

RESEARCH ON ADVANCED STRUCTURAL FRAMING SYSTEM FOR HIGH-RISE OFFICE BUILDINGS

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SUMMARY

We present an advanced structural framing system, which can construct cost-efficient high-rise office buildings with high additional value. Main components are comprised of, (1) earthquake-resisting core walls with boundary beams, which can bear almost all of the earthquake forces, (2) outer frames and (3) the inversed haunch beams of office areas released from earthquake force. These characteristics give flexibility to building planning and future possible renovations. The seismic response analysis results illustrated that the earthquake-resistance standards of Japan, as a severely seismic country, could be satisfied, and the boundary beams reduced the seismic response. The loading tests confirmed that the shear strength and bending characteristics of the earthquake-resisting walls with built-in steel could be evaluated by conventional design equations for reinforced concrete earthquake-resisting walls. It was also verified that the boundary beams proposed had a large equivalent damping factor and could decrease damage compared with boundary beams of normal cross-sections.

INTRODUCTION

Sustainability of current and future office buildings has become an important issue in terms of improving asset value and contributing to future society. Some methods of accomplishing sustainability are separation of the skeleton and infill, and integration of earthquake-resisting elements into building structures. Earthquake-resisting element integration, for example earthquake-resisting walls in the core and also tubular outer structure, have already been done. However, these methods alone are insufficient to ensure safety against large earthquakes in severe seismic countries like Japan. Furthermore, it is expensive to separate earthquake-resisting members and permanent load support members.

The authors propose an advanced structural framing system for constructing highly value-added high-rise office buildings at low cost. Main components of the proposed structural framing system are comprised of, earthquake-resisting core walls, which can bear almost all of the earthquake forces, and outer frames. The core walls are connected to each other by boundary beams working as dampers. These

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structural components release the floor framing of office areas from earthquake force; thus, it can be supported by simpler structural components, e.g. inversed haunch beams. These characteristics give flexibility to building planning and future possible renovations. Exactly, its system creates a tower building with multi-function by hybrid structure. (hereinafter called HMT for short) It is noted that HMT can be applied to hybrid buildings such as those where the upper stories are a hotel, etc.

In this paper, we report the composition of the HMT and, seismic response analysis results for a model building. Then, the loading test results of the earthquake resisting core walls with built-in steel and the boundary beams for the purpose of realizing the HMT are illustrated.

COMPOSITION AND MERIT OF HMT

The HMT's composition is shown in Figure 1. The outer frames are composed of the efficient structural combination, i.e. reinforced concrete columns and steel beams. The core wall system is composed of reinforced concrete earthquake-resisting walls and boundary beams that tie their walls together. In the earthquake-resisting wall, steels are built in as bending resistance reinforcement. The boundary beams are given the engineering cementitious composites with polyvinyl alcohol (hereinafter called PVA-ECC[1]), X-shaped steel bars, to achieve high energy dissipation and to minimize damage for the beam's function. The floor framing of the office areas between the outer frames and the core walls can be designed only for permanent load. Therefore, it can be supported by simpler structural components, e.g. inversed haunch beams, which the end of the steel beams were slenderized.



Figure 1. HMT's composition and subject building

Accordingly, HMT creates the merits of both planning and facility equipment as follows:

- (1) Three-dimensional free space: HMT realizes office space that ensures plane and vertical variability. Because changes can be made to meet future business requirements, the asset value of the building is improved. For example, it becomes feasible to maintain tenant-dedicated elevators and stairs, halls using the two-story space and free layout of well space (see Figure 2).
- (2) Free module: The efficient core planning can be created, since the core module is separated from the main module. Therefore the ratio of the office area to the total area can be augmented. For example, a compact planning can be efficiently made by setting the main module span to 7.2m and the core module span to 6.4m to match the elevator module.
- (3) Free mechanical space: The inversed haunch beams can create flexibility of ducting works. Therefore the equipment space of the air conditional systems is reduced by 25% compared to the concrete filled steel tube structure. For example, in the conventional structure, if a duct is threaded through a beam, it has to be divided due to the constraint of the opening area. If HMT is employed, the ducting can freely pass under the beam, thus enabling efficient duct laying while maintaining a larger area (see Figure 3).



Figure 2. Three-dimensional free space



KEY TECHNOLOGY OF HMT

The characteristics of individual key techniques for the HMT are described below.

Outer frames

The composition of the outer frames is shown in Figure 4. They comprised efficiently hybrid structure of reinforced concrete columns and steel beams. The reinforced concrete provided strength against vertical force applied to the column, and the steel provided strength against bending force applied to the beam. Because the outer frames bear a little of the earthquake force, they can be generally designed for permanent load. Accordingly, the beams can be used the same member in all the layers. Precast columns with the same bar arrangement could be used, in which the compression strength of concrete was increased for the lower stories. Steel plates were installed in the column at the beam flange levels. These panels and steel beams were jointed with high tension bolts. Therefore, this structure streamlined construction and saved cost.



Earthquake-resisting walls with built-in steel

Cross-sections of conventional reinforced concrete earthquake-resisting wall and earthquake-resisting wall with built-in steel are shown in Figure 5. Many large-diameter bending resistance reinforcing bars are arranged at each end of the wall, if there are designed by a conventional reinforced-concrete wall. Vertically connecting mechanical joints increase cost and construction time.



Figure 5. Conventional reinforced-concrete wall and earthquake-resisting wall with built-in steel

Furthermore, because a large number of the reinforcing bars is required, bending performance is impaired where these reinforcing bars to resist bending are located near the center of the cross-section. To solve these problems, these reinforcing bars were replaced with steel frames, bending resistance members were integrated and joints that used reinforcing bars were removed. Moreover, the bar arrangement for the wall was made with lap joints with small-diameter reinforcing bars under 19mm. Therefore, this improved the bending performance and streamlined construction.

Boundary beams

The composition of the boundary beam is shown in Figure 6. The energy dissipation capability was improved by converting second-step reinforcing bars to X-shaped steel bars. The PVA-ECC is mixed vinylon fibers (length 12mm and diameter 0.04mm) into the cement mortar. This PVA-ECC is capable of maintaining strength up to a tensile strain of 3%, and is expected to ensure fine cracks and decrease damage. PVA-ECC is a kind of high performance fiber reinforced cementitious composite (HPFRCC). This boundary beams have the function of a seismic response control device. Energy dissipation is realized from the yield of the reinforcing bars caused by deformation of the core walls during an earthquake.



SIMULATION ANALYSIS FOR MODEL BUILDING

Analysis condition

To clarify the feasibility of the HMT, its safety against earthquakes was verified using a simulation analysis according to the seismic design standard of Japan. The subject building was a 25-story office building about 100m heights, as shown in Figures 1. A representative cross-section of the core wall of the lowest story, a cross-section of the boundary beam of each story and a cross-section of the outer are shown in Figure 5 (b), Figures 7 (a) and (b), respectively. Floor framing that forms the office space was not incorporated into the analysis, because it bears only permanent load.



Input seismic motion was referred to the figure for the Building-Center-of-Japan standard artificial earthquake (BCJ-L2). This earthquake was larger than the Government standard artificial earthquake. The Government standard artificial earthquake is currently applied for this size of building in Japan in seismic design against large earthquakes. Figure 8 shows the acceleration response spectra of the BCJ-L2,

representative Government standard artificial earthquake (G-art), El centro (NS) and Taft (EW) with the maximum velocity standardized at 50 cm/sec.



Figure 8. Acceleration response spectra

Analysis model

The analysis model is shown in Figure 9. A plane analysis model was employed. Columns and beams of the outer frames, core walls and the boundary beams were modeled with linear members. A rigid floor was assumed and mass was concentrated at the floor location at each story.



A bending skeleton curve of the core wall was modeled tri-linearly from the bending moment(:M) -curvature (: Φ) relationship based on the Navier hypothesized section analysis [2]. The hysteresis of its bending skeleton curve was determined to be asymmetrical Muto's loop [3]. The compression and tension of the axial force during the earthquake were taken into account. The M- Φ relationship of the boundary beam tying the core walls was modeled tri-linearly with Sugano's equation [4]. The hysteresis of its bending skeleton curve was made as a Takeda's loop [5], and a rigid zone was incorporated into the space between the beam end and the center of gravity of the wall. Individual parts of the outer structure were modeled as follows. The M- Φ relationship of the column was modeled tri-linearly taking into account the axial force during the earthquake. The axial force was due to the shear force at the beam yield. Hysteresis was made to be a Takeda's loop. The M- Φ relationship of the steel beam was modeled bi-linearly by setting the bending yield as an inflection point. Its hysteresis was set to normal bi-linear. The panel took into account the shear stiffness by elasticity. The shear characteristics of the all members were set as elastic. The columns and core walls were fixed at the base of each member on the first floor where the earthquake motion was input. Internal viscous damping is assumed for the vibration model, in which damping factor is 3% for the instantaneous first natural period.

Analysis results

The natural period and analysis results of the Y direction are shown in Table 1 and Figure 10, respectively. The story drift angle shown in Figure 10 (b) was under 1/100, which was one of the design criteria. As shown in Figure 10 (c), the ratio of the shear force that the core wall bears was about 95% and 85% for the lower and middle stories, respectively. It is found that most of the seismic force was born by the core wall.



The stress of individual core wall (W2 in Figure 1) of the subject building and the rotation angle of the members (B2 in Figure 1) of the boundary beam are shown in Figure 11. The representative bending moment (:M) of the wall shown in Figure 11 (a) was lower than the yield bending moment (:My) of the wall. The shear stress (:S) in the wall shown in Figure 11 (b), for which the maximum value at the base of

wall was about 2 N/mm². It was lower than the shear crack stress (:Sc) and the ultimate shear strength (:Su). The rotation angle of the boundary beam shown in Figures 11 (c) was about 1/40. These results were specified as conditions of the loading test described in the next section.

An analysis model was constructed to review the effects of the boundary beam. An elastic boundary beam having an equivalent stiffness to the boundary beam's stiffness during an earthquake was incorporated into the model. Elastic characteristics were provided by setting the bending stiffness of the boundary beam to 1/50. Thus, the natural period during an earthquake was almost identical, and the boundary beam did not dissipate energy in the analysis model. The response of this analysis model (:Comp model) and the response of the HMT were compared at X direction, as shown in Figure 12. It was confirmed that the response largely decreased due to the vibration energy dissipation by the boundary beam.



LOADING TEST OF EARTHQUAKE RESISTING WALLS WITH BUILD-IN STEEL

Test purpose

The test was conducted to confirm that (1) the shear strength of the earthquake-resisting wall with built-in steel exceeded that of a reinforced concrete earthquake-resisting wall, and (2) the bending skeleton curve could be represented by the design equations of a reinforced concrete earthquake-resisting wall where the bending reinforcement was replaced with a steel frame. It is thus confirmed that the available study results for the reinforced concrete earthquake-resisting walls accumulated to present can be utilized in design.

Test specimen and test method

The configurations and sizes of the test specimens are shown in Figure 13. The test specimens were for the walls of lower stories of the model building shown in Figure 1 and Figure 7(a). The scale was about 1/5. Bearing plates were welded to the steel frame for connecting to the concrete. Vertical bar of wall and steel frames were anchored to stubs.

The test specimens were placed in the loading apparatus shown in Figure 14. Tests were carried out with static cyclic loading. Two kinds of tests were carried out: shear test (WS) and bending test (WB). To simulate the individual conditions, shearing destruction and bending destruction were induced by adjusting the ratio of bending moment / shear (:M/Q) in the testing apparatus. M/(QL) of the WS specimen was 0.67, and it of the WB specimen was 2.0. The axial force was set to have an axial load / axial strength ratio of 0.2. The material characteristics are shown in Table 2, 3. The concrete had higher strength than the design strength of 60 N/mm². It was 68.3 through 83.9 N/mm² for material ages of 41 through 110 days. Thus, the estimated concrete strength for the age at the tests was 79.9 N/mm² and 83.0

 N/mm^2 for WS and WB, respectively. The yield point of the steel frame was higher than the design value of 380 N/mm^2 .



Design strength	Age days	Compressive strength N/mm ²	Strain at compressive Strength $\times 10^{-6}$	Young's modulus ×10 ³ N/mm ²	Poisson's ratio
	41	68.3			
Fc60	81	79.9	3217	30.2	0.204
	110	83.9	3356	30.2	0.181

Table2. Mechanical characteristics of concrete

Table3. Mechanical characteristics of steel						
Kind	thickness	Yield point	Tensile strength	Strain at yield	Young's modulus	Part
	mm	N/mm ⁻	N/mm ⁻	×10 ⁻⁰	×10°N/mm²	
D6(SD345)		362	527	4137	1.700	Reinforcing bar
PL12(SM490A)	11.67	407	572	1979	2.058	Build-in steel

Test results

The load – displacement relationship for the shear test is shown in Figure 15(a). This figure also shows the calculated ultimate shear strength (:Qu) based on reference [6] in accordance with the method for reinforced concrete earthquake-resisting walls. The measured maximum strength from the tests was

higher than the calculated value. It was confirmed that the estimate of ultimate shear strength assuming the relevant wall as a reinforced concrete earthquake-resisting wall was usable for safe design.



The loading – displacement relationship for the bending test is shown in Figure 15(b). This figure also shows the calculated load – displacement relationship. The analysis model used in the calculation, and settings for the $M \cdot \Phi$ relationship and shear (:Q)-displacement (: δ) relationship, are shown in Figure 16. The same method was used here as for the characteristics settings for the analysis model shown in Figure 9. Rotation spring elements due to the extracted steel frames evaluated from the test results were incorporated into the top and base of the wall. Calculation results and test results agreed well. It was confirmed that the bending skeleton curve used for the HMT could be represented by the same evaluation method for reinforced-concrete earthquake-resisting walls.



Figure 16. Analysis model and settings for the bending and shear skeleton curve

LOADING TEST OF BOUNDARY BEAM

Test purpose

The purpose was to clarify the energy dissipation capability of X-shaped steel bars and the effect of reducing damage by using PVA-ECC in the boundary beams. Furthermore, variation of characteristics by introducing the pre-stress was confirmed. The tests confirmed the characteristics mainly up to the rotation angle of the member during a large earthquake: 0.03rad.

Test specimen and test method

The configurations and sizes of the test specimens are shown in Figure 17. Table 4 summarizes the test specimens. The test specimens were for the boundary beams of the each story of the model building. The scale was 1/2. Three kinds of test specimens were used: (a) normal concrete with a standard bar arrangement (NC), (b) PVA·ECC with X-shaped steel bars (FC-X), and (c) FC-X where pre-stress was introduced (FC-X-P). The reinforcing bars in each test specimen were adjusted to make the bending strengths almost identical. Reinforcing bars and pre-stress bar were anchored to stub.

The test specimens were placed in the loading apparatus shown in Figure 18. A static cyclic loading was carried out to accomplish inversed symmetry bending. The material characteristics are shown in Table 5, 6.



(a)Cross-section of NC (b)Cross-section of FC-X (c)Cross-section of FC-X-P Figure 17. Configurations and sizes of the test specimens

Table 4. Test specifien of bedridary beams					
Name of specimen		NC	FC-X	FC-X-P	
Concrete		normal	normal PVA-ECC		
Reinforcing bar	Horizontal bar	4-D19	2-D16	2-D13	
	X-shaped bar		2-D16	2-D13	
	Pt(%)	0.73	0.51	0.32	
Stirrup	Reinforcement	2-D10@75			
	Pw(%)	0.54			
Pre-stress	Diameter(mm)			32 Φ	
	Force(kN)			580	





Figure 18. Loading apparatus

Name of specimen	Compressive strength	Young's modulus	Strain at maximum strength	Tensile strength		
	N/mm ²	imes10 ⁴ N/mm ²	_	N/mm ²		
NC	36.1	2.92	1969	2.99		
FC-X	35.6	1.52	3850	4.73		
FC-X-P	38.7	1.83	3414	4.34		

Table5. Mechanical characteristics of concrete

Table6. Mechanical characteristics of steel

Kind	Yield point	Tensile	Strain at	Young's	Part
	_	strength	yield	modulus	
	N/mm ²	N/mm ²	×10 ⁻⁶	imes10 ⁵ N/mm ²	
D10(SD295)	360	495	1870	1.93	Stirrup
D13(SD390)	409	586	2290	1.78	Reinforcing bar
D16(SD390)	440	610	2350	1.87	Reinforcing bar
D19(SD390)	443	612	2260	1.96	Reinforcing bar

Test results

The load – displacement relationship is shown in Figure 19. The cracking conditions are shown in Figure 20. The strength of the NC started decreasing at a member rotation angle (:D/L) of about 1.5%. Slip characteristics were dominant in the hysteresis loop. However, the strength of the FC-X generally did not decrease up to the target rotation angle 0.03 of the member. The near spindle-shaped hysteresis loop was also observed. The hysteresis loop of the FC-X-P resembled the origin-oriented type due to the effects of the pre-stress. The cracking conditions of FC-X-P and FC-X were described as slight compared with those of the NC.



The equivalent damping factor (:heq) calculated from the test results is shown in Figure 21. The calculation method is shown in Figure 21. The equivalent damping factor of the FC-X was the largest

throughout almost all the member rotation angles. Accordingly, the boundary beam using X-shaped steel bars and PVA-ECC provided good energy dissipation performance and decreased damage.



CONCLUSIONS

The HMT proposed in this study is an advanced structural framing system that has many merits in planning and facilities. HMT is a hybrid structure using earthquake-resisting core walls, outer frames and inversed steel haunch beams. Also, it has seismic response control performance by combining core walls and boundary beams. As a result, HMT economically achieves high structural safety against large earthquakes. Furthermore, manufacturing efficiency is greatly increased by unifying the members of the outer frames and the boundary beams, the configuration and bar arrangement of the core walls, and the member of the floor framings that form the office areas. Utilizing the above-mentioned features, the structural construction cost is reduced by 15% compared to the concrete filled steel tube structure, and short-term construction is achievable. The seismic response analysis results illustrated that the earthquake-resistance standards of Japan, as a severely seismic country, could be satisfied, and the boundary beams reduced the seismic response. The loading tests confirmed that the shear strength and bending skeleton curves of the earthquake-resisting walls with built-in steel could be evaluated by conventional design equations for reinforced concrete earthquake-resisting walls. It was also verified that the boundary beams as dampers had a large equivalent damping factor and could decrease damage compared with boundary beams of normal cross-sections.

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