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Structural Design of an Ultra High-rise Building Using Concrete Filled Tubular Column with 780 N/mm^2 Class High-strength Steel and Fc150 N/mm^2 High-strength Concrete

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Abstract

In recent years, the performance requested for which an ultra-high rise buildings is diversified. Large spans are designed in order to gain wide workspace. Column positions are shifted in middle stories to provide space different from neighboring floors. Moreover, in the bottom layers of the building, it is becoming more important to expand freedom to plan flexibility such as creating publically opened wide atria that gives attractive free space. Earthquake-proof criteria is also changing not only human life protection deign but also a design that allows functional continuity. In order to achieve thee needs, as one of technology, we have developed ultra-high strength concrete filled tubular (CFT) columns of the box section that combine ultra-high strength concrete with specified strength of 150 N/mm^2 and ultra-high strength steel material with tensile strength of 780 N/mm^2. In this paper, the outline of development of an ultra-high strength CFT column is reported. Also, the structural design of the ultra-high-rise building using the CFT columns is reported.

Keywords: Concrete filled tubular column, Ultra high strength steel and concrete, Ultra high-rise building

1. Introduction

In recent ultra high-rise buildings, there are many cases where large spans are required to gain spacial freedom on typical floors and wide atria to allow continuity with the external spaces on the lower floors. In order to design these spaces, it is necessary to provide high strength in the structural members that constitute the building structure, particularly the columns. It is possible to avoid excessively large volume columns by using appropriate combinations of high strength materials. Therefore we have developed ultra high strength concrete filled tubular (CFT) columns that combine ultra high strength concrete with specified strength Fc150 N/mm^2 and ultra high strength steel material with tensile strength of 780 N/mm^2 (Morita et al., 2011; Sato et al., 2009). In this paper we describe structural design that applies ultra high strength CFT columns to an ultra high-rise building. The strength of the concrete and the steel in this design is the world's highest class.

2. Outline of Building and Structure

This building is an ultra high-rise building with 38 stories above ground (building height 199.7 m), 6 stories below ground (building depth 35.1 m), and three stories penthouse, with a total floor area of about 198,000 m² (Fig. 1). Building uses are office, hotel, and stores, etc. One fea-

Figure 1. Building appearance.

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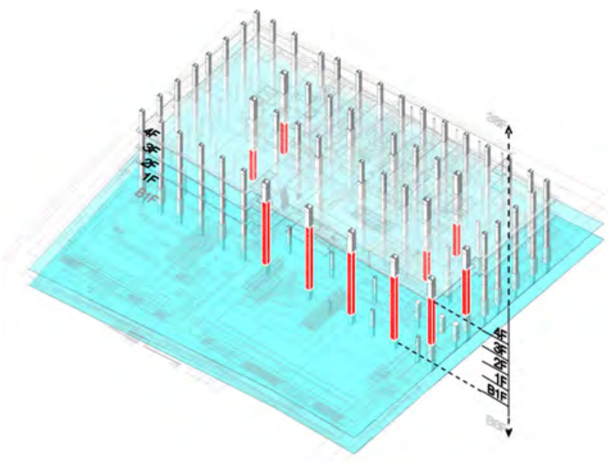

Figure 2. Location of ultra high strength CFT columns.

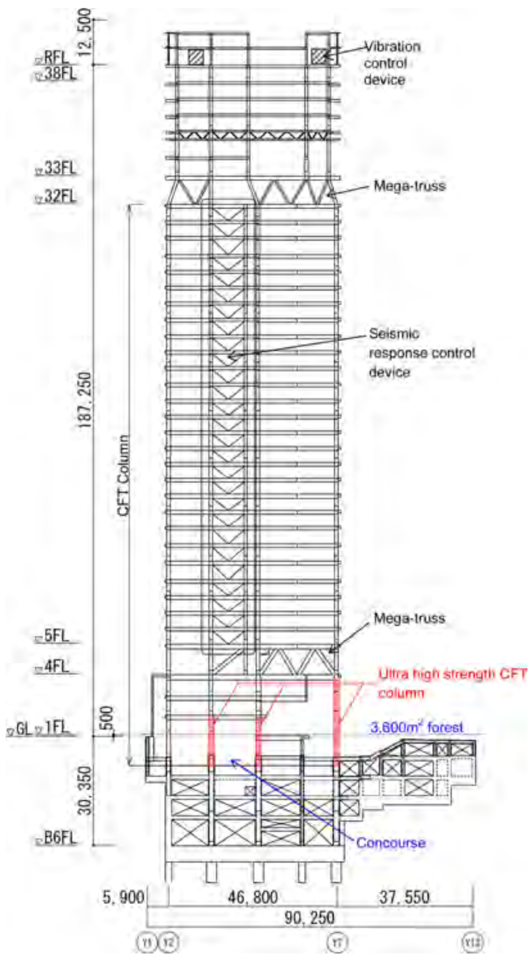

Figure 3. Framing elevation.

ture of the architectural planning is that a large space is provided between 3,600 m² forest developed on artificial ground at the first floor and the concourse of the Metro, so one task of the structural design was how to safely design the structural frame without causing major obstruction to this space.

Normally CFT columns are column members with excellent structural performance that utilize the characteristics of concrete, which is strong in compression, and steel, which are strong in bending (tension), so they are adopted in high-rise buildings. However, when conventional materials are used, the dimensions of the column cross-section are excessive on the lower stories when designing 200 to 300 m class ultra high-rise buildings having long spans. Also when a mega structure is adopted in which multiple members are combined together, the volume of the structure is too large, which excessively affects the core planning and the façade design, so there is a possibility that the intended building design cannot be finalized. Therefore, ultra high strength CFT columns have been developed with the aim of providing members that can satisfy the architectural planning and sufficiently satisfy seismic safety with a volume of column members similar to those of medium-rise buildings. These columns combine ultra high strength concrete with specified strength of Fc150 N/mm^2 and ultra high strength steel with a tensile strength of 780 N/mm^2, and they have been adopted on this building. Figure 2 and 3 show the parts where ultra high strength CFT columns have been adopted.

The structure of the building is an reinforced concrete structure below ground (steel and reinforced concrete structure in part), and an steel structure above ground (columns are CFT columns except the hotel floors). The structure form is a moment resisting frame with seismic shear walls below ground, and a moment resisting frame with seismic response control devices provided in the core above ground. Also, mega-trusses are used on 4F and 32F, which are structural changing floors where the positions of the columns are changed structurally. The framing of the mega-trusses enables different spans on the stories above and below. To ensure comfortability for the occupants during strong winds, vibration control devices (active mass dampers) are installed on the rooftop.

3. Seismic Design

3.1. Seismic design and analysis model

High seismic performance was required for this building, suitable for a building close to Tokyo Station. Therefore design criteria were set for the seismic performance that when subjected to earthquakes that would occur extremely rarely (Level 2), the members will remain within the elastic range, and the response story drift angle will not exceed 1/150. In addition, when subjected to extremely large earthquakes (Level 3), 1.2 times the Level 2 earthquakes, the building will remain safe (Table 1). As a result, the building will maintain very high seismic performance compared with an ordinary building. For this purpose, oil dampers as viscous dampers were provided in the core of the building, and buckling restrained braces using low yield steel (LY225 : yeild strength 255 N/mm^2)

Table 1. Design criteria

Seismic motion	Level 1	Level 2	Level 3
Official earthquake level	Rarely occurring earthquake	Extremely rarely occurring earthquake	Level 2 × 1.2
This project	Story drift angle ≤ 1/300 Stresses within the allowable stress	Story drift angle ≤ 1/150 Stresses within the elastic limit Plasticity ratio ≤ 1	Story drift angle ≤ 1/100 Member plasticity ratio ≤ 4 Story plasticity ratio ≤ 2
Ordinary ultra high-rise building	Story drift angle ≤ 1/200 Stresses within the allowable stress	Story drift angle ≤ 1/100 Member plasticity ratio ≤ 4 Story plasticity ratio ≤ 2	

Table 2. List of design input earthquakes

Seismic waves used		Level 1 Rarely occurring earthquake		Level 2 Extremely rarely occurring earthquake		Level 3 Level 2 × 1.2		Duration (sec)
		Maximum acceleration (cm/sec^2)	Maximum velocity (cm/sec)	Maximum acceleration (cm/sec^2)	Maximum velocity (cm/sec)	Maximum acceleration (cm/sec^2)	Maximum velocity (cm/sec)	
Official waves	Official 1 (Hachinohe)	72.5	10.5	362.4	52.3	543.6	78.5	120.00
	Official 2 (Kobe)	76.3	11.1	381.6	55.4	572.4	83.1	120.00
	Official 3 (uniform random)	66.8	11.7	333.9	58.7	500.9	88.1	120.00
Measured waves	El Centro NS (1940)	255.4	25.0	510.8	50.0	-	-	53.76
	Hachinohe NS (1968)	166.7	25.0	333.5	50.0	-	-	51.00
	Taft EW (1952)	248.3	25.0	496.6	50.0	-	-	54.40

were used in axial members as hysteretic dampers, each appropriately arranged so that seismic energy can be effectively absorbed.

The 45-node lumped mass model including above ground and below ground as shown in Fig. 5 was used to confirm the performance using nonlinear time history response analysis, using several design seismic motions as shown in Table 2 and Fig. 4. The nonlinear properties of the buckling restrained braces were as shown in Fig. 6, and bilinear hysteretic rule was used. The oil dampers were set as dampers with relief mechanism as shown in Fig. 7. As shown Fig. 8, the yield shear force of the buckling restrained braces in the transverse direction were set to about 20% of the Level 2 response shear force, and the relief load of the oil dampers was set to about 10%. Also,

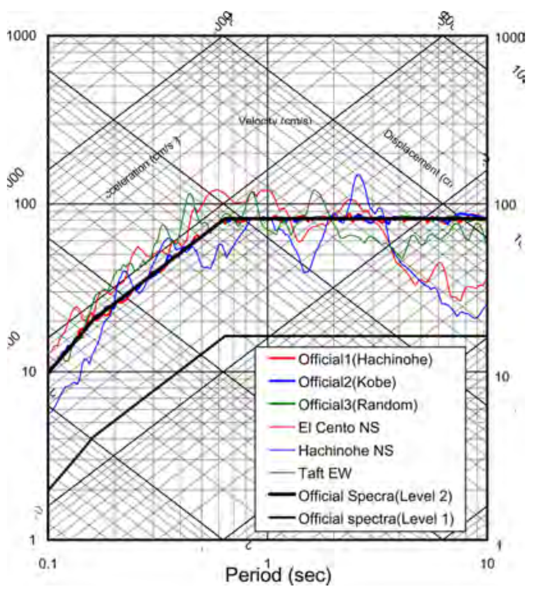

Figure 4. Pseudo velocity response spectra (Level 2 earthquakes).

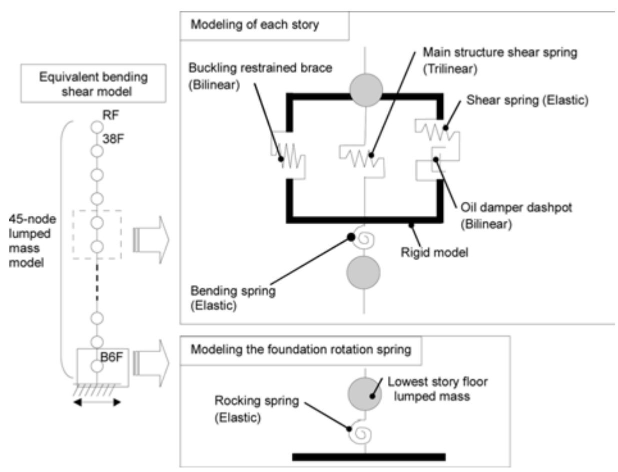

Figure 5. Time history response analysis model.

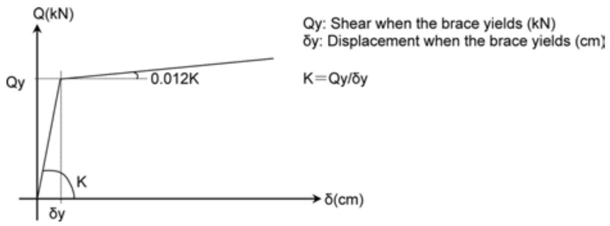

Figure 6. Restoring force property of buckling restrained brace.

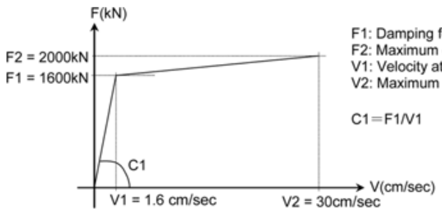

Figure 7. Restoring force property of oil damper.

the hysteretic restoring force characteristics of the main frame were set as normal trilinear. Damping was internal viscous type damping proportional to the instantaneous stiffness, and the damping factor was 2% of the primary natural frequency.

Also, nonlinear analysis with gradually increasing loads was carried out on a three-dimensional frame model, to confirm the safety of the members under each level of earthquakes and to confirm the failure mechanism. It was confirmed that with the design shear force coefficient (C_B = 0.065), the stress in each member was within the allowable stress, and when subjected to the shear force equivalent to Level 2 response (longitudinal direction C_B = 0.150, transverse direction C_B = 0.125), each member was within the elastic limit.

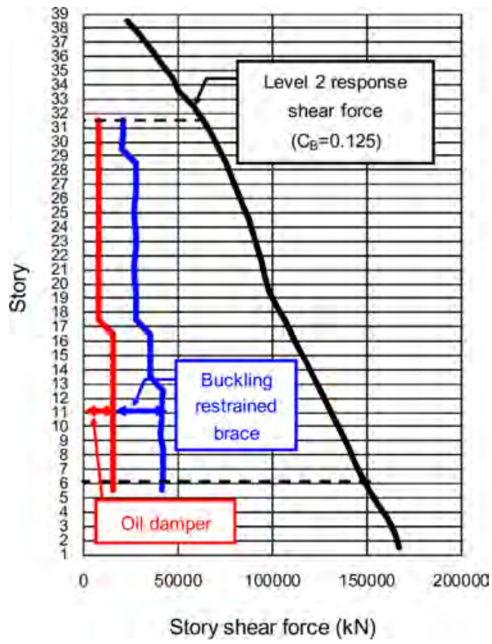

Figure 8. Ratio of shear strength of seismic response control system (in the transverse direction).

3.2. Time history response analysis

From the results of eigen-value analysis it was found that the primary natural period T = 4.378 seconds, and that the building is slightly stiff compared with buildings on a similar scale because the structure was required to satisfy high design criteria.

Figure 9 shows the results for the maximum response story drift angle in the transverse direction. In all cases, the design criteria are satisfied, so it can be seen that the seismic response control system used in this building reduces the amplification of the response of the stories above ground due to the earthquakes. Also, under the Level 2 earthquakes all members are within the elastic

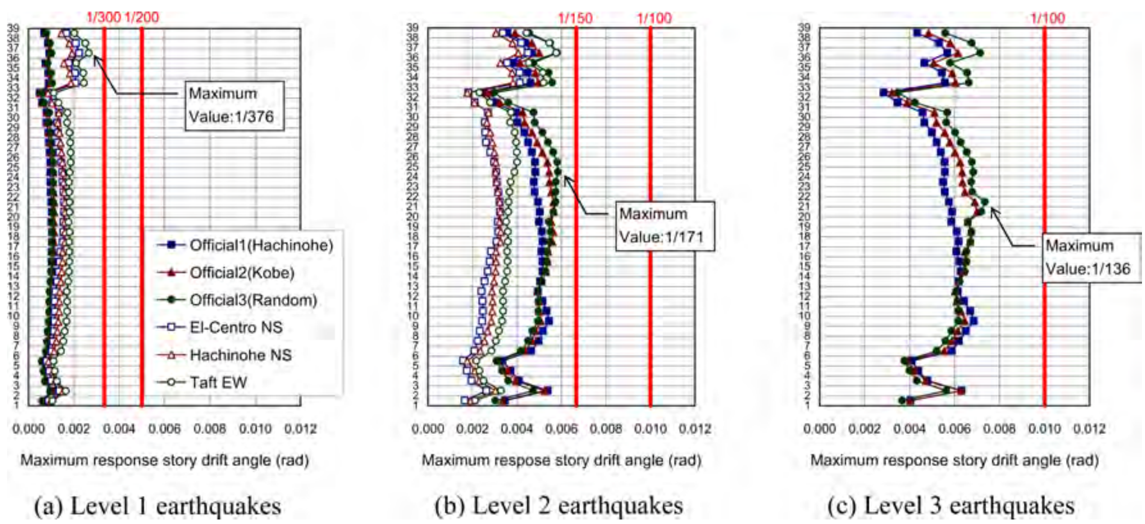

Figure 9. Time history response analysis results (in the transverse direction).

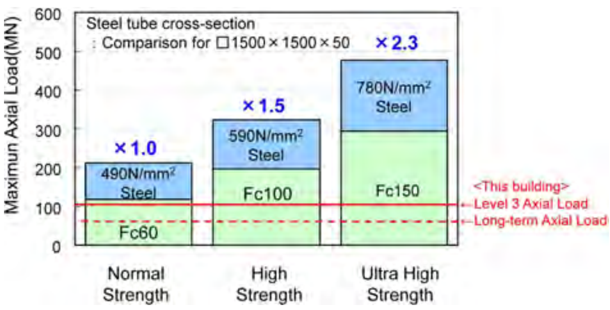

Figure 10. Comparison of axial load resistance of CFT columns.

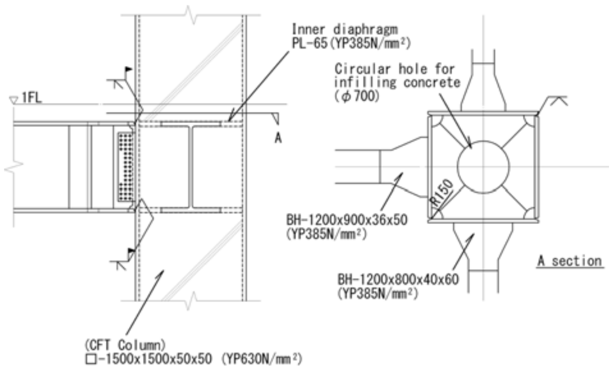

Figure 11. Details of ultra high strength CFT column.

Figure 12. Confirmation of failure mode.

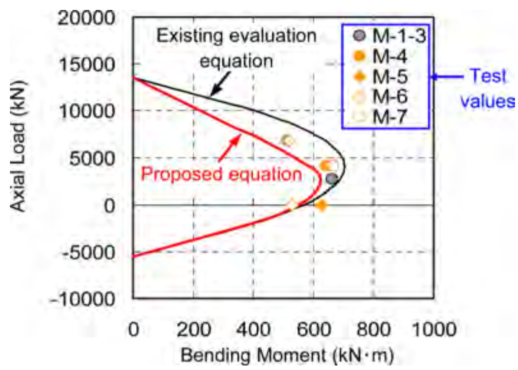

Figure 13. Proposed load resistance of ultra high strength CFT column.

range, and under the Level 3 earthquakes some beams become plastic, with maximum story plasticity ratio of 1.264.

4. Ultra High Strength CFT Columns

4.1. Use of ultra high strength CFT columns in the building

A total of ten ultra high strength CFT columns that combined 780 N/mm^2 steel and Fc150 N/mm^2 concrete were used in the building: six columns on the south face of the building spanning 14.4 m from the basement second story to the third story above ground, and four corner columns on 1F in the center of the building supporting 22-m floors above ground in both north and south directions. Figure 10 shows the comparison of the axial load resistance of CFT columns using standard materials (normal strength), high strength materials, and using the combination of the ultra high strength materials. The maximum axial load of the ultra high strength CFT column is 2.3 times that of the ordinary strength CFT column, so it has an extremely high axial load bearing capacity. As a result, columns that are arranged at 7.2 m spans in beam directions in the ultra high-rise building can be concentrated at 14.4 m, so it is possible to provide the large atrium space in the lower story portion of the ultra high-rise building. Also, as large cross-section CFT columns ?-1500×1500×50, the elastic deformation capacity is ensured by using reduced maximum axial load ratios compared with normal, such as maximum axial load ratio under long-term loads 0.13, and maximum axial load ratio during Level 3 earthquake 0.21. Figure 11 shows the details of the beam to column connection of the CFT column. The concrete pouring hole (ϕ700 mm) in the diaphragm has an opening area of about 20% of the column concrete cross-sectional area, for improved filling of concrete. Also, the haunches of the beams are to ensure that in a major earthquake the plastic zone of the beams go from the end of the beam towards the center of the beam.

4.2. Structural design of ultra high strength CFT column

Tests were carried out to confirm the structural performance in order to verify the validity of the load resistance evaluation equations for the strength of CFT columns using the combination of the ultra high strength materials. In this way, the failure mode was confirmed (Fig. 12), and after determining the method of evaluating the load resistance (Fig. 13), structural design was carried out (Sato et al., 2009).

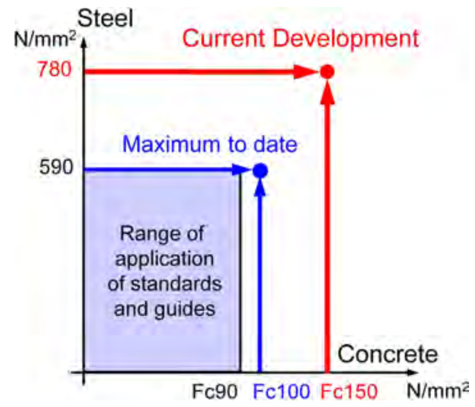

Figure 14. Combination of materials used in CFT columns.

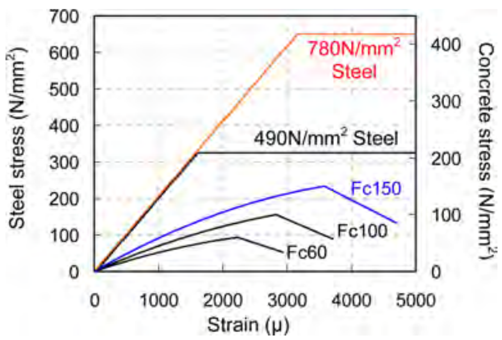

Figure 15. Stress strain relationship of concrete and steel.

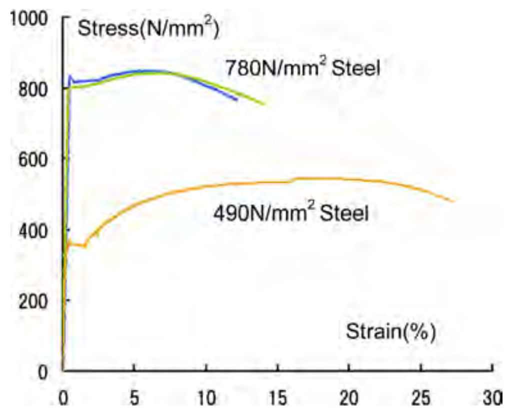

Figure 16. Steel stress-strain relationship.

As shown in Fig. 14, standard materials used in CFT columns are steel with strength 490 N/mm^2 and filling concrete with strength Fc60 N/mm^2. Even when high strength materials are applied to ultra high-rise buildings, the range of application to date has been steel of strength 590 N/mm^2 and filling concrete of strength Fc100 N/mm^2. The combination of ultra high strength steel 780 N/mm^2 and ultra high strength concrete Fc150 N/mm^2 used in this building is the world's highest class strength of CFT column. Figure 15 shows the stress-strain relationship for the

Figure 17. Concrete filling confirmation test.

steel and concrete. The limiting strain of the 780 N/mm^2 steel and the Fc150 N/mm^2 concrete are virtually the same, so this is a combination that can exhibit the advantages of high strength materials to the maximum extent. The stress-strain relationship of the 780 N/mm^2 ultra high strength steel is shown in Fig. 16. Although the strength is about 1.6 times greater than conventional steel, the ratio of the yeild strength to the tensile strength is high, and the strain at the tensile strength is small. Therefore in the design of this column, enough margin to remain within the elastic range is required.

4.3. Constructability of ultra high strength materials

Steel with strength 780 N/mm^2 has obtained the approval of the minister as construction material, it can be produced, and its performance has been verified in structural tests. Also, stable quality (constructability and strength) can be ensured for ultra high strength concrete Fc150 N/mm^2 as a result of the development of high performance super-plasticizer, and there is experience with application in reinforced concrete (RC) columns in the lower stories of ultra high-rise residential buildings. There is much construction experience in the use of this concrete in RC columns, but there have been very few studies done on its application to CFT columns. To actually apply these high strength materials to CFT columns, following issues regarding construction and production aspects had to be addressed.

Fc150 N/mm^2 class ultra high strength concrete contains much binder material in the ingredients, mainly cement, so when the concrete is fresh, viscosity is high. Therefore pump delivery technology that is capable of supplying concrete stably with high pressure is necessary in order to reliably pour the concrete into the steel tubes.

In order to fill steel tubes with Fc150 N/mm^2 class ultra high strength concrete, blending technology that minimizes shrinkage when the concrete has hardened is necessary.

In order to use high strength steel materials, it is necessary to select the welding materials and the various conditions (location of welding works, temperature, weld lo-

Figure 18. Factory weld construction test piece.

Figure 19. Site weld construction test piece.

cation, welding equipment, welding method) in accordance with special welding control methods.

In response to these issues, data concerning these high strength materials was accumulated by carrying out many material tests and construction tests, up to the level where construction can be carried out (Goto et al., 2011; Narihara et al., 2010).

Figure 17 shows a concrete filling confirmation test. Fc150 N/mm^2 concrete is raised to the top of the column under pressure from the pump vehicle. Various items were confirmed such as the stability when pumping the concrete, changes in the properties of the filling concrete, condition of the concrete after hardening, strength distribution, etc., and it was confirmed that construction can be reliably carried out.

Figure 18 is a full-sized test piece in a factory welding construction test. The corner welds and the diaphragm welds of the column were carried out by fabricating a box column at the factory. Also, Figure 19 shows a test piece for a site welding construction test that was carried out. The welding was carried out in the horizontal attitude at the actual site. The results of these tests confirmed the combination of steel material and weld material, the weld performance, and weld constructability. Figure 20 shows

Figure 20. Ultra high strength concrete-filled tubular (CFT) columns.

the photo under construction of CFT columns.

5. Conclusion

In this paper, the outline of the structural design of an ultra-high-rise building that applied ultra-high strength CFT column made of ultra-high strength steel material with tensile strength of 780 N/mm^2 and ultra-high strength concrete with specified strength Fc150 was described. The CFT column has high bearing capacity and elastic deformability. The adoption of the CFT column achieved high standard design criteria as well as slim framing and creating architectural spaces with high degrees of freedom.

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