at each level is the same. Consequently, the relative displacement differences of the radial strut near to the super column increases from top to bottom.

Analysis on Effect of Internal Forces of Cwss Caused by Vertical Deformation of Main Structure

Normal curtain wall structures are usually static structures. Deformation of the main structure does not usually cause additional internal forces. However, the CWSS of the Shanghai Tower is different due to its 60 meter height and being suspended.

Comparisons of additional moments caused by the super column compression deformation and gravity loading of the curtain wall are shown in Figure 3.26. Larger additional moments are

所示。实测数值约为理论计算数值的43%~77%，平均恰好为60%。(由于施工尚未完成，设备层的附加恒载尚未施加，因此图中位移量未包含其影响，但该位移量在总位移中所占比重较小)

主体结构竖向压缩对幕墙支撑结构的影响分析

每区幕墙支撑结构12~15层，约55~66m高，悬挂在上一区段楼面结构上，区间巨柱压缩量大，将影响幕墙系统底部竖向伸缩节点滑动行程的确定，同时还会使相应楼层环梁与主体结构楼面产生相对位移并由此引发连接环梁与楼面的径向支撑产生附加弯矩。

巨柱的区间压缩由两部分组成，一部分是由重力荷载产生的弹性变形，另一部分是随时间增长的混凝土收缩、徐变产生的非弹性变形。而主体结构的施工周期长，低区幕墙开始施工时，主体只施工到4区，竖向压缩变形还将继续发展。因此，在设计阶段就结合主体结构的施工顺序，对巨柱压缩产生的不利影响进行评估。

主体结构区间压缩量的计算

表3.7为各区巨柱50年后的竖向压缩量，由表3.7可知，在巨柱总的区间压缩变形中，非弹性变形所占比重较大，约为73%~97%，说明收缩徐变对幕墙影响很大。

由于徐变和收缩变形固有的离散性、模型的误差以及由材料性质和环境的随机性引压缩量计算可能有一定的误差。设计时，对分析结果考虑一定变异系数，按结构的设计使用年限为50年，95%保证率对结果进行调整，则竣工50年后巨柱的压缩量最终可确定为表3.7所示结果。区间最大压缩量达到了49.4mm，对底环梁竖向伸缩节点滑动构造影响较大。

幕墙支撑结构与主体结构楼面相对变形计算

巨柱压缩导致环梁与对应楼面的产生相对位移(图3.25)。各个位置径向支撑与楼面间的位移差呈现大小相间的分布规律，最大位移差可达48mm左右，而最小位移差仅为不足5mm。图中位移差较大且各层位移差大小基本相同的位置均位于楼面次框架柱附近(图3.18)，如图中3号、9号、15号等位置，而位移差从高到低呈现渐增规

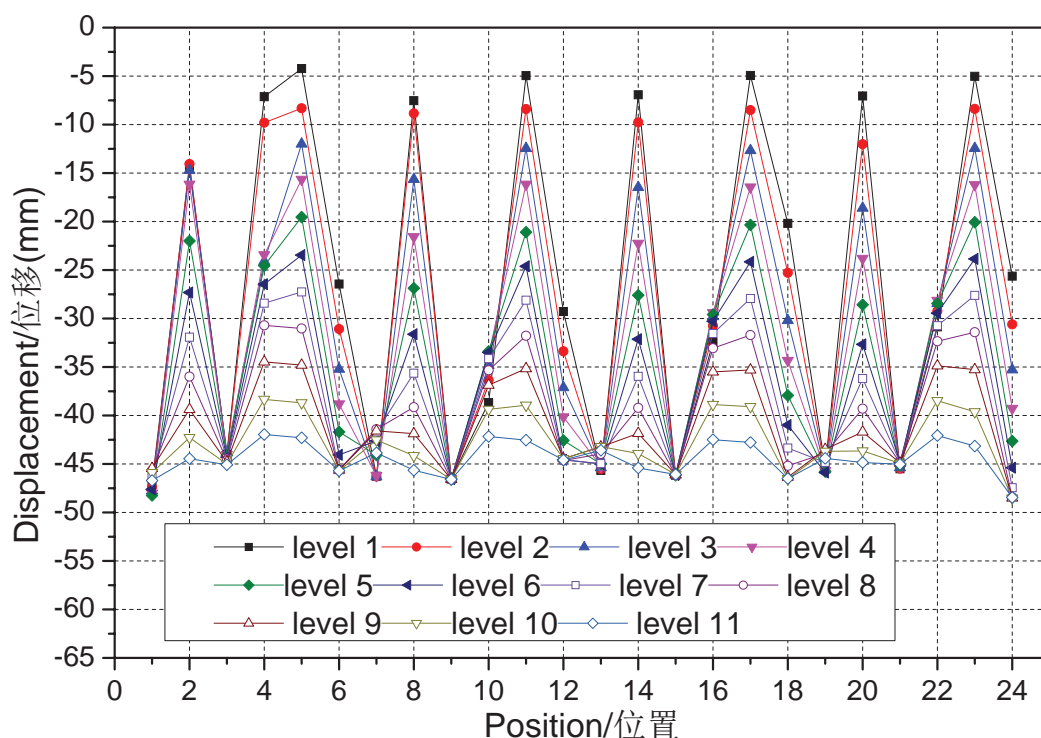


Figure 3.25. relative displacement between main structure of CWSS (Source: TJAD)
图3.25. 环梁与主楼相对位移差 (来源: TJAD)

produced under the 45.9 mm super column compression. The maximum additional moment is 42kN moment at point 3, which is equivalent to 30% of the moment capacity of the radial strut. This is much larger than the 15kN moment when compression of super columns is not considered. The analysis suggests that additional moment in the radial support, caused by the compression of super columns, increases the moment significantly. Therefore, it is necessary that compression of super columns is considered in the member strength design of the CWSS.

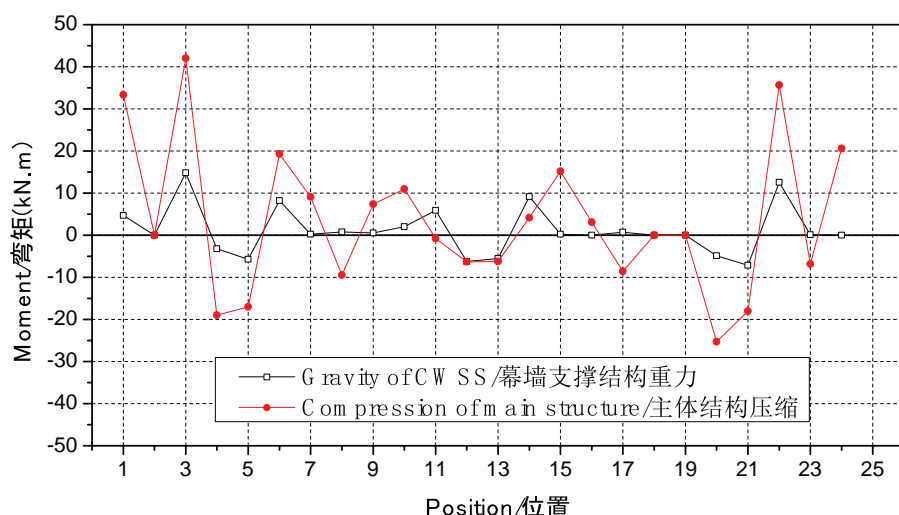


Figure 3.26. Comparison of additional moments caused by super column compression and gravity of curtain wall (Source: TJAD)
图3.26. 巨柱压缩与幕墙自重引起的径向支撑附加弯矩对比 (来源: TJAD)

律的位置则位于巨柱附近如8号、14号、20号等位置 (图3.18)。

产生图3.25中所示位移差分布规律的原因是由于楼面边梁在巨柱与楼层次框架柱间铰接, 当本区巨柱发生竖向压缩时, 次框架钢柱并不会受其影响发生竖向缩短, 所以, 当外围幕墙环梁在巨柱压缩带动下发生45.9mm沉降时, 即与楼面次框架柱间出现了45.9mm的竖向位移差, 也即是巨柱的区间压缩量。而位于巨柱附近的径向支撑其内端支撑点会在巨柱的带动下发生竖向沉降, 越位于上部的径向支撑由于巨柱累计长度较长其竖向位移也就越大, 因而与外围的环梁位移差也就越小, 当巨柱截面在区间不发生变化时, 各层巨柱压缩量一致, 因此位于巨柱附近的径向支撑两端的位移差也就呈现从上到下逐渐递增的等差数列。

主体结构竖向变形对幕墙支撑结构的内力影响分析

常规幕墙结构作为静定的次结构, 主体结构变形一般不会引起其次内力, 而上海中心幕墙支撑结构作为周长近300m, 高度60m的悬挂结构, 主体结构的变形将在幕墙支撑结构的径向支撑中产生附加内力。

图3.26为巨柱压缩与幕墙系统自重引起的径向支撑的附加弯矩的对比。如图3.26所示, 45.9mm的巨柱压缩量下径向支撑产生了较大的附加弯矩, 附加弯矩最大的位置

Conclusions

Because of large scale, special structural systems, and heavy loading and complicated interaction with the main structure, the design and buildability of the CWSS of the Shanghai Tower was researched, especially for the following aspects:

1. The CWSS without expansion joints is easy to construct, install and has satisfactory service performance and also enables cost savings.
2. The hinge structure in the single direction for VSJBRB improves the mechanical properties of structure, speeds up the construction of the joint and simplifies the construction process.
3. Pre-adjustment of the elevation of the suspension joint and pre-adjustment of the length of the sag rods according to the construction simulation analysis ensures that the geometry of the curtain wall will meet the visual requirements when construction of the curtain wall is complete.
4. The effect of compression deformation of the main structure on the CWSS was evaluated in order to guarantee safety in service and strength capacity of the CWSS.

发生在3号位置，最大附加弯矩为42kN.m，约为径向支撑抗弯承载力的30%。远大于不考虑巨柱压缩时，幕墙自重作用下径向支撑的附加弯矩15kN.m (此处支撑长3m)。

这表明巨柱压缩引起的各个位置环梁与楼面间不均匀的竖向变形差会显著增加径向支撑的附加弯矩。因此，构件强度设计时考虑巨柱压缩引起的影响是非常有必要的。

结语

上海中心大厦外幕墙支撑结构体系特殊，尺度大，荷载重，幕墙结构与主体结构相互作用复杂，对其可建造性相关问题进行了专项研究。

1. 幕墙支撑结构体系采用无伸缩节点方案，便于施工、安装、维护，并大幅度节省造价。
2. 竖向伸缩节点采用单向铰接构造(V口双向铰接)，改善了结构受力，消除了施工过程对节点受力的影响，简化了施工过程。
3. 结合幕墙支撑结构施工过程分析，采用吊点标高调整和吊杆长度调整措施，保证了幕墙安装后几何形态满足建筑视觉要求。
4. 对主体结构压缩变形对幕墙支撑结构带来的不利影响进行了评估，保障了幕墙支撑结构的安全承载和正常使用。

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