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Structural Design and Construction of Hyundai-Hyperion Project, Seoul, Korea

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Abstract

Hyundai-Hyperion Project is a residential development located at Mok-dong, Seoul, Korea. The development comprises 3 residential towers ranging in height from 189m (54 stories) to 254 m (69 stories) above ground, nine levels of podium and six levels of basement for parking, retail area and club house and nine stories development store. The total floor areas of the different parts of the development are 225,600 m^2 (Towers), 41,000 m^2 (Podium), and 138,000 m^2 (Basement). The towers typically comprise a central concrete core with perimeter composite columns and floors. Two or three outrigger levels are located at approximately quarterly height of the towers to provide lateral stiffness. Hyundai Construction & Engineering Company has started the construction work in November 1999, and completed the works in July 2003. In this paper, Hyundai-Hyperion Project, which would be one of the tallest buildings in Korea, has been introduced. Key points in the structural design process and engineering characteristics were briefly reported. These include the effect of connecting the tower with podium, restraining effects of basement floor on tower, lateral stability system, wind and dynamic behavior, axial shortening prediction, and outrigger construction.

Keywords: Hyundai-Hyperion, Tall building, Lateral stability system, Column shortening

1. Introduction

Hyundai-Hyperion Project is a residential development located at Mok-dong, Seoul, Korea. The development comprises 3 residential towers ranging in height from 189m (54 stories) to 254m (69 stories) above ground, nine levels of podium and six levels of basement for parking, retail area and club house and nine stories development store. Fig. 1 and 2 show a front view and framing plan of tower.

The total floor areas of the different parts of the development are 225,600m^2 (Towers), 41,000m^2 (Podium), and 138,000m^2 (Basement). The towers typically comprise a central concrete core with perimeter composite columns and floors. Two or three outrigger levels are located at approximately quarterly height of the towers to provide lateral stiffness.

Fig. 1. Front View

The architect of this project is Yeh Art Group Inc., in

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Korea, and structural engineering services have been provided by Ove Arup & Partners International Limited and Dong Yang Structural Engineers Company. Hyundai Construction & Engineering Company has started the construction work in November 1999, and finished of work in the July 2003.

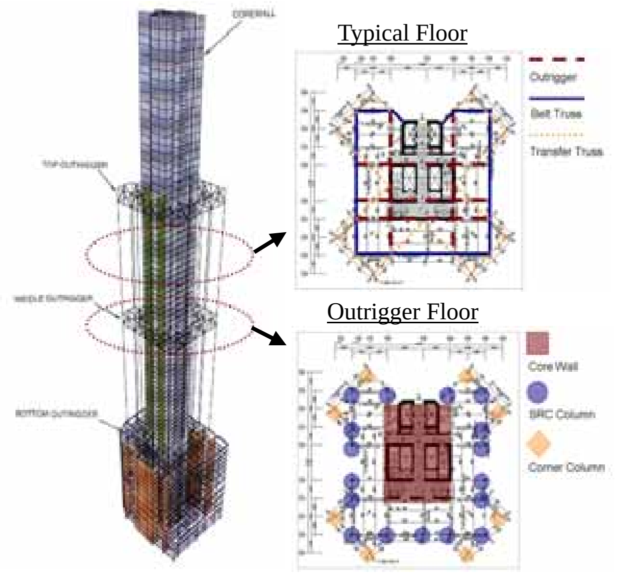


Fig. 2. Framing Plan of Tower

2. Effect of Connecting the Tower with Podium

The arrangement of the buildings is that the residential towers are located on the top of the podium structure. The layout of the podium is ‘L’ shaped and the towers are located approximately equal-distant from each other.

In general, there are two approaches to the design of the combined podium and tower arrangement. These are:

(a) The combined podium and tower arrangement is designed as a monolithic structure in which the behavior of the towers is very much influenced by the podium structure;

(b) The combined podium and tower arrangement is designed as separated structures by including appropriate movement joints with the podium.

For method (a), the dynamic effect of the towers as well as the thermal movement and shrinkage effect of the podium slabs would need to be addressed. For method (b), movement joints introduce a long term maintenance issue.

2.1. Dynamic Effect

The towers will tend to deform laterally by different degrees and with different natural periods. For the case where the podium is connected monolithically with the towers, large forces will be generated in the beams and slabs of the podium and towers when the towers are deforming 180° degrees out of phase (Fig. 3). Under this condition, the podium slabs will tend to restrain the towers via strut or tie action. In the likely event that the towers move in phase for some duration, then the restraining effect on the towers will be minimal except for the small contribution due to podium moment frame action.

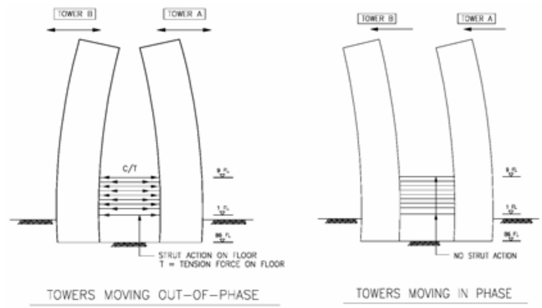


Fig. 3. Dynamic Effect of Towers

In summary, monolithically connecting the podium has the potential to increase the structural requirements in the podium slabs, but does not significantly increase the stability of towers.

2.2. Thermal Movement and Shrinkage Effect

The podium linking the towers is mainly open to the environment and service as parking space. Therefore, unlike other projects in which the podium is enclosed, the podium slabs in this instance will be subject to the full range of ambient temperature variation. The analysis to examine the effect of thermal movement and long term shrinkage on the slab has been carried out.

Temperature range was applied to as -11°C to +32°C for all slabs. Clearly the maximum forces will depend on the ambient temperature when the slab is cast. For the purpose of this exercise, the maximum expansion and contraction is based on the average of the above ranges. Shrinkage strain on the slab was applied to as 1.89×10^{-4}. Overall shrinkage effect could be reduced via delay joints. However for this exercise, the delay joint concept has not been allowed for in the shrinkage analysis. The effect on the towers themselves due to axial slab forces are represented in the Fig. 4, and numerical results of the analysis are shown in Table 1 in terms of the impact on the induced base moment.

2.3. Movement Joints

One implication of introducing movement joints is that the podium will have to be a freely standing structure. This has the benefit that differential vertical

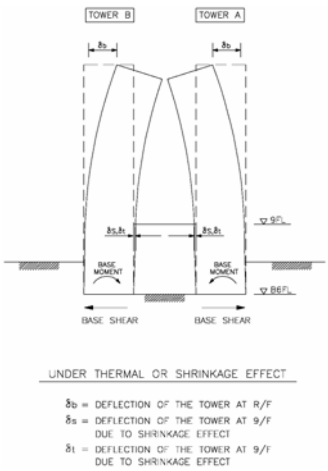


Fig. 4. Thermal Movement and Shrinkage

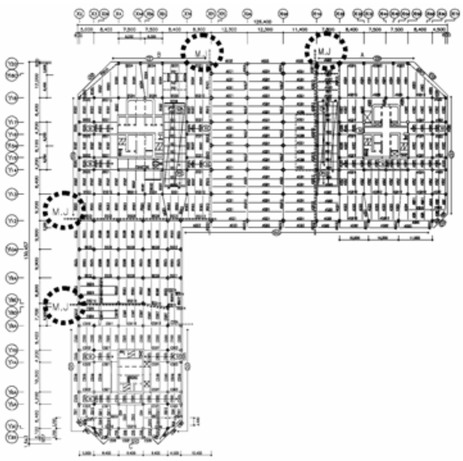


Fig.5. Movement Joint Locations

movements between the tower and podium can be easily dealt with. A delay joint be required in the podium slab to allow the differential axial shortening during construction.

As the basement slab be used to prop the basement walls, no movement joint be allowed in the basement slabs. The majority of the ground slab is covered, thereby sheltering it from thermal stresses and further removing the need for joints.

Table 2. Magnitudes of the Movement due to Different Load Cases

Load Case	Movement (mm)
Thermal	±10
Shrinkage	+8
Wind (X direction)	±61
Wind (Y direction)	±46
Wind (Torsion)	±10

3. Restraining Effects of Basement Floor on Tower

Referring to the floor plans of the basement, B1, B3 and B4 are the most appropriate levels in which the slab can be mobilized to restrain the towers without significant architectural planning implication.

Analytically, a 50m width of floor slab has been assumed as a beam strip in each orthogonal direction on B1, B3 and B4 levels to restrain the tower (Fig. 6).

Fig. 6. Assumed Beam Strip

Table 1. Numerical Analysis Results for Thermal Movement and Shrinkage

Loading	Additional Base Moment (kN-m)	Additional Base Shear (kN)	δ_t , δ_s (mm)	Additional Deflection at Roof, δ_b (mm)
Thermal Movement	383,100	6,038	8.6	50.3
Shrinkage	304,700	4,803	6.9	40.0

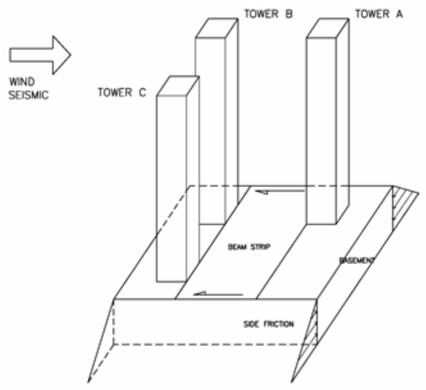


Fig. 7. Transmission of Beam Strip Forces to the Retaining Wall and Soil

The forces on the beam strip are transmitted to the earth by side friction of retaining wall and soil (Fig. 7). The thickness of the floor slab is determined to be 300mm.

The stiffness of the beam strip is calculated by simple beam theory. These stiffness of the beam strip have been back substituted to the analytical model as a ground springs to determine the influence on the tower, the force attracted by the basement slab as well as the forces on the foundation.

3.1. Effect on Tower

The restraining effect of the basement slabs can slightly reduce the total story drift of the building. More significant is the reduction in base overturning moment (Table 3).

3.2. Effect on Basement Slab

The in-plane forces induced on the basement slabs due to tower should be considered in conjunction with the propping force on the slab due to the earth load and the gravity loads on the floor. The magnitudes of the forces attracted on the individual basement slab and the amount of floor reinforcement required are summarized in the Table 4.

Table 3. Total Moment Resisted by Foundation

Direction of Wind	Without Restraining Effect (kN-m)	With Restraining Effect (kN-m)	Percentage Reduction
X direction	1,281,924	900,975	30%
Y direction	1,685,330	1,342,233	20%

Table 4. Forces Attracted on the Basement Slab and Amount of Floor Reinforcement

Level	Forces (kN)		Reinforcement (%)	
	X direction	Y direction	X direction	Y direction
B1	31,299	25,057	0.6	0.4
B3	6,152	5,773	0.2	0.08
B4	2,114	2,277	0.04	0.03

3.3. Effect on Foundation

As a result of the restraining effect of the basement slabs, the total moment resisted by the foundation is fairly reduced, and these are summarized in the Table 3.

4. Lateral Stability System of Tower

Lateral stability system comprises two or three outriggers and a reinforced concrete core with steel moment frame. In order to reduce the amount of moment at the core and lateral displacement, as shown in Fig. 8, outrigger is provides for this project.

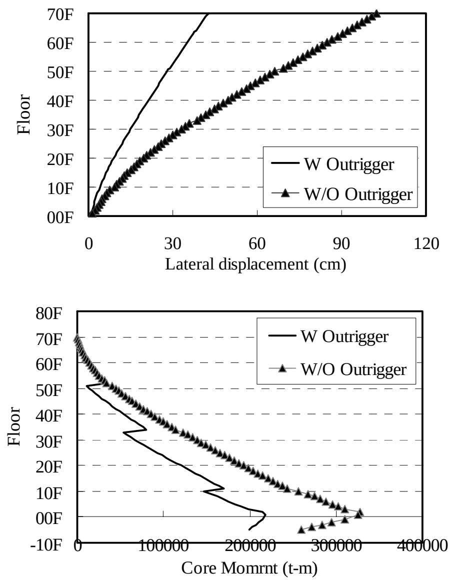


Fig. 8. Effect of the Outrigger

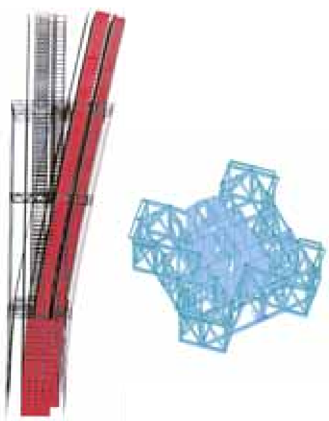


Fig. 9. Adopted Scheme

5. Wind and Dynamic Behavior of Tower

5.1. Acceleration Prediction According to NBCC

In advance of carrying out the wind tunnel studies, NBCC (National Building Code of Canada) prediction of the acceleration which are likely at the top of the tallest tower has been undertaken. The NBCC represents one of the most detailed and reliable means of acceleration prediction in the absence of wind tunnel data.

The mass of the building, including 25% live load, is equivalent to approximately 324 kg/m^3. The mass and building stiffness characteristics were used to determine the NBCC predicted acceleration for the wind speed of 10 years return period. The results are shown in the Table 5.

Table 5. NBCC Predicted Accelerations for Ten Years Return Period

Period (sec)	X direction Wind		Y direction Wind	
	Along Wind (milli-g)	Across Wind (milli-g)	Along Wind (milli-g)	Across Wind (milli-g)
6.8 (X dir.) 6.3 (Y dir.)	7	14	6	15

Table 6. Evaluated Peak Accelerations via Wind Tunnel Test

Tower	Direction	Peak Acceleration (milli-g)
A	X	11
	Y	10
B	X	14
	Y	7
C	X	11
	Y	10

5.2. Limiting Acceleration Criteria

Fig. 10 represents the limiting accelerations which should be adopted. The Fig. 10 shows the recommended NBCC limits for residential buildings together with the ISO Criteria and the Davenport Criteria for offices. From the structural engineer's international experience, it has been suggested that the following limiting values be adopted for different codified criteria.

(a) 80% of the '2% objecting' Davenport limiting criteria for offices should be used for residential developments

(b) 1.5% g (15 milli-g) should be used as an appropriate limiting criteria for luxury accommodation compliant with the NBCC specified range of 1% - 3% for a ten years return period wind event.

It is evident that these predictions are within the 80% of the '2% objecting' Davenport criteria and NBCC criteria of 1.5% g.

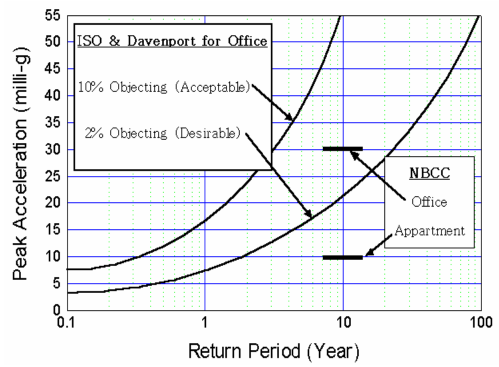


Fig. 10. Limiting Acceleration Criteria

5.3. Wind Tunnel Test

Wind tunnel studies have been carried out in the Research Institute of Hyundai Construction and Engineering Company. The accelerations are evaluated via force balance test on the basis of 2% damping, and summarized in the Table 6.

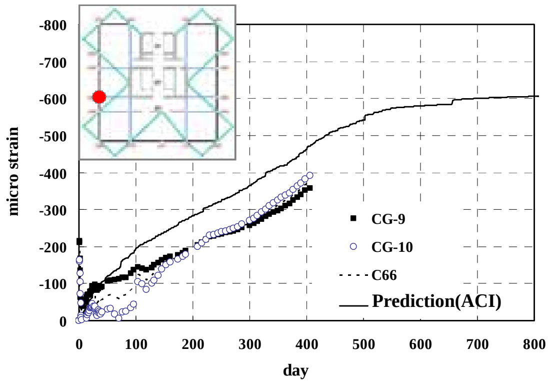
Wind tunnel test results show the comparatively similar values to the NBCC predicted accelerations, and satisfy the above limiting criteria.

Fig. 11. Wind Tunnel Test

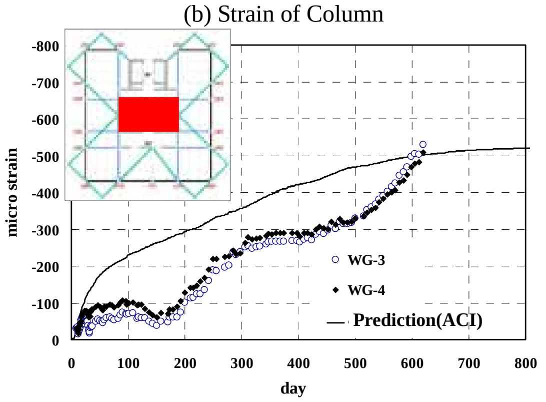
6. Axial Shortening Prediction

The differential shortening of column and core wall is due to different amount of axial shortening of columns with respect to the core wall under various contribution factors. The factors include:

(a) Shrinkage effect of concrete



(b) Strain of Column



(b) Strain of Core

Fig. 12. Axial strain (Prediction & Measurement)

(b) Elastic shortening of steel and concrete
(c) Long term creep effect of concrete

The axial shortening predictions of the tower were performed initially according to the general material data, properties of axial members, loading schedule and construction sequence.

The compensation for axial deformation may be based on the absolute shortening or the differential shortening. The target for compensation process is set to be the absolute shortening, and in this case the compensation values should be applied to the core walls as well as composite columns. The shortenings up to casting floor are compensated for automatically in the leveling process of horizontal floor system; thus, only the post-floor-deformation may need compensation.

The axially differential deformation between core wall and perimeter column will induce additional loading onto the outrigger members. The outrigger members may be overstressed and undergo plastic deformation, which ultimately cause instability problem to the overall structure. Therefore, the special considerations for reducing the effect of differential deformation in the outrigger floor should be required. Fig. 12 shows actual measurement value and prediction value of axial shortening at the column and the core. From the figure, it confirmed that necessary of consideration about differential shortening. The applied method is described in the next chapter.

7. Outrigger Construction

7.1. Location of Outrigger Floors

Table 7 shows the location of outrigger floors in each tower.

Table 7. Location of Outrigger and Belt Truss

Tower	Height above Ground (m)	Stories above Ground	Outrigger Floor
A	254	69	50th 32nd 9th
B	216	59	32nd 9th
C	200	54	32nd 9th

7.2. Connection between Outrigger and Shear Wall

The most complicated part of the outrigger construction is the outrigger to core wall connection. Three possible connection methods of outrigger are identified and the merits of each are summarized below:

(a) Embedded plate detail (Fig. 13(a))
The advantage of this simple detail is its ease of construction. The disadvantage is that in order to deal with the potentially large forces in the outrigger, it is expected that the load transfer into the concrete could become a little problematic.

(b) Encased top and bottom tie detail (Fig. 13(b))
Here, the top and bottom chords for the outrigger pass through the core in the form of vertical plates. The plates provide a good interface between the steel and concrete components. In this case, the large forces between the top and bottom chord are transformed via shear in the core walls.

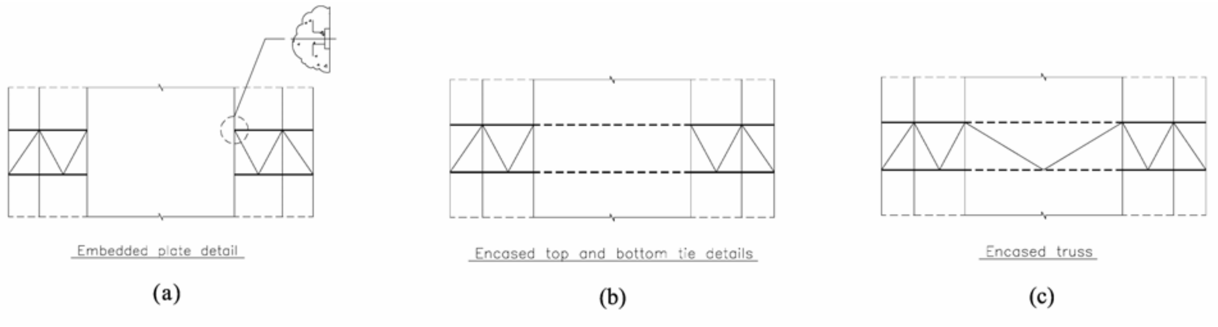


Fig. 13. Connection between Outrigger and Shear Wall

(c) Encased Truss (Fig. 13(c))
    Where the forces on the top and bottom boom are large, it may be necessary to supplement the strength of the core locally by adding truss elements to the top and bottom tie.

The method (b) is determined to be used in this project.

7.3. Differential Shortening of Core and Column

The core walls and the perimeter columns will undergo axial shortening due to elastic loading, shrinkage effect and long term creep effects. The sectional properties of the walls and columns are different and are each subjected to different loading. As a result the shortening experienced will be different on each of the elements. This differential shortening needs to be addressed in the design, and there are two main approaches:

(a) The outriggers are connected to the perimeter column late in construction to reduce the effect of differential shortening. Lateral stability should be maintained without outrigger function during the construction stage. The outriggers are designed to resist the differential movement which may arise after connection.
(b) The outrigger is connected to the perimeter column during the construction stage. The outrigger resists lateral forces with core wall, but the very large force would arise in outrigger and column due to differential shortening. It may be reasonable to introduce a means of adjusting the outrigger such that the effect of differential shortening can be minimized.

The approach to be adopted will depend on the construction sequence, the finalized design of the structural elements and the degree of predicted axial shortening. In the following, the adopted methods to each tower are described.

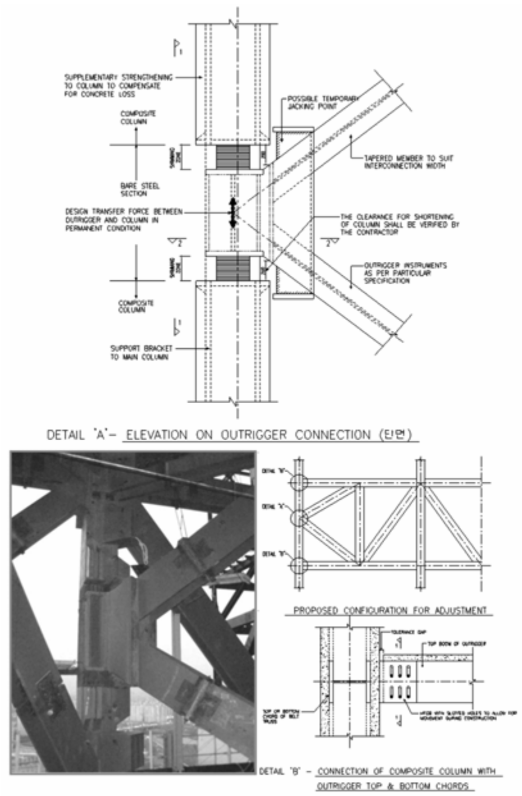


Fig. 14. Connection Details for Outrigger Adjustment

7.4. Outrigger Construction Sequence

In tower A, the installation of outrigger is immediately followed by its connection to the perimeter column. To adjust the outrigger movement periodically during the construction stage, special details like Fig. 14 are applied to the connection. The outrigger can contribute to the lateral stability during the construction stage without the effect of differential shortening by the adjustment. This connection will be locked after construction stage, since the differential shortening may be negligible in this time. The sequential adjustment steps (Fig. 15) are:

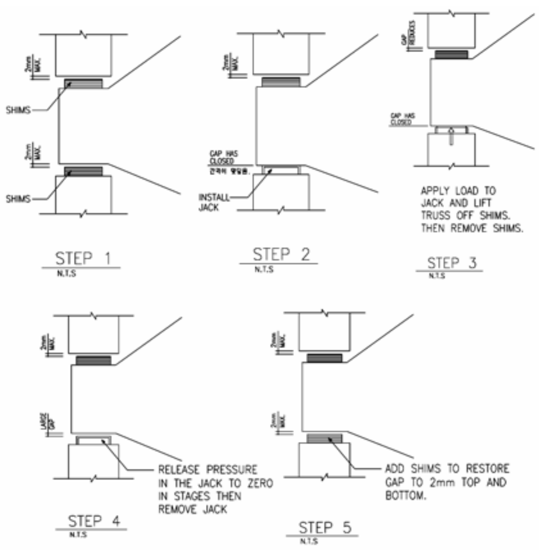


Fig. 15. Sequential Adjustment Steps

(a) Step 1
Gap on top and bottom should be not greater than 2mm on wind condition. This condition should exist after the outrigger at a particular level is required to be effective in resisting wind load during construction.

(b) Step 2
Continually monitor and install shims to maintain 2mm gap. If the gap has closed either at the top or bottom faces, check the load exerted on the outrigger truss. When the load reaches the maximum allowable, install jack and commence adjustment.

(c) Step 3
Apply load to jack, lift truss off shims, and then remove shims.

(d) Step 4
Release pressure in the jack to zero stage, and then remove jack.

(e) Step 5
Check that the 2mm gap is maintained. Add shims to restore gap to 2mm top and bottom.

The connection of outriggers to the column is carried out nearly in the end of construction stage in the tower B. After installation of the outrigger, delay joint is introduced in the connection. The term delay implied that the specified joints in outrigger structures are connected loosely over certain period of time during the construction stage. Thus the core wall and perimeter column can undergo a different extent of axial shortening without inducing any additional stresses to the outrigger members.

Since the core walls of the tower C have relatively smaller stiffness compared to the tower B, they cannot solely resist lateral forces for the full time of construction. Hence, the bottom outriggers of 9^{th} floor had connected to the perimeter column when the top outriggers of 32^{nd} floor were installed. The top outrigger connected nearly in the end of construction stage.

8. Conclusion

In this paper, Hyundai-Hyperion Project, which one of the tallest building in Korea, has been introduced. Key points in the structural design process and engineering characteristics were briefly reported. These include the effect of connecting the tower with podium, restraining effects of basement floor on tower, lateral stability system, wind and dynamic behavior, axial shortening prediction, and outrigger construction.

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References

1) ACI Committee 318.(1995) Building code requirements for structural concrete (ACI 318-95). American Concrete Institute (ACI)
2) AISC.(1989) Allowable stress design specification. American Institute of Steel Construction (AISC)
3) Architectural Institute of Korea (AIK). (1995) Code for design of reinforcement concrete structure. AIK
4) Bungale S. Taranath. (1997) Steel, concrete, & composite design of tall buildings. McGraw-Hill
5) Bryan Stafford Smith & Alex Coull. (1991) Tall building structures : analysis and design. John Wiley & Sons