# ダブルチューブ合成構造の耐震性に関する解析的研究

ジュナス，ナスルディン
九州大学大学院人間環境学府空間システム専攻博士後期課程 : インドネシア・ハサヌディン大学空間システム専攻

河野，昭彦
九州大学大学院人間環境学研究院都市・建築学部門

松尾，真太朗
九州大学大学院人間環境学研究院都市・建築学部門

https://doi.org/10.15017/26753

出版情報：都市・建築学研究. 20, pp.119-127, 2011-07-15. 九州大学大学院人間環境学研究院都市・建築学部門

都市・建築学研究　九州大学大学院人間環境学研究院紀要　第20号，2011年７月
J. of Architecture and Urban Design, Kyushu University, No.20, pp.119 ~ 127, July. 2011

# ダブルチューブ合成構造の耐震性に関する解析的研究

# AN ANALYTICAL STUDY ON THE EARTHQUAKE-RESISTANT PROPERTY OF DOUBLE TUBE HYBRID SYSTEM

ジュナス　ナスルディン*，河野昭彦**，松尾真太朗**
Nasruddin JUNUS*, Akihiko KAWANO** and Shintaro MATSUO**

Investigation on Double Tube Hybrid System (DTHS) through an analytical study is conducted as a part of the proposal on the seismic design method for Double Tubes Hybrid System (DTHS) for buildings. This structural system comprises Energy Dissipation Structural Walls (EDSWs) as the interior tube and Spandrel Wall Frame (SWF) as the exterior tube. The hysteretic behavior of EDSWs and SWF have been experimentally investigated and reported elsewhere. They indicated a stable elasto-plastic manner under cyclic lateral loading, and had an ample energy dissipation capacity. In order to establish a reliable performance-based seismic design method for DTHS, a further investigation of overall building through an analytical study is needed. Three building models, 3-story, 6-story, and 12-story are simulated by a frame analysis method. The structural behaviors of DTHS are investigated by performing static and dynamic response analyses. It is proved that high-rise building model utilizing proposed structural system is an effective structural system for DTHS, in which overturning moment dominates rather than shear, which is desirable in the view point of structural design. Application in low-rise building is proved also as an effective method to increase the structural performance of DTHS even though the design strength is set slightly larger than the value for controlling the deformation in high-rise building.

***Keywords:*** *Double Tube Hybrid System (DTHS), Energy Dissipation Structural Wall System (EDSWS), Spandrel Wall Frame (SWF), Overturning moment, Frame analysis method*
ダブルチューブ合成構造，エネルギー吸収壁構造，スパンドレル壁骨組，転倒モーメント，骨組解析手法

## 1. INTRODUCTION

The DTHS comprises Energy Dissipation Structural Walls (EDSWs) as the interior tube and Spandrel Wall Frame (SWF) as exterior tube. The hysteretic behavior of EDSWs and SWF have been experimentally investigated and reported elsewhere. They indicated a stable elasto-plastic manner under cyclic lateral loading, and had an ample energy dissipation capacity. The seismic design method of DTHS should be investigated for practical design. According to that, analytical studies are done using many model buildings. One of the floor plan of those is shown in Fig. 1. In its basic concept, this structural system comprises RC core walls as the interior tube and the outer frames which consist of close-spaced columns tied at each floor level by deep spandrel walls to form DTHS. The interaction mechanism between the

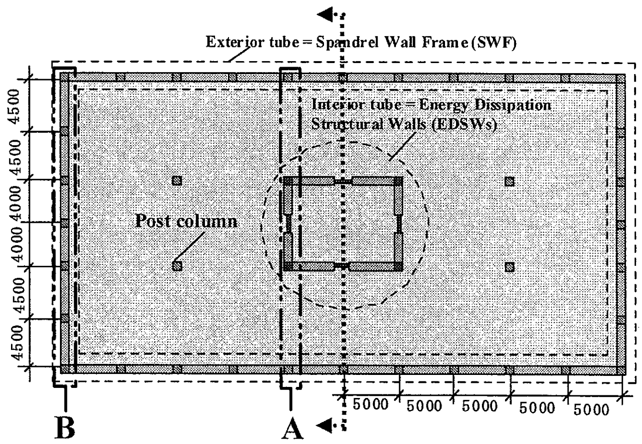



Fig. 1 The floor plan of the DTHS

* Doctor candidate Department of Architecture
空間システム専攻博士後期課程
Department of Architecture, Hasanuddin University, Indonesia
インドネシア　ハサヌディン大学　空間システム専攻
** Department of Architecture and Urban Design
都市・建築学部門

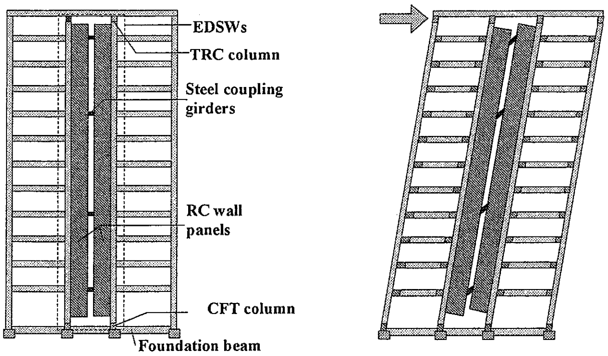


Fig. 2 Plan of structure A – EDSWs and its collapse mechanism

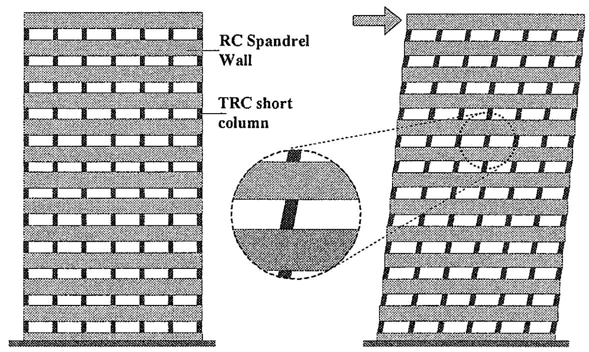
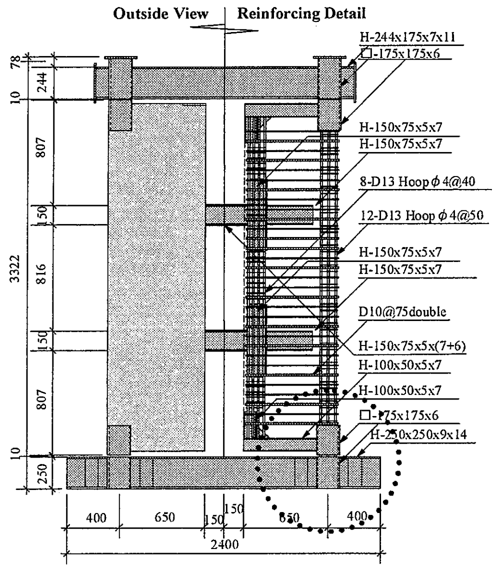
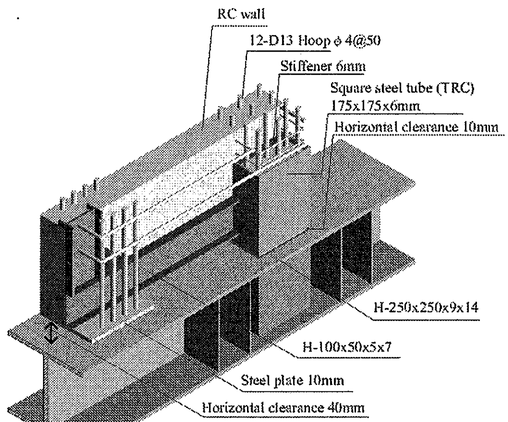


Fig. 3 Plan of structure B – SWF and its collapse mechanism

interior and exterior tubes improves an overall lateral force resistance of the building where the interior tube (RC core) which has enough story shear strength, well resists the overturning moment. The proposed DTHS comes up with a novel concept in which the overall deformations in the whole building are guaranteed by the interior tube made of the Energy Dissipation Structural Walls (EDSWs) as shown in the Fig. 2. EDSWs have a

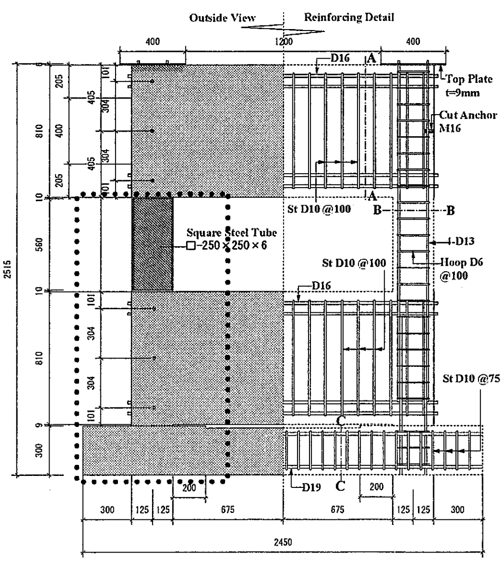


(a) EDSWs specimen

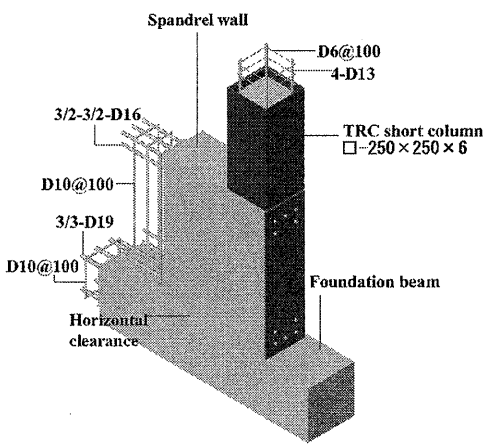


(b) Detail of the bottom part

Fig. 4 Component of the interior tube



(a) SWF specimen



(b) Detail of the important part

Fig. 5 Component of the exterior tube

role to resist the overturning moment and make a harmony with the collapse that occurs in perimeter frames which have a role to support the vertical loads. Therefore, both of these systems can interact well to control the damage and fail in the same mechanism of the sway as shown in Figs 2 and 3.

Fig.1 shows the floor plan of the proposed DTHS is assumed as an office building which composed of a slab system, post column, and two kinds of new constructional system in order to obtain a high seismic performance.

These new constructional systems are EDSWs as the interior tube portion and SWF as the exterior tube potion. The details of each part are shown as followings:

**Slab system**: The flat floor slab is adopted for the slab system in which no beams are installed. The floor slab is slightly thicker and more heavily reinforced in both directions than the ordinary floor system. There are extra reinforcing bars in the floor slab at the columns to transfer the loads properly.

**Post column**: This column has a prominent role to sustain the vertical load

**Interior tube** (Fig.4): The interior tube is EDSWs, which have been experimentally proved that the coupled shear walls behave in a very ductile manner under cyclic lateral loading, and had an ample energy dissipation capacity[1]. EDSWs are composed of coupled reinforced concrete (RC) walls linked by steel coupling girders. The RC walls are not anchored to the foundation beam directly, but supported by very short RC columns encased in square steel tubes (TRC column). The important characteristic of EDSWs is caused by an existence of horizontal clearance or slit between wall panels and foundation and roof beams as shown in Fig. 4(b). Furthermore, H-shaped steel are used as the coupling girders, which are designed to develop a coupling action to resist most of the overturning moment that induced in the building and to act as an energy-dissipating devices (passive damper) for the damage control design. Fig. 2 shows the desirable of plastic collapse mechanism of EDSWs, where the TRC column work as ductile plastic hinges. As aforementioned, this part guarantees overall deformation that occurs in whole building, especially to resist the overturning moment that is induced in the building due to severe earthquake.

**Exterior tube** (Fig.5): The SWF is composed of RC spandrel walls and TRC columns. The TRC columns are experimentally proved to have the extraordinary deformability. In the plastic collapse mechanism of the SWF, the TRCs yield at the both ends as shown in Fig. 3. The capacities concerning other collapse mechanisms are much larger than that of TRC column yielding mechanism. Therefore, the TRC short column is expected to yield in the early loading stage, so that the columns may behave as the hysteretic dampers. The elasto-plastic behavior of the SWF showed sufficient deformability and horizontal force carrying capacity as the exterior tube of DTHS buildings[2].

## 2. OUTLINE OF THE ANALYSIS

The objective of this analytical study is to predict the structural behavior of DTHS by performing static and dynamic response analyses.

### 2.1 Analytical model

The analytical models are 3, 6, 12 story DTHS buildings. Only a half of symmetrical plan (Fig. 1) was considered for object of investigation. Fig. 6 shows the analytical frame models of the DTHS which consists of a plane frame of EDSWs and a plane frame of SWF which is linked by rigid rods with end pin connections. The height of each story is 3.6m and distribution of mass are1.3ton/$m^2$, which is the average in a unit area of a floor considering all dead loads and live loads for each

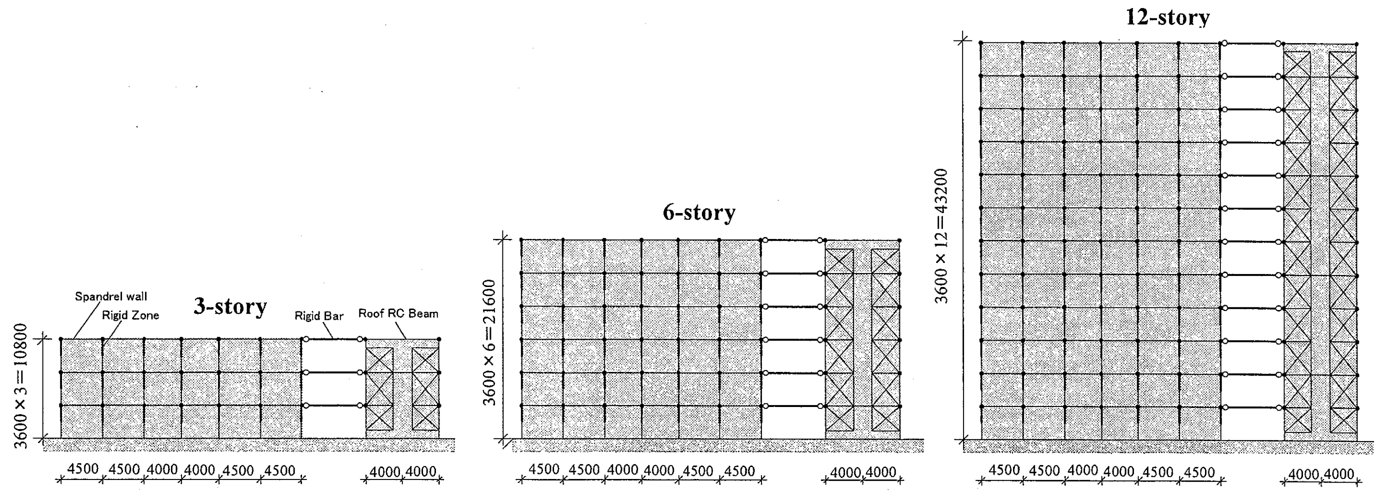



Fig. 6 Analytical Model

story. The detailed design of the interior tube and exterior tube concerned to analytical study are described as followings:

**Interior tube (EDSWs)**: The model for EDSWs is the similar model in reference 3). The top columns of EDSWs are TRC column and the bottom columns are CFT (concrete filled tubular) columns. The reason why the bottom columns are made by CFT is that the stress levels in those columns are very severe. The width to thickness ratio of the square steel tube for TRC and CFT columns are assumed to be 30 and the amount of the main reinforcement bars for the column was minimum requirement ($p_g$=0.8%) specified in the Japanese standard[3]. The wall panels are RC walls which width and thickness are 3000mm and 600mm respectively. The two RC walls are modeled by two elastic braced members which are assumed to remain in elastic. The steel coupling girders behave in elastic and plastic. One coupling girder is placed in the third floor in the 3 story building, two coupling girders are placed in the second and sixth floors in the 6 story building, and four coupling girders are placed in third floor and every 3 stories in12 story building. The details of sections of EDSWs are shown in Table 1.

**Exterior Tube (SWF)**: The spandrel walls are modeled as rigid bars because they have sufficient strength and rigidity than the TRC short column whereby only the TRC short column will yield. Therefore, the stresses

Table 1 Detail of Interior Tube (EDSWs)

| No. Stories | $F_c$ (N/mm$^2$) | RC Coupled Wall | | | | Steel Tube | | | Roof RC Beam | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall Thickness (mm) | Coupling Girder (mm) | $\sigma_y$ (N/mm$^2$) | $\sigma_u$ (N/mm$^2$) | Column Section (mm) | $\sigma_y$ (N/mm$^2$) | $\sigma_u$ (N/mm$^2$) | Section (mm) | Reinforcement Ratio ($p_t$) (%) | $\sigma_y$ (N/mm$^2$) |
| 3 | 24 | 600 | H-800×400×9.38×40 | 350 | 500 | 600×600 | 350 | 500 | 700×300 | 2.25 | 350 |
| 6 | | | | | | 750×750 | 350 | 500 | 850×450 | 1.42 | 350 |
| 12 | 36 | | H-800×400×18.2×40 | 350 | 500 | 900×900 | 350 | 500 | 900×550 | 1.56 | 350 |

$F_c$=compressive strength of concrete, $\sigma_y$=yield strength of steel, $\sigma_u$=ultimate strength of steel, $p_t$=reinforcement ratio of main steel bars

Table 2 Detail of Exterior Tube (SWF)

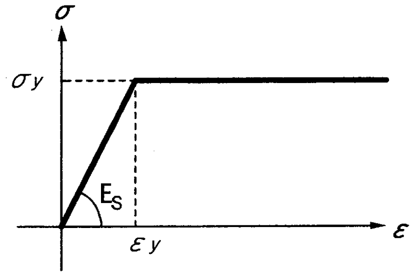
| No. Stories | TRC Column | | | | | | | | Spandrell Wall | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Available at | Position | $B$×$D$ (mm) | Reinforcement | Concentrated Reinforcement at the Section Center (mm2) | Fc (N/mm$^2$) | $\sigma_y$ (N/mm$^2$) | $\sigma_u$ (N/mm$^2$) | Available at | $B$×$D$ (mm) | Reinforcement | Fc (N/mm$^2$) | $\sigma_y$ (N/mm$^2$) |
| 3 | All stories | - | 500×500 | 2-D19 1-D22 | - | 24 | 350 | 500 | All stories | 500×1800 | 4-D38 | 24 | 350 |
| 6 | All stories | Corner | 800×800 | 1-D25 | 15000 | | 350 | 500 | All stories | 500×2000 | 6-D38 | | 350 |
| | | Inside | 500×500 | 2-D29 | - | | | | | | | | |
| 12 | 9-12stories | Corner | 800×800 | 5-D32 | 15000 | 60 | 350 | 500 | All stories | 500×2000 | 10-D38 | 36 | 350 |
| | | Inside | 500×500 | | - | | | | | | | | |
| | 5-8 stories | Corner | 800×800 | 6-D32 | 15000 | | | | | | | | |
| | | Inside | 600×600 | | - | | | | | | | | |
| | 1-4 stories | Corner | 800×800 | 6-D32 | 15000 | | | | | | | | |
| | | Inside | 650×650 | | - | | | | | | | | |

$B$=width of cross section, $D$=height of cross section, $F_c$=compressive strength of concrete, $\sigma_y$=yield strength of steel, $\sigma_u$=ultimate strength of steel

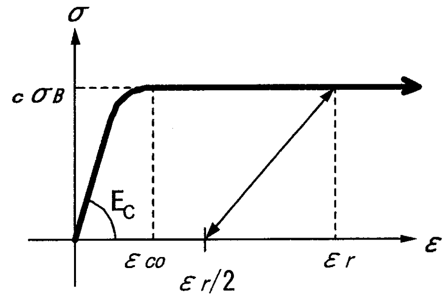
Table 3 Seismic Ground Motion for Dynamic Analysis

| Kind of Seismic Wave | Seismic Name | Mark in Graph | Originally | | | Analysis | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | PGA(m/sec$^2$) | PGV(m/sec) | Duration( s) | PGV(m/sec) | PGA(m/sec$^2$) | PGV(m/sec) | PGA(m/sec$^2$) |
| Natural Wave | El Centro NS | ● | 3.42 | 0.382 | 53.7 | 0.5 | 4.5 | 1.0 | 9.0 |
| | Hachinohe NS | ■ | 2.25 | 0.407 | 36.0 | 0.5 | 2.8 | 1.0 | 5.5 |
| | Tohoku NS | ○ | 2.58 | 0.373 | 41.0 | 0.5 | 3.5 | 1.0 | 6.9 |
| | JMA Kobe NS | ▽ | 8.21 | 0.926 | 50.0 | 0.5 | 4.4 | 1.0 | 8.9 |
| | Taft N021E | ◆ | 1.53 | 0.183 | 54.4 | 0.5 | 4.2 | 1.0 | 8.4 |
| Artificial Wave | Yokohama | ▲ | 3.13 | 0.562 | 40.0 | 0.5 | 2.8 | 1.0 | 5.6 |
| | BCJ-L2 | × | 3.56 | 0.807 | 120.0 | 0.5 | 2.2 | 1.0 | 4.4 |
| | JSCA Hachinohe(EW)* | ▼ | 4.38 | 0.521 | 60.0 | 0.5 | 4.2 | 1.0 | 8.4 |
| | JSCA Tohoku (NS)* | ◣ | 3.5 | 0.568 | 60.0 | 0.5 | 3.1 | 1.0 | 6.2 |
| | JSCA Kobe (NS)* | ◢ | 4.7 | 0.587 | 60.0 | 0.5 | 4.0 | 1.0 | 8.0 |

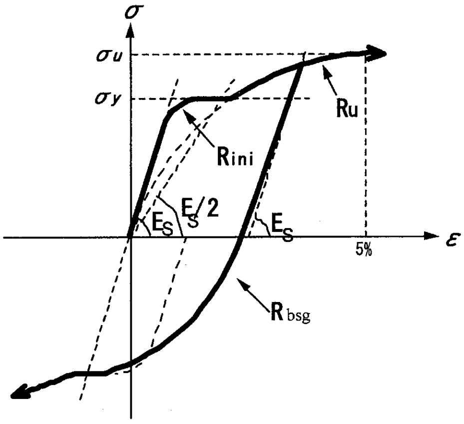
PGV: Peak Ground Velocity, PGA: Peak Ground Accelaration
*Reference 10




(a) Stress-strain relation model for steel bars




(b) Stress-strain relation model for concrete (Popovics's model)



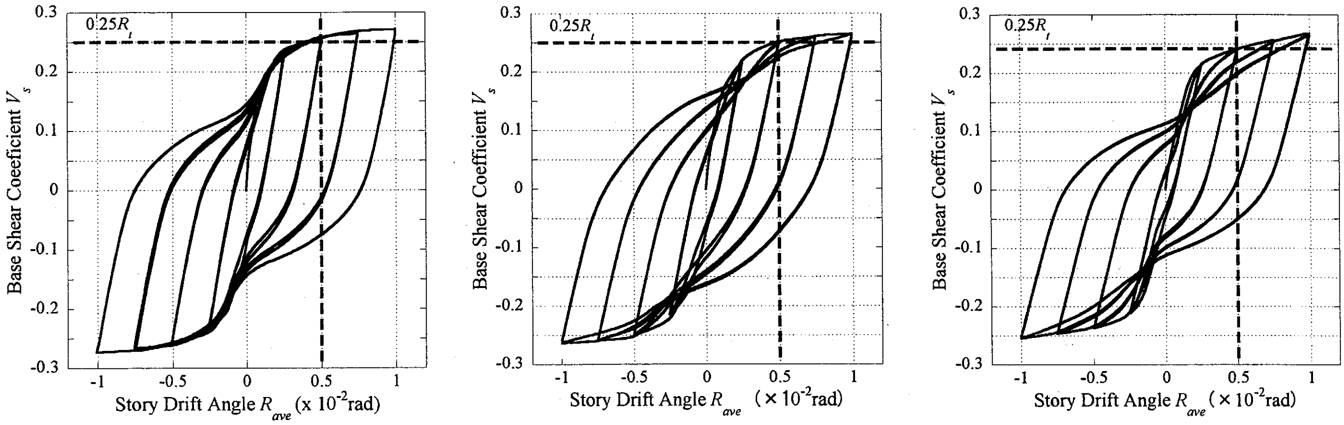

(c) Ohi and Akiyama's model for steel tube and H-shaped beam

Fig. 7 The constitutive of material

inducing the walls are small. The cross sections of columns are modeled as un-deteriorated RC members in order to simulate the TRC column. The cross sections of columns are constant in all stories in the 3 and 6 story model buildings. As for the 12 story building, the cross sections of columns changed every four stories in the vertical direction. Moreover, the large tensile force was likely to be induced of 6 and 12 story model buildings in the corner columns. The concentrated reinforcements in elastic are located in the center of a column cross section in order to avoid the tensile plastic elongation. Therefore, it is necessary to check whether the tensile stress in concentrated reinforcements exceed the tensile yield. Similarly, the large compressive forces are induced in corner columns, therefore, the dimensions of the cross sections of corner columns are designed to be larger than the others. Moreover, the wall is rigidly connected to the column, and a rigid zone is provided in the column portion in which length is the height of the spandrel wall. Details of the exterior tube portion are shown in Table 2.

### 2.2 Analysis method

The analytical models are simulated by a two-dimensional frame analysis method that shown in reference 5). The method has an ability to perform structural analysis of plane moment-resisting frames consisting of beam-column elements by taking the account of the geometric nonlinearity and material nonlinearity. The geometric nonlinearity is introduced by adopting the moving coordinate system for each beam-column element. In the moving coordinate system, it is assumed that the axial deformation and the flexural deformation are expressed by a linear and polynomials displacement functions, respectively. The element coordinate system moves within the global




(a) 3-story model building (b) 6-story model building (c) 12-story model building

Fig. 8 Load-Story Drift Angle Relationship coordinate system as a frame deforms. The element stiffness is calculated by the Gaussian numerical integration using three cross sections (integration point). The cross sections of a beam-column element are numerically integrated by dividing the section into a number of layers, referred to as stress fibers. In static analysis, the distribution of horizontal loads in height direction was based on *Ai* distribution specified in the order for enforcements of the Act on the Building Standard Law of Japan[6]. The horizontal loads are proportionally increased at all floor levels. In a dynamic analysis, the Newmark $\beta$ method was used, where $\beta$ is 0.25. The damping factors of 3 % for the first and second vibration modes according to the Rayleigh damping method. A couple seismic motions are used for dynamic analysis as shown in Table 3. The peak ground velocity (PGV) is corresponding to 0.5m/s or 1.0m/s. Detail of PGV and PGA are shown in Table 3.

The stress-strain of steel reinforcement is shown in Fig. 7 (a). The stress-strain model for concrete is shown in Fig.7 (b). The Popovic's function is used until the peak stress, and the stress is kept constant after the peak[7]. The unloading curve returns to the half of the experienced maximum strain. Fig. 7 (c) indicates steel tubes and steel shapes which are Oi and Akiyama's model as shown in reference 8).

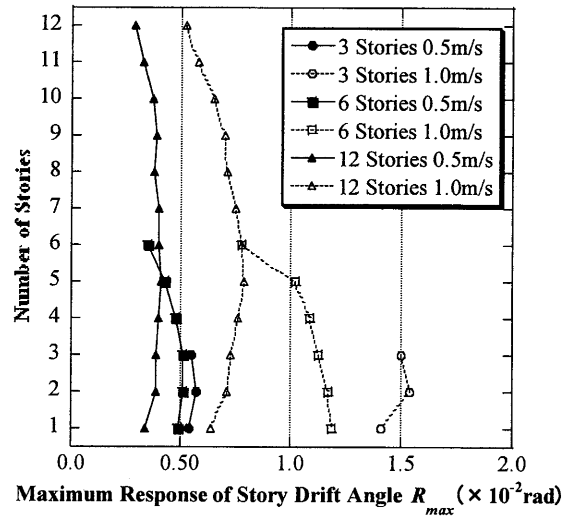


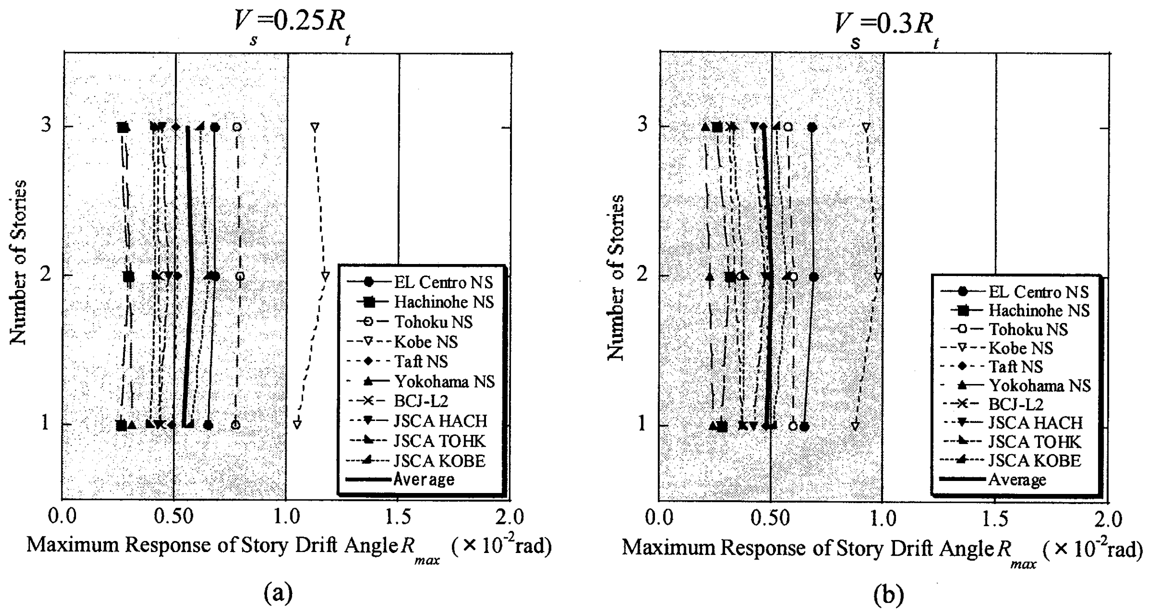

Fig. 9 The maximum response of story drift angle (Average comparison)

The evaluation of seismic performance is shown in the followings: The ultimate strength design is adopted. The coefficient of the design seismic loads is provided as $0.25R_t$, which $R_t$ indicates the spectral acceleration factor, which concept is the spectral acceleration nondimensionalized by the peak[6]. The variables of analysis are the total number of building's stories (3-story, 6-story, and 12-story), the kind of seismic waves (5

Table 4 Detail of Exterior Tube (SWF) ($V_s$=0.3$R_t$)

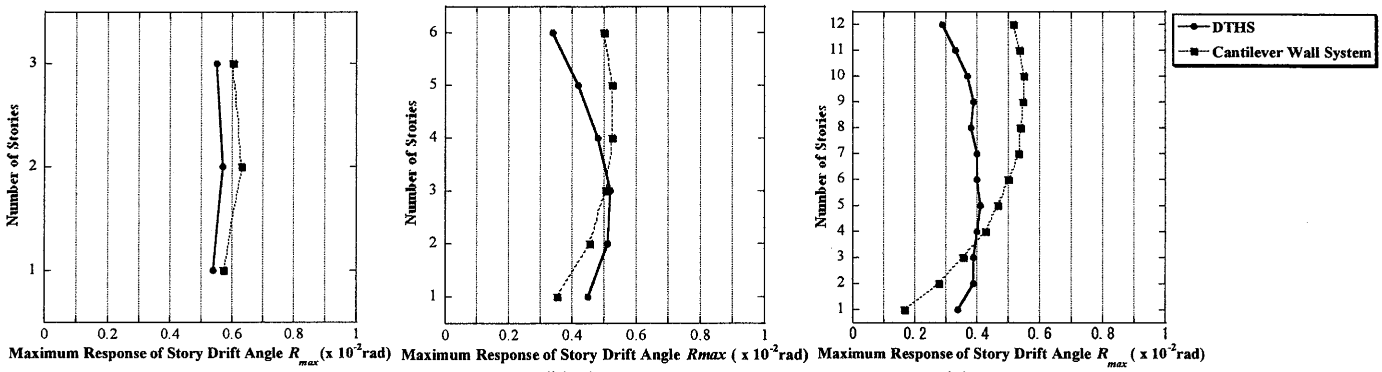
| No. Stories | TRC Column | | | | | | | | Spandrel Wall | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Available at | Position | $B$×$D$ (mm) | Reinforcement | Concentrated Reinforcement at the Section Center (mm2) | $Fc$ (N/mm$^2$) | $\sigma_y$ (N/mm$^2$) | $\sigma_u$ (N/mm$^2$) | Available at | $B$×$D$ (mm) | Reinforcement | $Fc$ (N/mm$^2$) | $\sigma_y$ (N/mm$^2$) |
| 3 | All stories | - | 500×500 | 4-D22 | - | 24 | 350 | 500 | All stories | 500×1800 | 4-D38 | 24 | 350 |

$B$=width of cross section, $D$=height of cross section, $F_c$=compressive strength of concrete, $\sigma_y$=yield strength of steel, $\sigma_u$=ultimate strength of steel




(a) (b)

Fig. 10 The average value of maximum story drifts response (3-story)




(a) 3-story (b) 6-story (c) 12-story

Fig. 11 Maximum response of story drifts (structural comparison to 0.5m/s)

natural seismic waves and 5 artificial seismic waves), and intensities of seismic waves/PGV (0.5m/s and 1.0m/s).

## 3. ANALYSIS RESULTS AND DISCUSSION

### 3.1 Horizontal load-story drift angle relationship

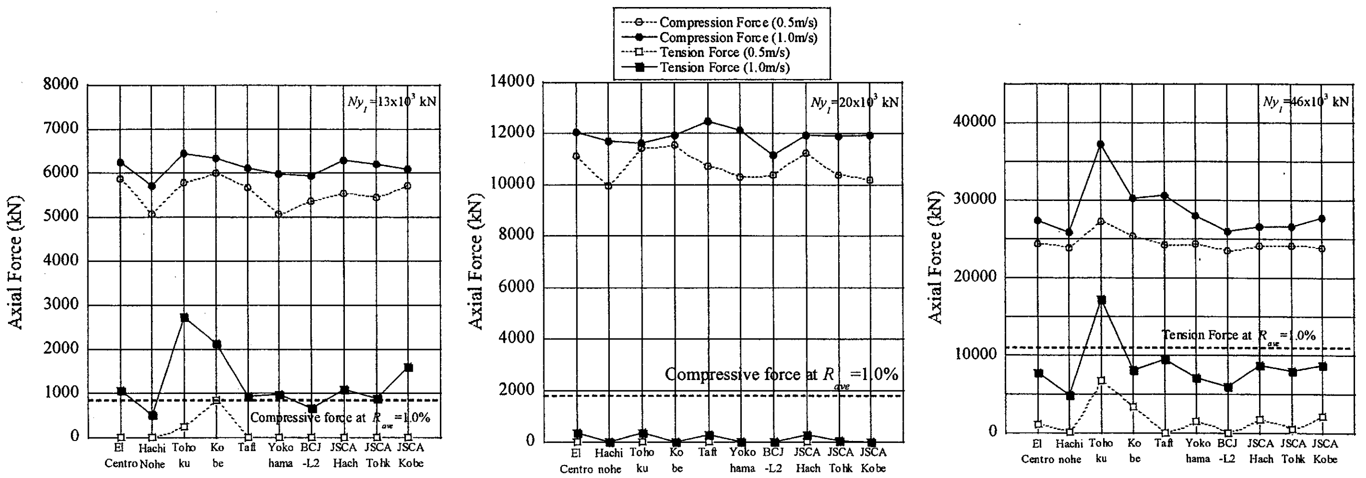
Fig. 8 shows horizontal load-story drift angle relationship for each model. The vertical axis indicates base shear coefficient ($V_s$) to represent the horizontal loads. The horizontal axis indicates the average of story drift angles ($R_{ave}$) which is obtained by dividing the top horizontal displacement by the height of building. As shown in the Fig. 8, when $V_s$ equal to $0.25R_t$, the $R_{ave}$ is equal to around 0.005rad which is the design criterion for seismic loads for the building. Due to characteristic of the EDSWs, the coupling girders yield in the very small story drift angle in the entire building. The hysteresis curve is the type of a spindle shape, so that the high energy absorption capacity can be expected. Moreover, the EDSWs have capabilities to equalize the story drift angles in all stories and to prevent the SWF from a story collapse mechanism, even though the SWF is designed so as the TRC columns yield early.

### 3.2 The maximum response of story drift

Fig. 9 shows the maximum story drift angle $R_{max}$ of each model which is obtained by performing a dynamic analysis. $R_{max}$ is the average of responses by ten seismic waves. The solid line and the dash line indicate the responses of two intensity levels of 0.5m/s and 1.0m/s respectively. If we pay attention to the number of stories, it seems that responses tend to grow in low-rise models. The high-rise model, in other hand, tends to be smaller responses. Similar results were obtained in the previous research [9)10)]. Therefore, in the seismic design, it is preferable to set the coefficient of base shear slightly larger for low-rise buildings than high-rise building in order to control the deformation.

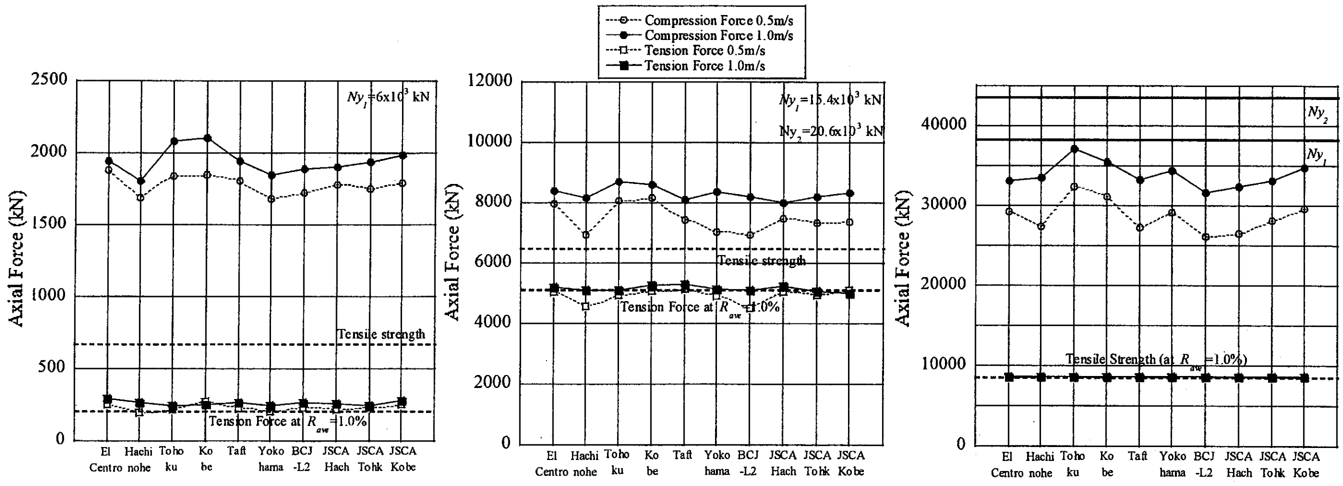
The $V_s$ of $0.25R_t$ is adopted as the ultimate strength for usual wall-frame system structures as prescribed in the design guideline [11)]. However, as shown in Fig. 10 (a), the responses of 3 story building under seismic motion of PGV=0.5m/s exceed the $R_{max}$ of 0.01rad, which is the design criterion for the DTHS.

It is confirmed that the 6 and 12 story buildings do not exceed the $R_{max}$=0.01rad. Fig. 10 (b) shows the 3 story DTHS model which is designed being based on $V_s$ of




(a) 3-story (b) 6-story (c) 12-story

Fig. 12 The maximum axial force in the CFT column of EDSWs




(a) 3-story (b) 6-story (c) 12-story

Fig. 13 The maximum axial force in the corner column of SWF

$0.3R_t$. The model is strengthened by increase of reinforcements of the TRC column as shown in the Table 4. If we assumed the $R_{max}$=0.01rad is the limit of the deformation in the maximum of seismic ground velocity of 0.5m/s, $V_s$=0.25Rt is not indicate as a preferable value to control the deformation. Pay attention on the responses value of $V_s$=03Rt, all of deformation occurred within $R_{max}$=0.01rad. Therefore, the responses of the model based on $V_s$=0.3Rt satisfy the design criterion of $R_{max}$=0.01rad.

### 3.3 Comparison of The DTHS to Cantilever Wall System

Fig. 11 shows the average values of $R_{max}$ by seismic motion PGV of 0.5m/s for the DTHS and cantilever wall system. The cantilever wall system is ordinary structural system which is the weak beam type frames installed by cantilevered RC walls behaving in a manner of flexural yielding. The detail of analytical study is shown in the reference 9). All these models were designed based on the responses of $V_s$=0.25$R_t$. Comparing to two types of structures, the responses of DTHS are suppressed and equalized the story drift angle in the vertical direction than the cantilever wall system. In the lower stories, the responses of DTHS are larger than that of cantilever wall system. This is caused that the sizes of the column system. This is caused that the sizes of the column section in the lower stories are relatively small compared to the cantilever wall system, due to the columns sections of DTHS are same in vertical direction.

### 3.4 The maximum of axial force in the CFT columns of EDSWs

Fig. 12 (a) to (c) shows the response of axial force in the CFT column of EDSWs by dynamic analysis, whereas the compressive force and tension force at $R_{ave}$=1.0% are obtained by static analysis. In the figure, $Ny_1$ is described as followings;

$$Ny_1 = A_C F_C \tag{1}$$

in which $A_C$ denote the cross sectional area of concrete, $F_C$ is concrete strength. $Ny_1$ is the yield strength of axial compression which is considering only concrete.

It can be observed that the responses is not indicated a remarkably large with respect to $Ny_1$ in all models. This is caused by the characteristic of EDSWs, which is the axial force in the edge column can be controlled by adjusting the shear strength in steel coupling girders. Hence, the axial force that occurs in CFT column does not become a problem in design. The difference of dynamic to static response is caused by dynamic magnification effect. Especially, the axial force response is very sensitive with regard to bending moment or shear force.

### 3.5 The maximum of axial force in the corner columns at the first story of SWF

Fig 13 (a) to (c) indicate the maximum response of axial force occurs in corner column of SWF in the first story. As shown in Fig. 13 (a) to (c), the 3 and 6-stories model unreached the tensile strength. As for 6-story model, the concentrated reinforcement in the center of section yield. The maximum responses of the compression force are small values. Therefore, it can be concluded that the fluctuation of axial force in 3-stories and 6-stories model does not become a big problem in the design. Regarding to 12-stories model, all cases reach the yield strength in tension and also shows an extremely large value in the compression. In the figure, $Ny_1$ is shown in Eq. (1), and $Ny_2$ is described as followings;

$$Ny_2 = A_C F_C + A_S \sigma_y \tag{2}$$

in which $A_C$ and $A_S$ denote the cross sectional area of concrete and cross sectional area of concentrated reinforcements respectively. Whereas, $F_C$ and $\sigma_y$ are concrete strength and yield strength of the concentrated steel bars respectively. $Ny_2$ is the yield of axial strength in axial compression, considering concrete and concentrated reinforcement. As shown in the figure, in the seismic ground velocity of 1.0m/s, a number of seismic waves are seen close to $Ny_1$. In the case of a large compression force occurs on the corner column, there are uneasiness concern to the performance and the stable deformation. As a result, when the overturning moment due to shear force in the high-rise building becomes an important issue, it must be paid attention on the excessive of axial force in corner column of SWF.

## 4. CONCLUSIONS

The analytical studies for seismic design method of Double Tube Hybrid System (DTHS) buildings are done to 3, 6, 12-story models by performing a static and dynamic response analyses. The following conclusive remarks are obtained:

1. It is preferable to set the coefficient of base shear of low-rise buildings slightly larger than high-rise buildings in order to control the deformation. $V_s$=0.25Rt is not indicate as a preferable value to control the deformation for low-rise building. In other hand, in the response of $V_s$=03Rt, all story drift angles remain within 0.01rad. Thus, in order to satisfy the design criterion, value of $V_s$=0.3Rt is required in seismic design for low-rise building.
2. The high-rise building such as 12-story model may have a problem regarding to the excessive of axial force in corner columns of Spandrel Wall Frame (SWF), which should be paid attention in practical design.
3. It is proved that high-rise building model utilizing proposed structural system DTHS is an effective structural system which is dominated by overturning moment rather than shear, which is desirable in the view point of structural design. Moreover, this system also proved an effective system for low-rise building.

## ACKNOWLEDGEMENTS

Authors greatly acknowledge to Professor Emeritus Kenji Sakino of Kyushu University for his valuable advice so that this study can be carried out, and a contribution of former graduate student Yasukochi J. of Kyushu University to the analytical study reported in this paper is also acknowledged.

## REFERENCES:

1) Nasruddin J, Nakahara H., and Kawano A.; Design Method for Preventing Brittle Failures of Energy Dissipation Structural Walls, Journal of Habitat Engineering Vol.3 No.1, March 2011, page 1-10
2) Nasruddin J., et.al.; Experimental Study on RC Frame Composed of Tubed Reinforced Concrete Short Columns and Spandrel Beams (Part 1 and Part 2), Annual Architectural Research Meeting of AIJ Kyushu Chapter, March, 2011
3) Sakino K., Nakahara H., Experimental Study on Composite Structural Walls with Steel Hysteretic Dampers, Proceeding of 17th Congress of IABSE, Chicago, 2008,
4) Architectural Institute of Japan, Standard for Structural Calculation of Reinforced Concrete Structures, February 2010 (in Japanese)
5) Kawano A., Griffith, M.C., Joshi, H.R. and Warner, R.F. : Analysis of Behavior and Collapse of Concrete Frames Subjected to Seismic Ground Motion, Depart. Civil and Environmental Eng., The University of Adelaide, Nov. 1998, Research Report No. R163
6) Minister of Land, Infrastructure, Transport and Tourism: Enforcement Order of the Building Standard Law, Notification No. 1793 of the Ministry of Construction /November 27, 1980, Stipulation of the value of Z, methods of calculating and $R_t$ and $A_i$ and standards for the designation by the Designated Administrative Organization of districts where the ground is extremely soft (in Japanese)
7) Popovics, S.; Numerical Approach to Complete Stress-Strain Curve of Concrete,"Cement and Concrete Research, Vol.3, pp.583-599, 1973
8) Meng, L., Ohi, K. and Takanashi, K. : A Simplified Model of Steel Structure Members with Strength Deterioration for Earthquake Response Analysis, Journal of Structural and Construction Engineering, AIJ, No. 437, pp 115-124, 1992 (in Japanese)
9) Masuda S., Sakino K., An Analytical Study on Cantilever Wall Systems, Annual Meeting of AIJ, Kyushu Chapter, March, 2008 (in Japanese)
10) Yasukochi J., et.al., Analytical Study on Seismic Performance of Composite Structural Wall-Frame Structure with Hysteresis Dampers, Annual Meeting of AIJ, August, 2008 (in Japanese)
11) Architectural Institute of Japan, Design Guidelines for Earthquake Resistant RC Building Based on Ultimate Strength Concept, Nov. 1990 (in Japanese)
12) Japan Association of Building Engineers: Building Construc. and Management JSCA Wave Calculations (JSCA-10·300·L2-1), 2002.6 (in Japanese)

（受理：平成23年6月2日）