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PASSIVE CONTROLLED SLENDER STRUCTURES HAVING SPECIAL DEVISES AT COLUMN CONNECTIONS

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ABSTRACT

The G is 0.98m/sec^2 and acting eternally. This value is two times of maximum horizontal acceleration of large earthquakes. The permanent large gravity forces caused by this acceleration to the Earth make buildings collapse and kill many people during and after big earthquakes. The structural systems for gravity forces such as columns and beams have to keep the original structural configuration under the excitations of earthquakes. Especially, the columns should not be collapsed in compression as shear failure or buckling failure.
In this paper, a new idea for very sender truss-type-columns having special column joints is discussed. The joints are strong enough for compression force but weak for tension force and have large deformation capacity.

1. INTRODUCTION

Flanges of chord joints in truss/column structures are conventionally reinforced with ribs. Furthermore, the joints are designed to provide tensile strength up to the fracture strength of the bolt. However, there is a risk of chord member buckling, which would cause a loss of support on the compressive side leading to building collapse as shown in Figure-1. This is because the chord members are subject to the constant compressive load of the building, as

well as alternating compressive and tensile loads during large earthquakes.
This study proposes the newly developed truss/column system shown in Figure-2. This system provides the chord joints with sufficient supporting strength against large compressive forces. Furthermore, a limiting force device (hereinafter called damper joint) is incorporated into the truss columns that allows plastic deformation only under tensile force .

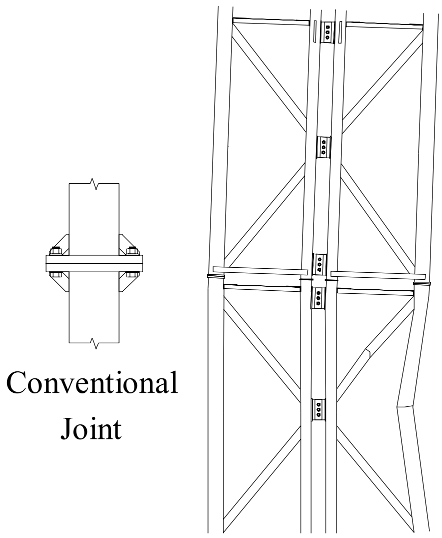


Figure-1 Conventional System

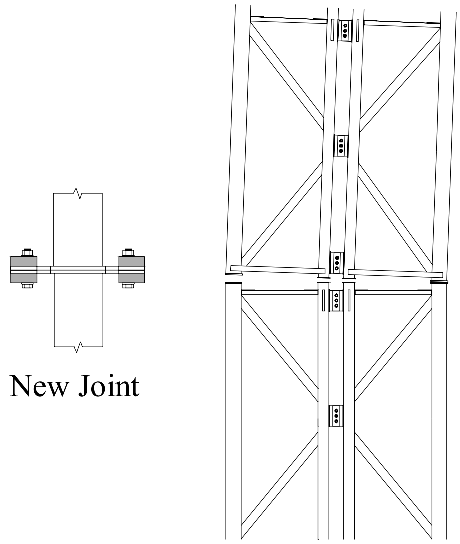


Figure-2 New System

2. MEMBER OF EXPERIMENTS ON DAMPER JOINT

2.1 Test specimen

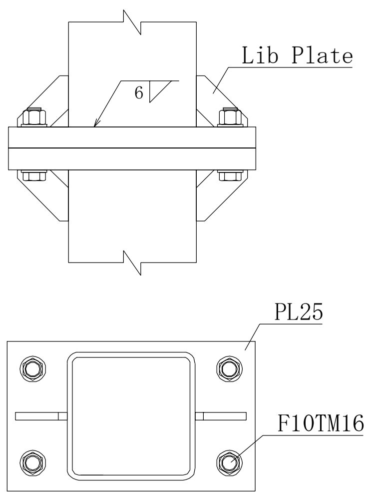
Two kinds of test specimens were used: a conventional joint and a damper joint. These specimens were extracted from the chord joints of the truss columns. Details of the specimens are shown in Figure-3 and 4. The damper joint specimens had four flange thickness: 6mm, 9mm, 12mm and 16mm. The joint configuration was designed to provide the flanges with deformation capacity. The flanges were made thin. The widths of parts expected to deform were decreased. The width were decreased to minimize the thermal effects of welding part of the steel pipes and the flanges on flange deformation. Furthermore, bolt edge distance was made large to minimize prying action.
Furthermore, the rigidity of the flange below the bolt was ensured by placing a rigid plate (thickness 30mm) underneath the bolt. This bolt was designed so that there was no clearance under it at the flanges. An initial tension was introduced into the specimen bolt to ensure that it was always in tension.

2.2 Experiment plan

In the experiment, the specimen was placed as shown in the left of Figure-5. A displacement-control-type static testing machine was used. The loading plan is shown in the right of Figure-5. The tension loading was controlled by displacement and followed the loading plan. The compression loading was controlled by load up to the point where the compressive stress at the steel pipe became 50MPa (compressive axial force 168kN). If the displacement did not return to 0mm even when the steel pipe reached this compressive stress, the loading was extended until the compressive stress became a maximum of 90MPa (compressive axial force 300kN).

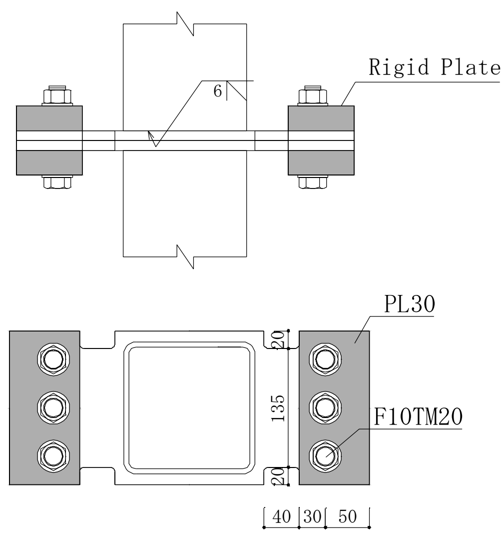
2.3 Experiment results and discussions

The hysteresis characteristics of the joint are shown in Figure-6. A hinge occurred in the damper joint specimen a at each end of the part where the flange width was reduced. Furthermore, it showed a stable hysteresis loop up to a 40 mm deformation cycle observed between the two central flanges (hereinafter called axial displacement). In the conventional joint specimen, the bolt reached failure at near the calculated fracture strength Tu of the bolt.



Square Pipe：□150x150x6
Flange Material：SS400
Flange Thickness：25mm
Bolt：F10TM16 4pieces

Figure-3 Conventional Joint



Square Pipe：□150x150x6
Flange Material：SN490C
Flange Thickness：6,9,12,16mm
Bolt：F10TM20 6pieces

Figure-4 New Joint

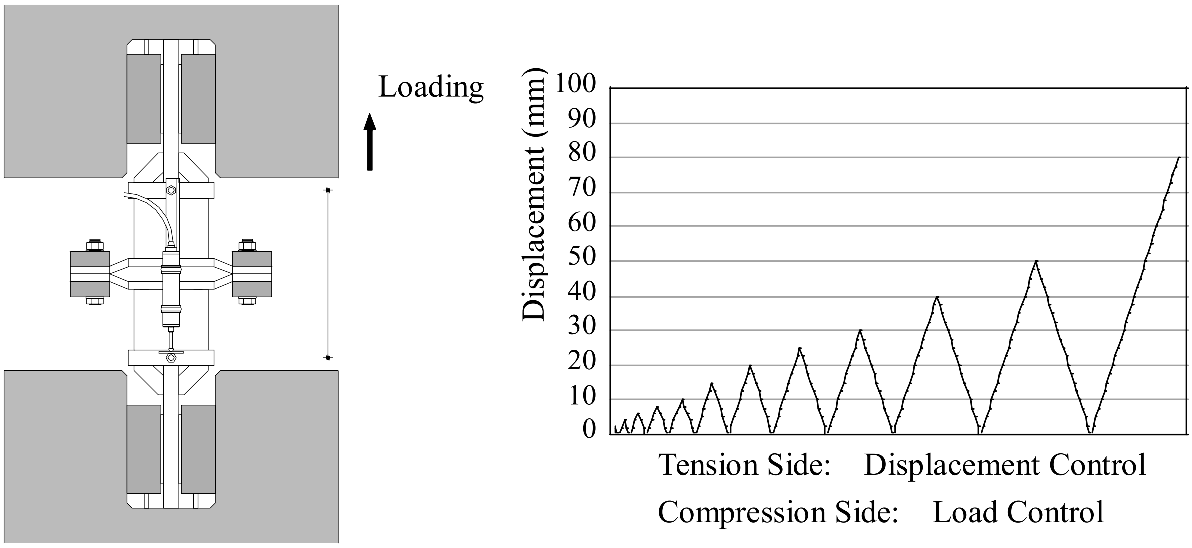


Figure-5 Loading Method

The structural performance of the specimens is shown in Table-1. In the quantitative evaluation of the energy absorbing capacity of the damper joint, the hysteresis curve was divided into the skeleton part and the Bauschinger part. The parameters used were the energy absorption sWp in the skeleton part, the energy absorption bWp in the Bauschinger part, and the total energy absorption Wp. It can be seen from Table-1 that the damper joint has higher deformation capacity and energy absorbing capacity than the conventional joint.

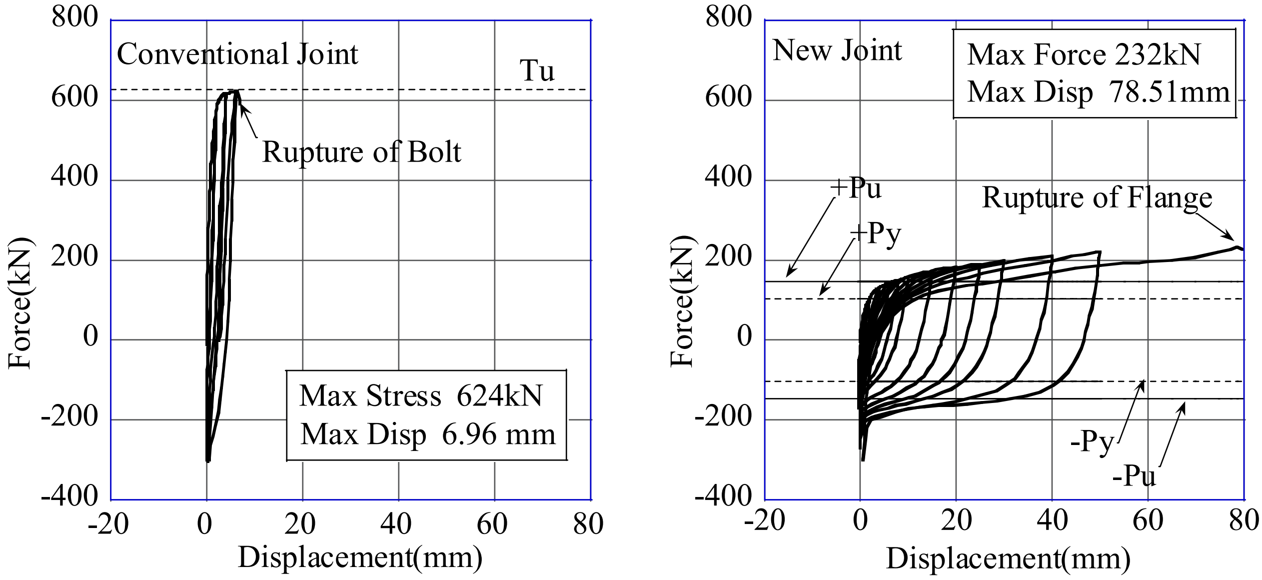


Figure-6 Hysteretc Behavior of Joint

Table-1 Performance of Joint

Specimen	6mm	9mm	12mm	16mm	Conventional
P_y (kN)	51	102	196	366	386
P_{max} (kN)	171	232	375	640	624
δ_{max} (mm)	85	79	40	52	7
${}_sW_p$(kNmm)	8400	10000	10600	21300	3300
${}_bW_p$(kNmm)	21600	48800	40500	23700	2100
W_p(kNmm)	30000	58800	51100	45000	5400

2.4 Modeling of damper joint hysteresis characteristics

On the basis of the experiment results, the modeling is aimed at approximating the hysteresis characteristics and the energy absorbing capacity of the damper joint up to an axial displacement of 30mm. The hysteresis model that accounts for the Bauschinger effect made by Akiyama was revised, and the revised model was used. The hysteresis model incorporated the elastic unloading rigidity K1, the rigidity K2 of the tensile side of the skeleton part, the rigidity K3 of Bauschinger part, the rigidity K4 of the compressive side of the skeleton part, and the rigidity K5 for the bearing of the upper and lower flanges when the axial displacement was smaller than 0mm. Figure-7 compares the experimental values and the model values for the 9mm damper joint specimen. For all specimens, this model closely simulated the experimental hysteresis behavior, and was applicable to 90% of total energy absorption Wp and 70 through 90 % of individual parts.

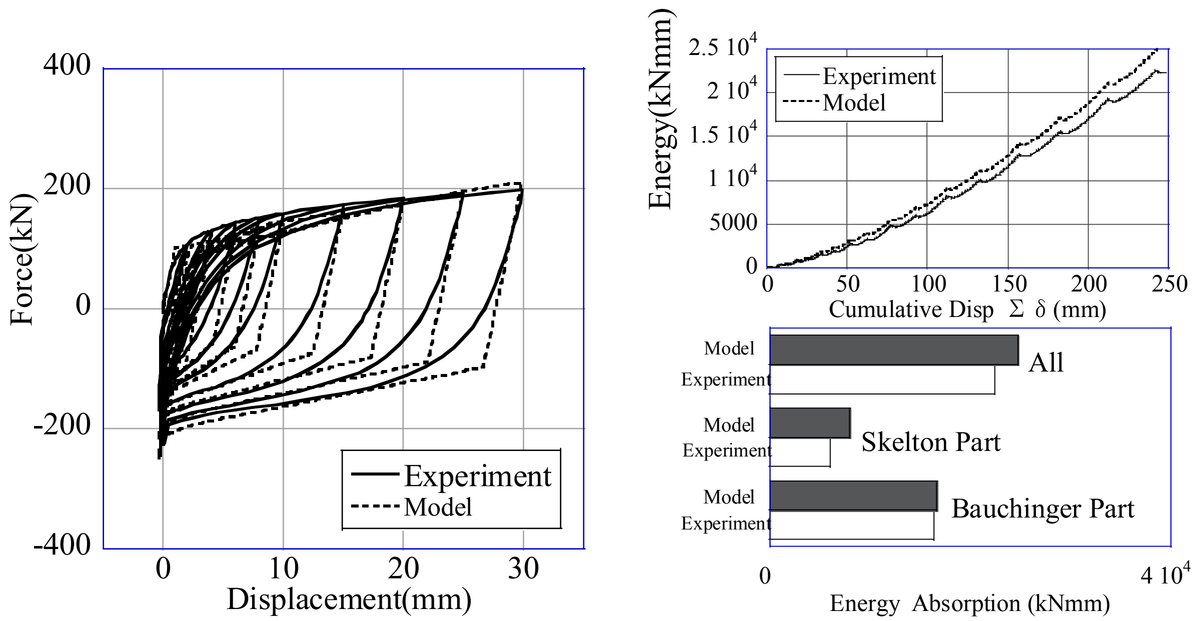


Figure-7 Correspondence of Model and Experimental Result

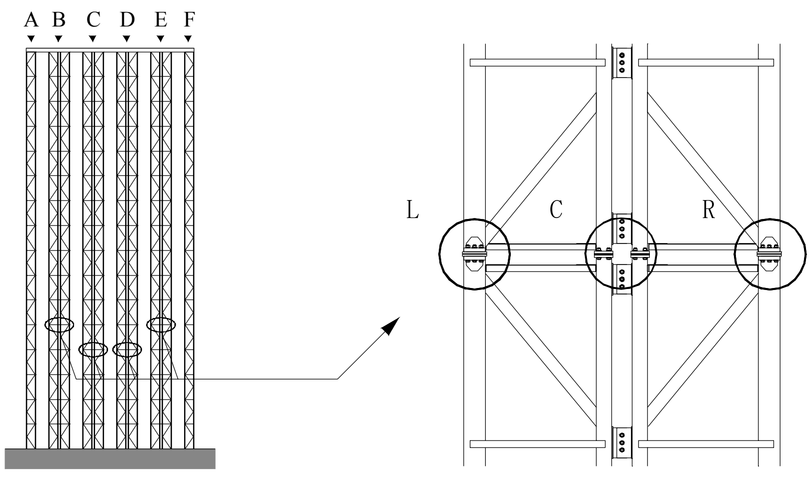
3. SEISMIC RESPONSE ANALYSIS FOR TRUSS COLUMN STRUCTURES

3.1 Analysis building

The analysis building is shown in Figure-8. It is a rack warehouse 49.7m high with a primary natural period of 2.73sec. The truss column structure was analyzed where the chord members were placed with the damper joints installed, and the truss column structure was analyzed where the chord member joints were made as rigid joints. The response properties of these two structures were compared. The joints were placed at Columns B through E, whose layout is shown in the right of Figure-8. In the damper joints, steel members SN 490C 16mm thick and SS 400 6mm thick were employed for the right/left chord members of the damper joint and for the central chord member, respectively.

3.2 Analysis model

The analysis building was converted to a dynamic model, as shown in Figure-9. The masses were concentrated at each node. The top beam was modeled to have a rigid zone at the part that corresponded to the width of the truss column. Furthermore, the beam end part was modeled to create a hinge at the beam ends when the total plastic moment was reached. The damper joint part was modeled as three parallel springs that assumed Navier's hypothesis. Individual springs expressed the joints of the truss columns. The central two chord joints were integrated into one spring. For the hysteresis characteristics of each spring, the hysteresis model described in the previous section was employed.



Story	A, F		B～E		Brace
	Outside Chord	Inside Chord	Outside Chord	Center Chord	
1～2	□150x150x12	□175x175x12	□175x175x12		
3～5	□150x150x9	□175x175x9	□175x175x9	□125x125x4.5	□100x100x3.2
6～16	□150x150x6	□175x175x6	□175x175x6		

Top Beam	H396x199x7x11

Figure-8 Analysis Building

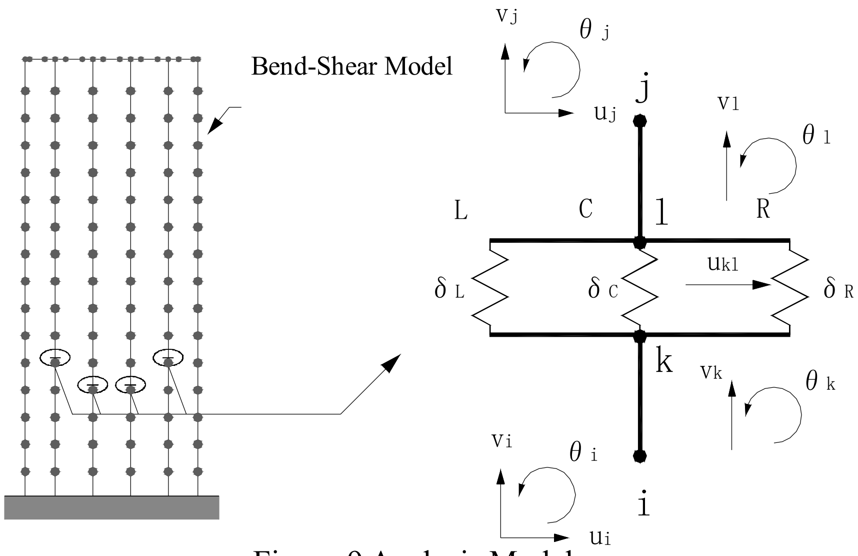


Figure-9 Analysis Model

3.3 Analysis results and discussions

The experiment employed the input seismic movement of the Hachinohe EW in 1968 with its maximum velocity record amplified to 50 kine. From the maximum response shown in Figure-10, it is found that the damage control truss structure reduced the maximum relative displacement and the moment of the lowest layer by 14% and 27%, respectively. As a result, the following findings were made. While the maximum compressive stress in the chord members exceeded the design buckling stress in the conventional truss structure, those in the damage control truss structure did not reach the design buckling stress.

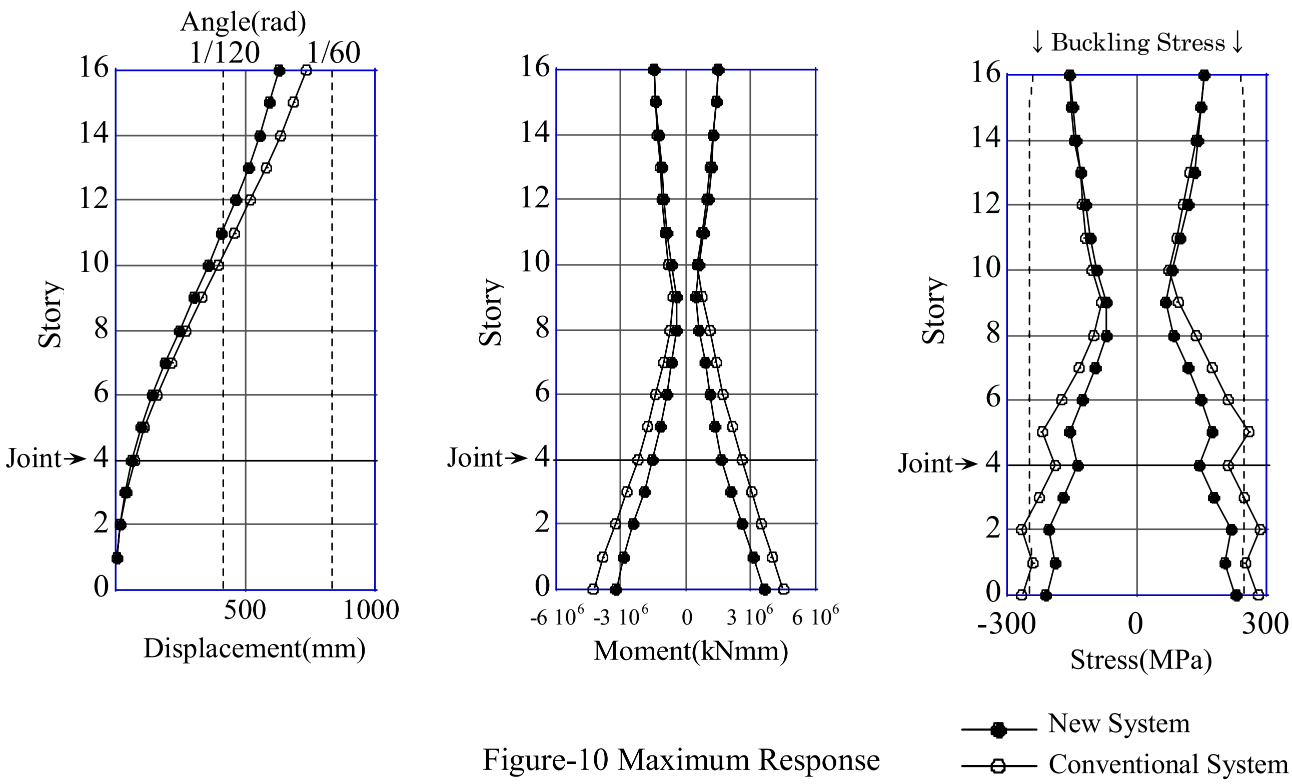


Figure-10 Maximum Response

The history of each input energy is shown in Figure-11. The quantity of each input energy is shown in Table-2. The maximum value of the elastic strain energy of the main structure in the damage control truss structure was decreased by 35%. From these analysis results, the following conclusions can be drawn. For the damage control truss structure, during the large earthquakes, base isolation effect can be expected in the upper structure of the damper joint by plasticizing of the damper joints and by lifting of the chord members. Furthermore, the overturning moment acting from the upper part to the lower layer can be reduced. Thus, the damage control truss structure avoids buckling of the chord members of the lower layers. The maximum deformation and the energy absorption of each damper joint set in place are shown in Figures-12 and 13. The deformation and the energy absorption of the damper joints did not exceed the values obtained from the member experiments. Therefore, there is found to be no risk of failure in the joints for this input.

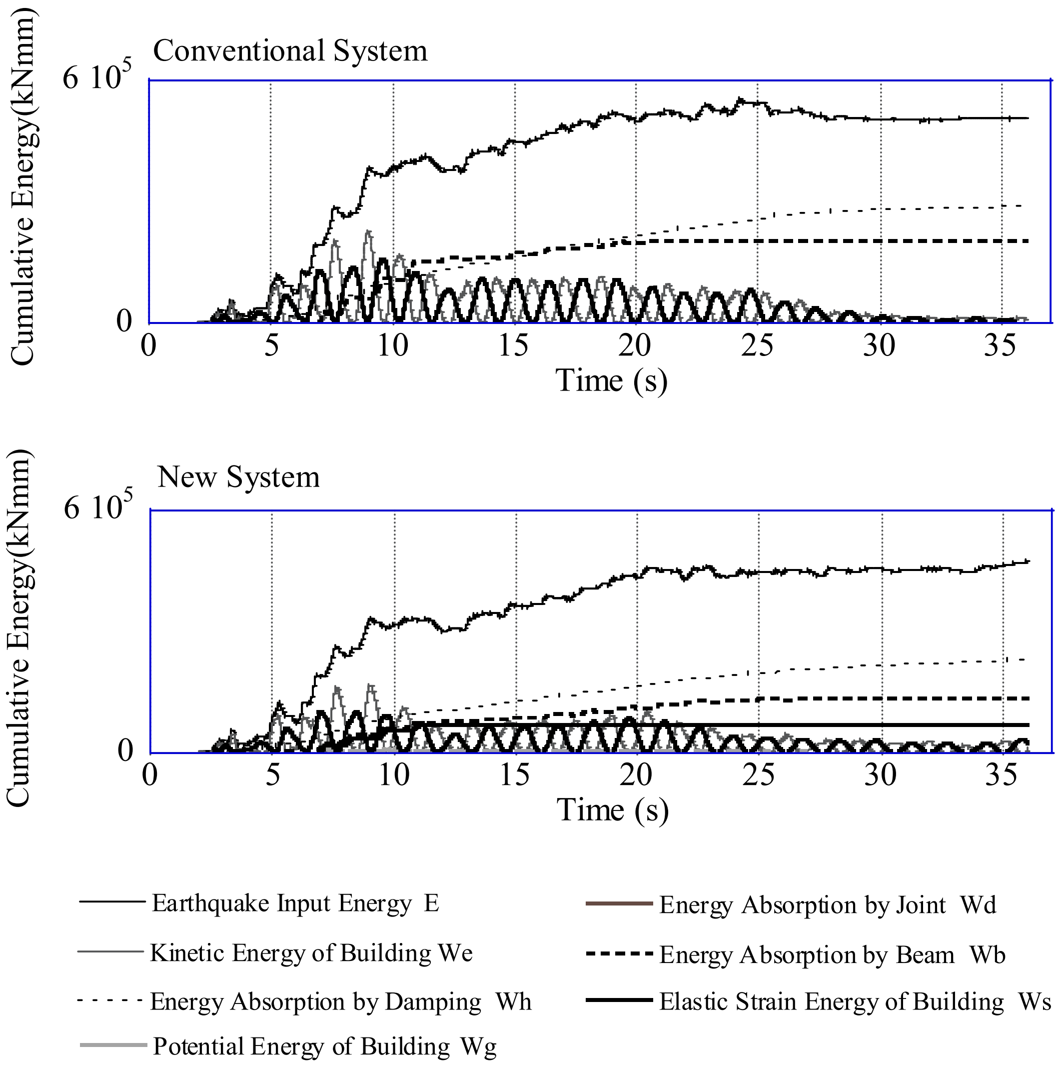


Figure-11 Time History of Energy

Table-2 Energy Amount

Energy	Conventional System	New System
E (kNmm)	507700	473900
Wh (kNmm)	291800	231100
We (kNmm)	9200	8600
Ws (kNmm)	3500	25600
Wd (kNmm)	-	136100
Wb (kNmm)	203300	69900
Wg (kNmm)	0	2500
Ws_{max} (kNmm)	158700	103600

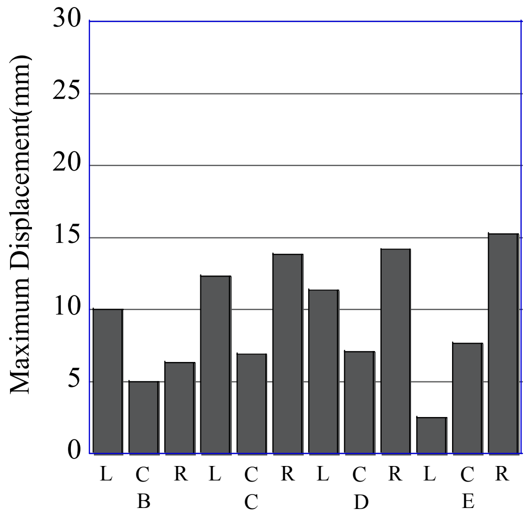


Figure-12 Maximum Displacement of Joint

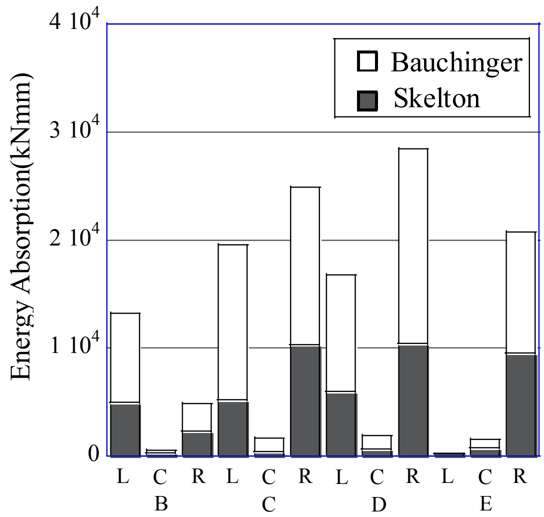


Figure-13 Energy Absorption of Joint

4. TRUSS COLUMN PARTIAL FRAME EXPERIMENT

4.1 Specimens

From the seismic response analysis, the inflection points of moment in the truss column were observed in eighth layers. Therefore, the specimens were extracted from the mid-point of the fourth through the eighth layers of the analysis building. Experiments were conducted on specimens where the damper joints were installed in the chord members (hereinafter called damage control truss frame) and specimens where the joints of the chord members were rigid joints (hereinafter called conventional truss frame). Then, the difference between the properties was identified. By applying a horizontal load to the specimen top at the inflection point, the stress configurations at the specimens and at the truss columns that occurred during an earthquake were matched. The joints were placed between the fifth and sixth layers. The specimens were taken from midway between the fourth layer the above so that the boundary conditions of the chord members in the lower layer would cause buckling identical to that in the real truss column.

4.2 Experiment plan

As shown in Figure-14, the specimen was laid down and the experiment was carried out with the specimen's leg fixed to the footing beam. Firstly, tension was applied to the PC steel bars placed along the chord members using a hydraulic jack, and a constant vertical load was applied to the chord members. During the horizontal loading, the load was controlled by a load cell placed between the hydraulic jack and the fixing bolt. A constant vertical load was applied by horizontally moving the lower axial force beam along with the movement of the top. After the vertical load was applied, the horizontal load was applied using the upper hydraulic jack. The loading plan is shown in Figure-15. Loading was controlled with a deformation angle against the height of the specimen at the top (hereinafter called top deformation angle). The loading was determined to be static repeated loading. Furthermore, a buckling preventer was introduced to prevent out-of-plane deformation of the specimen during the horizontal loading. The buckling preventer was placed where the buckling length of the specimen's chord member became the same as the buckling length in the real truss column structure.

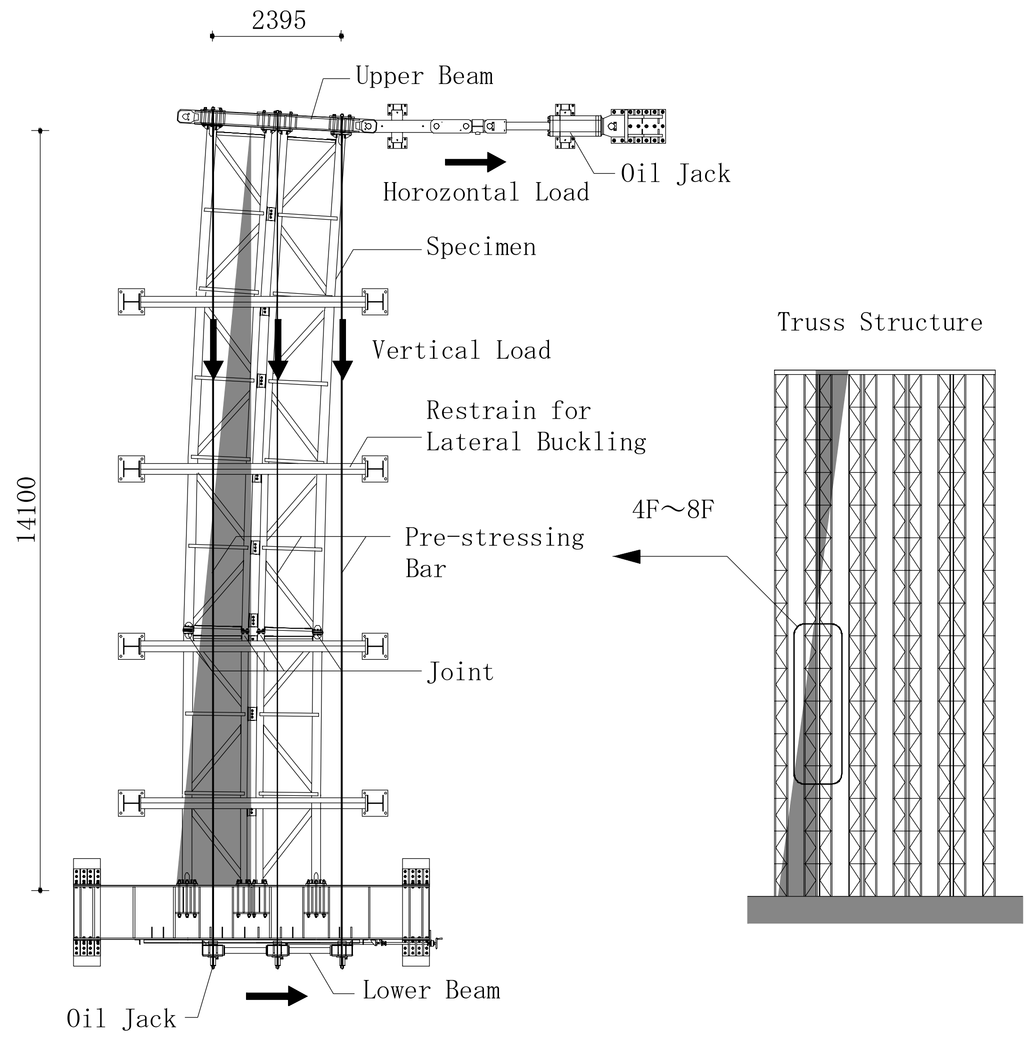


Figure-14 Loading Method

Table-3 Test Specimen

Member	Chord on Joint	Chord under Joint	Inside Chord	Brace
Section	□ 175x175x6	□ 175x175x9	□ 125x125x4.5	□ 100x100x3.2
σ_E(MPa)	308	271	322	319
σ_y(MPa)	379	409	434	435
λ	45	46	63	36

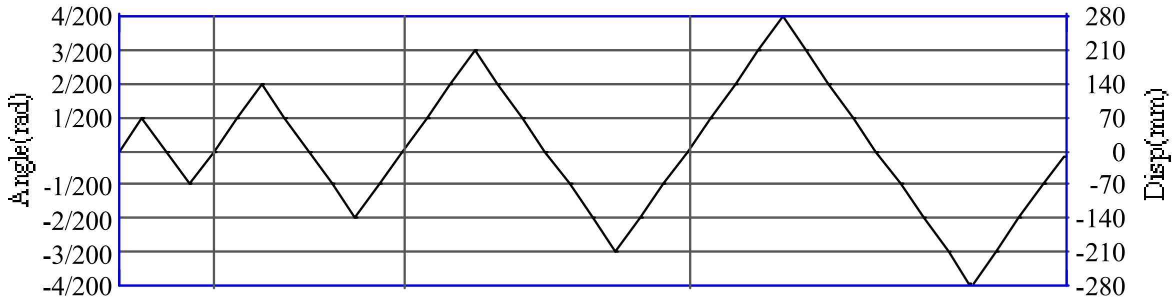


Figure-15 Loading Program

4.3 Experiment results

Figure-16 shows the horizontal load at the specimen top and the hysteresis characteristics. The damage control truss frame showed a stable loop up to the cycle with a top deformation angle of 3/200 (horizontal displacement 210 mm) and reached the damper joint flange failed at the cycle with a top deformation angle of 4/200 (horizontal displacement 280 mm). No members had buckled up to the point where the flange failed. The conventional joint specimen showed compressive strain in the chord member caused by the diagonal member at the compressive side at the cycle with a top deformation angle of 2/200 (horizontal displacement 140 mm). Furthermore, it showed decreased rigidity due to the extrusive chord member strain caused by the diagonal member on the tensile side. Finally, the chord member buckled at a top deformation angle of 4/200 (horizontal displacement 280 mm). Figure-17 shows the stress in the chord member immediately above the joint. The stress in the chord member in the damage control truss frame is smaller than the elastic limit stress of the chord member obtained from the material experiment. The chord stress in the conventional truss frame exceeded the yield stress of the chord member obtained from the material experiment at the top deformation angle of 3/200 (horizontal displacement 210 mm). Thus, buckling could be expected at this point in the chord member. Figures-18 and ・9 show the hysteresis characteristics and structural performance, respectively, of the damper joint obtained from the experiment,. The structural performance of the joint exceeds the deformation.

capacity and the energy absorbing capacity required for the Hachinohe EW 50 kine obtained from the response analysis. The experiment confirmed that the damage control truss frame maintained the main structure within the elastic region and provided sufficient ductility in the frame by limiting the stress in the damper joint. The conventional truss frame also showed a level of deformation capacity, but it would be difficult to incorporate this ductility in the design and to predict the material strength of the chord member obtained in the experiment.

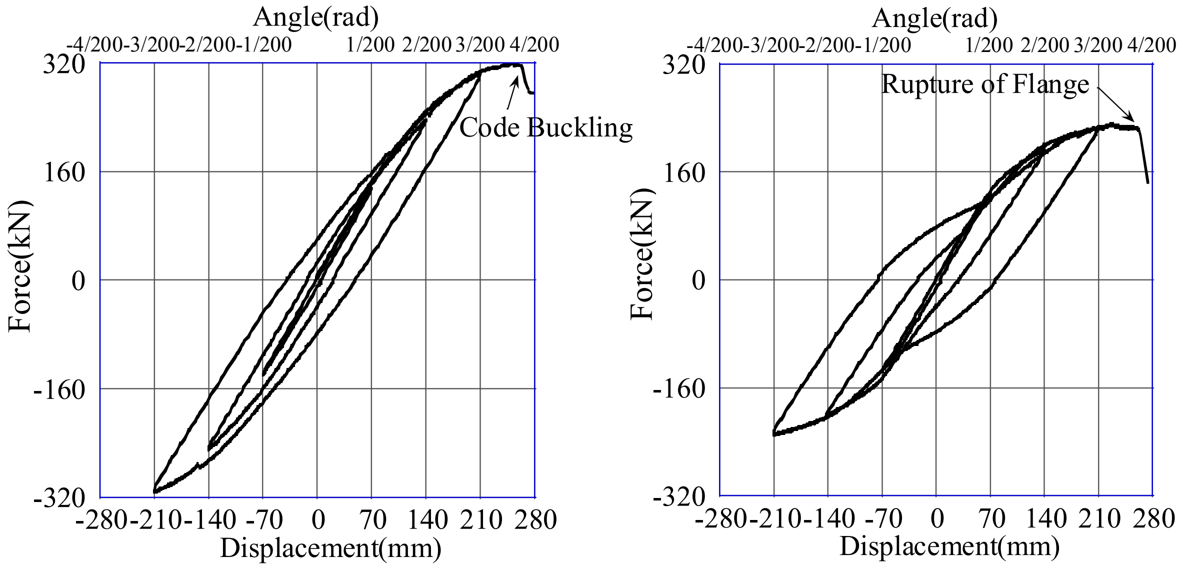


Figure-16 Hystretic Behavior of Partial Frame

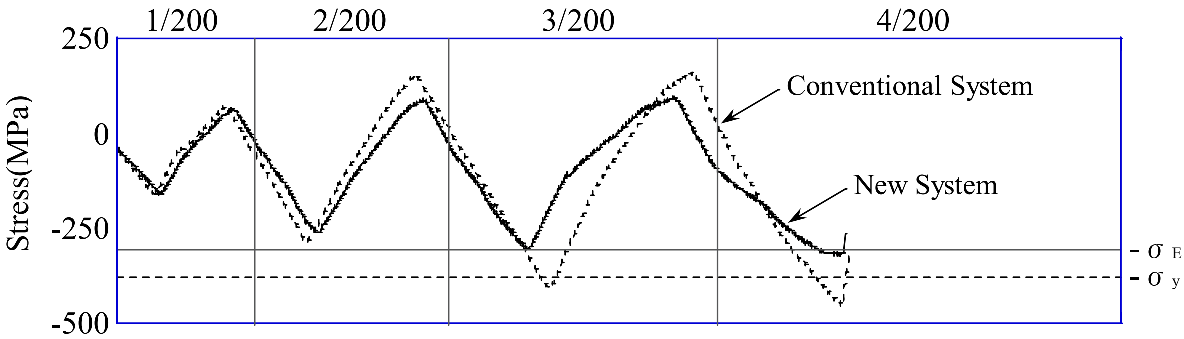


Figure-16 Hystretic Behavior of Partial Frame

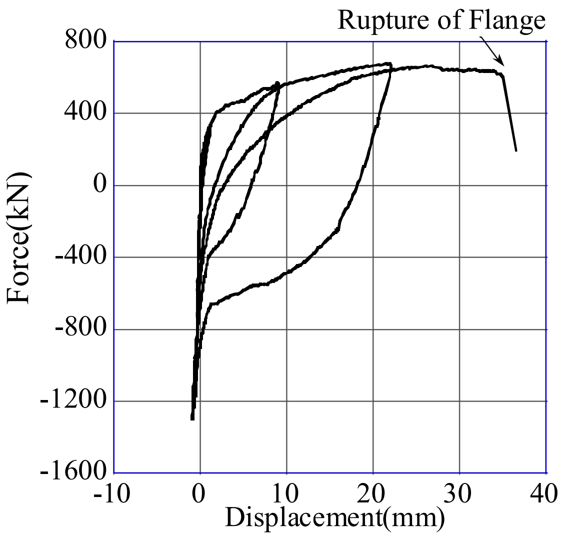


Figure-18 Hystretic Behavior of New Joint

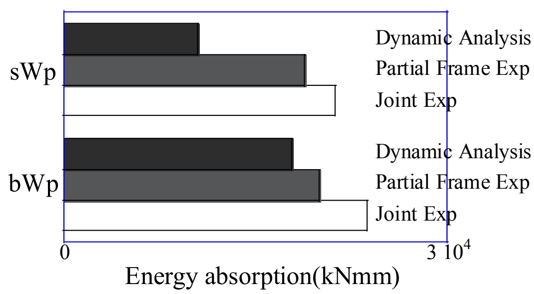


Figure-19 Performance of New Joint

5. CONCLUSIONS

The following findings were obtained from this study.

1) The damper joint was shown to provide excellent deformation capacity and energy absorbing capacity.
2) The response analysis showed that the seismic input to the building was decreased by installing the limiting force device at the chord joint of the truss column.
3) The real-scale partial frame experiment confirmed that member buckling was prevented and a high ductility frame could be constructed by installing the limiting force device at the chord member of the truss column.