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Experiments of Damage-controlled Truss Beam with Buckling Restrained Member

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Abstract

In this study, as a new method to realize a resilient large space structure, we tried at realizing a truss frame that is not only seismic performance but also economical and restoration. Steel frame with truss beam, which is usually designed as elastic structure against seismic loading, can be designed as ductile and energy-absorbing structure by replacing lower chords to buckling restrained members. In this paper, cyclic loading tests of a normal truss beam and a damage-controlled type truss beam with a buckling restrained (BR) member were conducted to examine energy dissipation capacity and damage state of the specimens as damage-controlled truss beam.

The specimens are comprised of three models, which are cantilever truss beams with a total length of 10m. Specimen TRUSS1 is a normal type without BR member and TRUSS2 and TRUSS3 are damage-controlled types with BR members which are installed at the second lower chord positions from the fixed end that are critical positions against buckling. TRUSS2 and TRUSS3 have different degree of fixation of connections of diagonal member and upper chord just above the BR members. TRUSS2 had rigid connection, and in contrast TRUSS3 had a semi-rigid connection to improve rotation characteristics.

As a result, the following conclusions can be drawn. (1) TRUSS1 showed strength deterioration due to the global and local buckling of the compression horizontal chord member in R =0.0125rad. The maximum strength of TRUSS1 was 171kN, it was larger than those of TRUSS2 and TRUSS3, however less ductile. (2) Although the maximum strengths of TRUSS2 and TRUSS3 were about half that of TRUSS1, they showed stable elasto-plastic behavior up to R = 0.02rad. Equivalent plastic deformation capacity of TRUSS2 and TRUSS3 were more than 6 times that of TRUSS1, indicating excellent ductility. (3) Ultimate state of TRUSS1 was determined due to local buckling and out-of-plane buckling of the upper and lower chord members around the fixed end. On the other hand, TRUSS2 and TRUSS3 using the BR member avoid the buckling in their members. TRUSS2 showed slight plasticity around the BR member, but TRUSS3 showed no damage. TRUSS3, which the upper chord joint behaved as a semi-rigid joint, and it also reduced the damage to the joint.

Seismic performance such as strength and ductility of the proposed truss beam frames can be controlled by designing the BR member. It suggests an excellent seismic performance and restorability that enables continuous use by slight inspection of the BR member even for unexpected large earthquakes.

Keywords: Parallel Chord Truss, Buckling Restrained Member, Damage-controlled, Ductility, Restorability

1. Introduction

Truss frames have been used conventionally for spatial structures such as production facilities. Truss beams can be rationally designed for long-term loads. On the other hands, seismic performance of the truss beam is determined by buckling of chords. Therefore, in the current design code of Japan usually requires elastic design where the occurrence of buckling is defined as ultimate limit state neglecting the ductility of truss beams, and those members are given sufficient strength. However, it is difficult to specify the damage state for a larger earthquake than expected, and it is difficult to realize both economical design and quick restoration after an earthquake.

On the other hand, it is possible to enhance the ductility by providing force-limited mechanism to the part of members and to make the damage-controlled truss frame structure. It has been studied the improvement of ductility of truss beams using plasticity of shear panels[1], and eccentric connections[2], and buckling restrained (BR) members such as double steel pipes[3, 4], as force-limited mechanism.

In this study, as a new method to realize a resilient large spatial structure, we tried at realizing a truss frame that was not only seismic performance but also economical advantage and restorability after an earthquake. In this paper, cyclic loading test of a damage-controlled type truss beam (Fig.1) with a BR member around the truss beam end are conducted to examine the capacities and ductility of the beam.

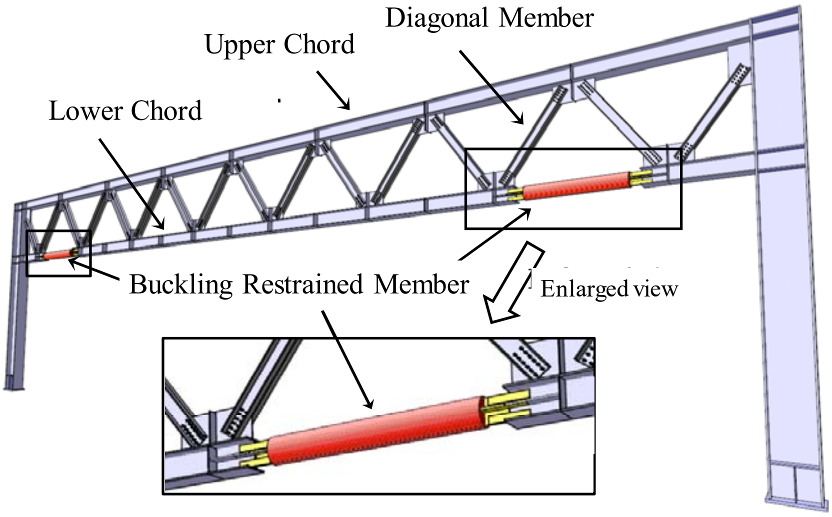


Fig. 1 – Damage-Controlled Truss Structure

2. Experimental Program

2.1 Specimen

Details of test specimens are illustrated in Fig.2 and the test parameter of specimens is listed in Table 1. The specimens are 1/2 scaled parallel chord trusses in the shape of cantilever which represents the half span of Warren truss beams with a span of 40 m. By applying concentrated loads to the vertical direction at the tip of the specimens, the stresses of the truss beams by seismic loads are generated. The length from the fixed end to the force-applied end is L = 10m, and the distance between the center of gravity of the upper and lower chords is h = 1.25m. A rolled H-section steel of H-150x150x7x10(SN400B) was used for chord members, so that the strong axis was perpendicular to the force-applied plane. The diagonal member is double C-75x40x5x7 (SS400). The specimens are rigid trusses with no eccentricity in joining of the individual members. The chords and the diagonal members are friction-joined with high-strength bolts to gusset plates welded to the flange of the chords.

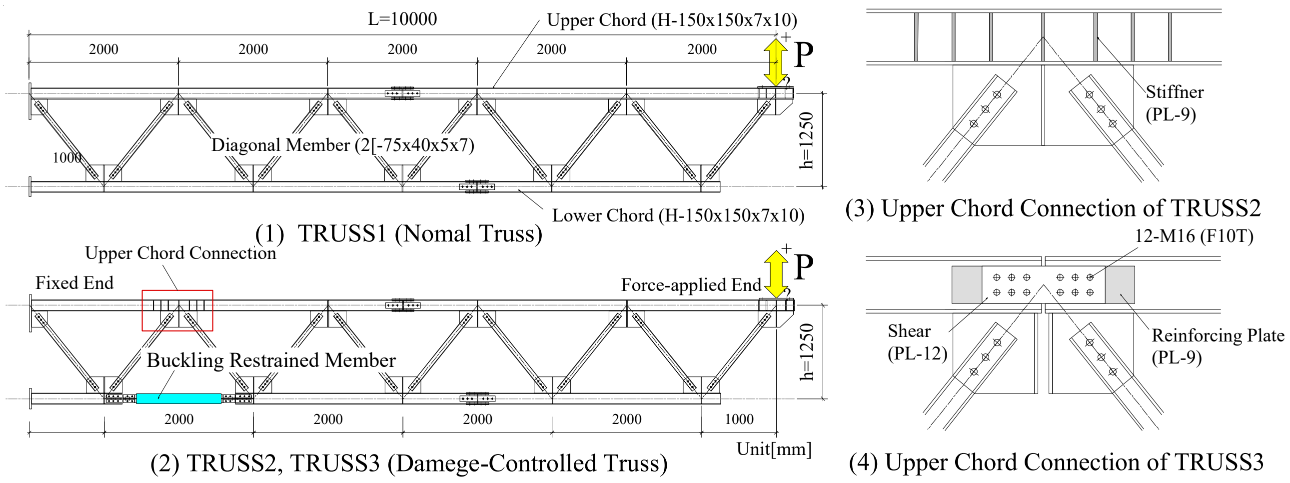


Fig. 2 – Details of Specimens

Table 1 – Condition of Specimens

Specimens	**Types**	**Buckling Restrained Member**	**Upper chord connection**
TRUSS1	Nomal Truss	None	Rigid Joint
TRUSS2	Damage-Controlled Truss	Existence	Rigid Joint (Stiffeners)
TRUSS3			Semi-Rigid Joint

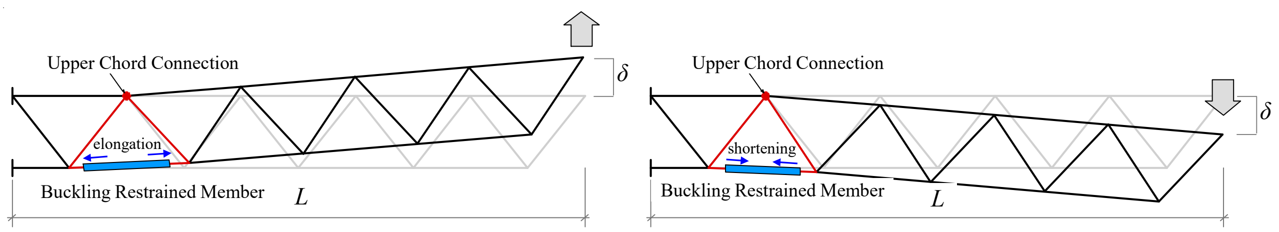


Fig. 3 – Characteristics of Deformation

As shown in Fig.2, specimen TRUSS1 is a normal truss without BR member and TRUSS 2 and TRUSS 3 are damage-controlled trusses with BR members in the lower chord. TRUSS 2 and TRUSS 3 have different details at the connection between the upper chords and diagonal members (named "the upper chord connection") right above the BR members. The BR members in TRUSS 2 and TRUSS 3 are installed in the second lower chord from the fixed end which are the most critical positions against buckling.

The BR members have a structure in which core materials (SN400B) with a cross section of 16×80 mm are covered with steel pipe filled with concrete [5]. The axial yield load of the core materials of the BR members are 375 kN. Ratio of the buckling strength of the conventional chord to the yield strength of the BR member is designed to be 2.5 not to occur buckling in the members.

Characteristics of deformation about TRUSS2 and TRUSS3 are shown in Fig. 3. When axial deformation due to yielding of the core materials progress in the BR members, the truss beams shows deformation that bends upper chords at the joint, so that adequate deformation followability is required for these joint part (for example Ref.[6]).

The upper chord connection of TRUSS2 is a rigid joint similar to other joints, furthermore the chord is reinforced with stiffeners as shown in Fig. 2 (3). On the other hand, the upper chords of TRUSS3 are separated as shown in Fig. 2 (4), and realize semi-rigid joint using shear plates (PL-12) which connect webs of the upper chords with high-strength bolts (12-M16). At this time, in order to increase the strength of the webs of upper chord connections, additional reinforcing plates (PL-9) are attached to both sides of the webs. In this way, the upper chord connections of TRUSS3 has a low rotational stiffness, and has improved deformation followability comparing to TRUSS2. Coupon test results of the steel used for the members are summarized in Table 2.

Table 2 – Coupon Test Results of Materials

Member		**Section**	**Material**	**Yield Point (N/mm^2)**	**Tensile Strength (N/mm^2)**	**Breaking Elongation (%)**
Upper Chord	**Flange**	H-150x150x7x10	SN400B	306	444	43
	Web			329	457	40
Diagonal Member		C-75x40x5x7	SS400	312	478	33
Shear Plate		PL-12	SN490B	372	576	33
Other Plate		PL-9	SS400	370	464	38
Buckling Restrained Member (Core Material)		t=16, w=80	SN400B	293*	440*	36*

*By Mill Sheet

2.2 Load application and measurement

Test setup and loading system are shown in Photo 1 and Fig. 4, respectively. The specimens are fixed to the reaction wall through a fixing jig. The specimens and the fixing jig are joined with high-strength bolts through a thick end plate welded to the fixed end of the chords. A 1000kN hydraulic jack was connected to the tip of the specimen, and static force was applied in the vertical direction. With regard to the lateral stiffening of the truss beam, as shown in Fig. 4 (1), lateral bracing pillars are provided at all positions where the upper chords and diagonal members intersect constraining horizontal movement of the specimens.

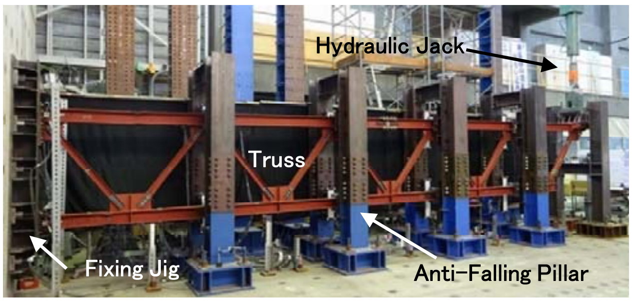
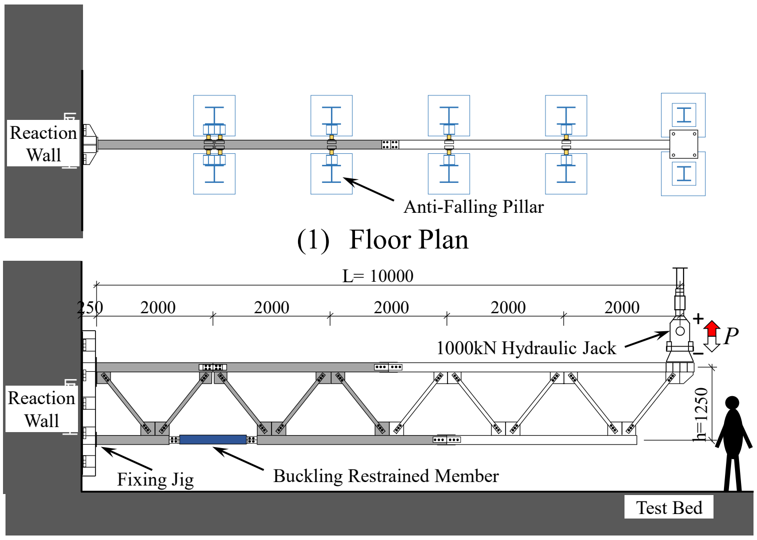


Photo 1 – Test Setup (TRUSS1)



(1) Floor Plan

(2) Elevation

Fig. 4 –Loading System

Specimens are loaded by cyclic and incremental loading protocol based on rotation angle $R(=\delta/L)$ of the whole beam which is defined the ratio of vertical displacement δ of the hydraulic jack to the span L. Peak values of the loading amplitudes are $R=\pm0.0025$, ±0.005, ±0.0075, ±0.01, ±0.0125, ±0.015, ±0.02rad, respectively and each amplitude have two cycle loadings. In the case that specimens have residual strength after final cycle, push over loading in the positive displacement side was conducted.

The arrangement of measuring vertical displacement is shown in Fig.5. Vertical load P at the tip of the truss beam are measured by load cell connected to the jack, vertical displacements at each node are measured by LVDTs, and strain of each individual members (▼) are measured by strain gauges.

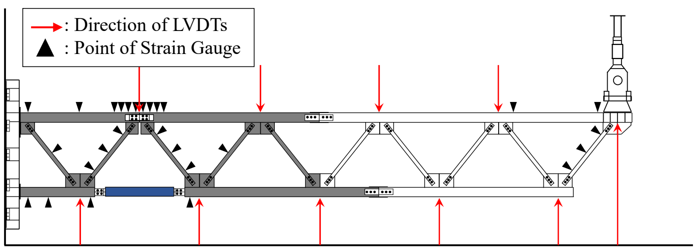
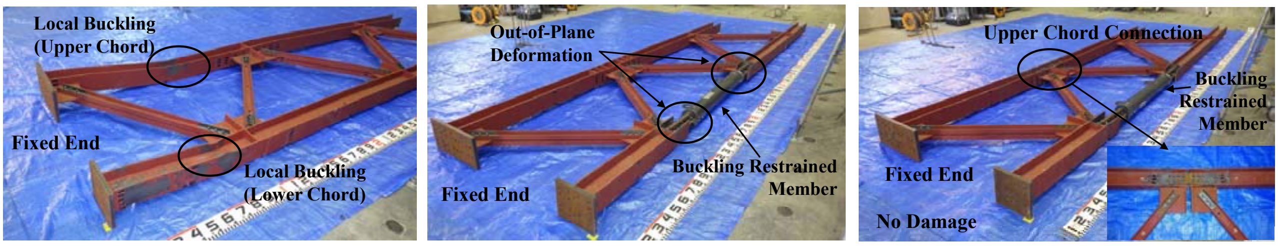


Fig. 5 – Arrangement of Measuring Points

3. Experimental Result

3.1 Damage conditions at the end of tests

Damage states of each specimen after the end of tests are shown in Photo 2. The conventional truss TRUSS1, shown in Photo 2 (1), the ultimate state determined due to local buckling of the upper and lower chord flanges at the fixed end and out-of-plane buckling of chords. TRUSS2 with the rigid upper chord connection, as shown in the Photo 2(2), showed slight bending deformation to the out-of-plane direction at both ends of the BR member but no significant deterioration was observed in individual members and BR member. In addition, due to the additional bending that is occurred by yielding of the upper chord slight bending deformation remained in the upper chord straddling the part. TRUSS3, as shown in the Photo 2(3), which has a semi-rigid joint at the upper chord, did not show any bending deformation in the BR member or upper chord like a TRUSS2, and damages such as buckling of individual members were not observed.

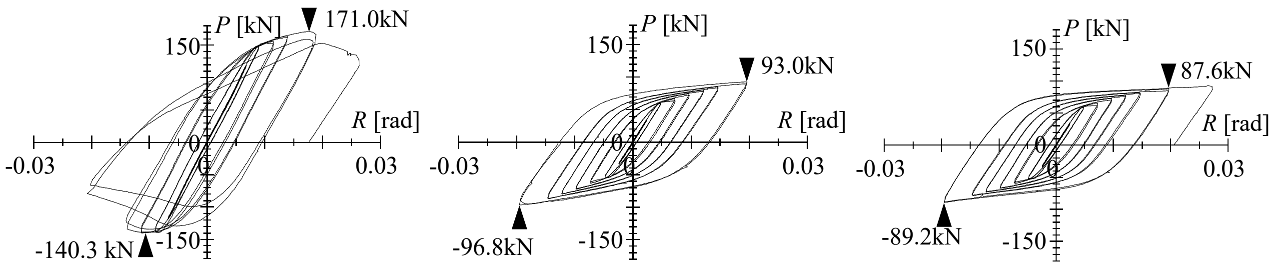


(1)TRUSS1 (2)TRUSS2 (3)TRUSS3

Photo 2 – Damage Conditions of Each Specimen After the End of Tests

3.2 Hysteresis curves

Hysteresis curves, the tip load and rotation angle of the whole beam relationships of each specimen are shown in Fig.6. In the figure, vertical axis represents the vertical load P of the hydraulic jack, and horizontal axis represents the rotation angle of the whole beam R. The points marked with ▲ are the maximum load points up to $R = \pm$ 2.0%, and the values are also shown around these marks.



(1) TRUSS1 (2) TRUSS2 (3) TRUSS3

Fig. 6 – Hysteresis curves

As shown in Fig. 6 (1), TRUSS1, a conventional truss, showed almost linear elastic up to $R = \pm$ 0.005 rad. Subsequent applied load caused decrease in rigidity, but a stable hysteresis curve was shown up to $R = \pm$ 0.01 rad. In the vicinity of R = -0.0125 rad, overall buckling of the member occurred in the lower chord at the fixed end, and the strength deteriorated was observed when the maximum load on negative side reached -140.3 kN. For positive side, the maximum load reached to 171 kN at R = + 0.02 rad. At this time, no damage such as local buckling was observed on the upper chord at the fixed end, however after showing the maximum load local buckling and strength deterioration occurred due to subsequent load for both positive and negative loading.

On the other hand, hysteresis curves of TRUSS2 and TRUSS3 in Figs. 6 (2) and (3) showed a stable behavior. In both cases, the BR member yielded around the first $R = \pm$ 0.005 rad, resulting in decrease in stiffness, and showed a gradual strain hardening until $R = \pm$ 0.02 rad. The loads at R = + 0.02rad are 93.0kN for TRUSS2, 87.6kN for TRUSS3, and those at R = –0.02rad are -96.8kN for TRUSS2, and -89.2kN for TRUSS3 respectively, which are about half of the maximum load of TRUSS1. It indicates that the BR member works as force-limited mechanism. The load of TRUSS2 is about 10% larger than that of TRUSS3, this is because bending resistance of the upper chord rigid joint of TRUSS2 is added to the its strength.

3.3 Ductility of truss beams

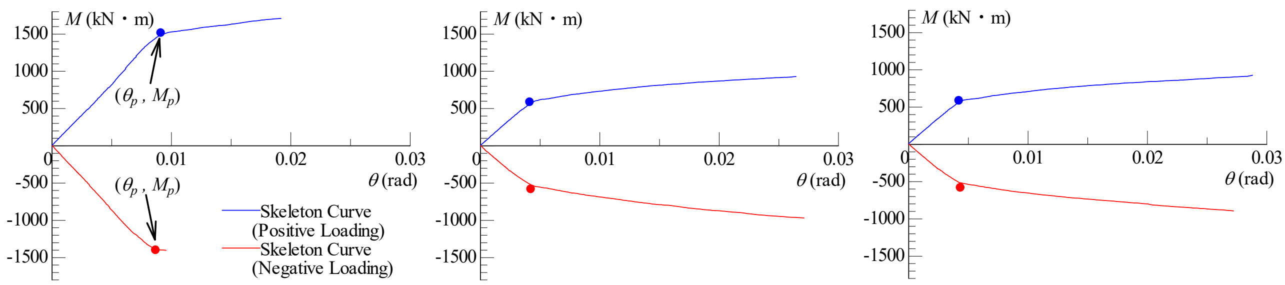
The skeleton curves, which are converted into moment (M=P×L) -rotation angle (θ=R) relationship, are shown in Fig.7. Those skeleton curves were created by the method shown in Ref. [7]. The equivalent plastic deformation capacity for both positive and negative sides obtained from the skeleton curves are listed in Table 3. The equivalent plastic deformation capacity is calculated by equation (1).

$$ {}_M\bar{\eta}_s = {}_M W_{ps} / (M_p \cdot \theta_p) \quad (1) $$

${}_M W_{ps}$: plastic strain energy disspations of the whole member, M_p: full plastic moment (TRUSS1:moment when the chord buckles, TRUSS2,3:moment when the BR member yields), θ_p :elastic deformation corresponding to full plastic moment

Table 3 – Equivalent Plastic Deformation Ratio

	Loading	M_p kN・m	θ_p %	${}_M W_{ps}$ kN・m	${}_M\bar{\eta}_s$
TRUSS1	Positive	1,513	0.91	14.4	1.0
	Negative	-1,403	0.87	1.80	0.1
TRUSS2	Positive	584	0.41	16.0	6.6
	Negative	-584	0.42	15.8	6.5
TRUSS3	Positive	584	0.41	17.8	7.2
	Negative	-584	0.44	15.0	6.1



(1) TRUSS1 (2) TRUSS2 (3) TRUSS3

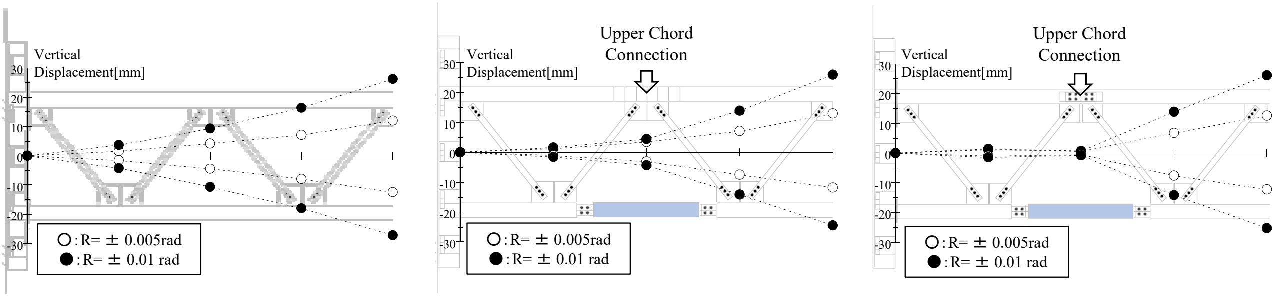
Fig. 7 – Skeleton Curves of Moment-Rotation Angle Relationship

The equivalent plastic deformation capacity of TRUSS1, a conventional truss beam, is 1.0 in a positive force and 0.1 in a negative force. For the damage control type truss TRUSS2, equivalent plastic deformation capacity is 6.6 in positive force, 6.5 in negative force and that of TRUSS3 is 7.2 is positive force, 6.1 in negative force respectively. In both cases, the equivalent plastic deformation capacity were almost same in both positive and negative loadings, and showed excellent ductility of 6 times and more of TRUSS1.

3.4 Vertical displacement distribution of truss beams

Vertical displacement distributions of the chord around the buckling restrained member of each specimen are shown in Fig.8. ○ marks in the figure indicate the displacement distribution at R = ± 0.005rad where yielding of the buckling restrained members of TRUSS2 and TRUSS3 starts, and ● marks indicate R = ± 0.01rad where yielding of the buckling restraint members has proceeded. The vertical displacement distribution of TRUSS1 shown in Fig. 8(1) shows a smooth deformation curve for both member angles, like a cantilever beam. Since the size is proportional to the rotation angle, the deformation curve is assumed to be within elastic range.

TRUSS2 shown in Fig. 8(2) shows a deformation curve that bends starting from the position of the upper chord connection at $R = \pm 0.01$ rad, although it is almost the same as TRUSS1 at $R = \pm 0.005$ rad. This is because the plastic deformation of the BR member increased between 0.005 and 0.01rad, and the bending deformation of the whole truss beam concentrated on the upper chord connection, verifying the deformation state similar to Fig. 3. The vertical displacement distribution of TRUSS3 in Fig. 8 (3) showed the same tendency of TRUSS2. Vertical displacement is almost zero from the fixed end to the upper chord connection, however, the displacement occurs at the upper chord connection and increases linearly to the loading point. From the results, the upper chord connection of TRUSS3 is considered to behave as a pin joint.

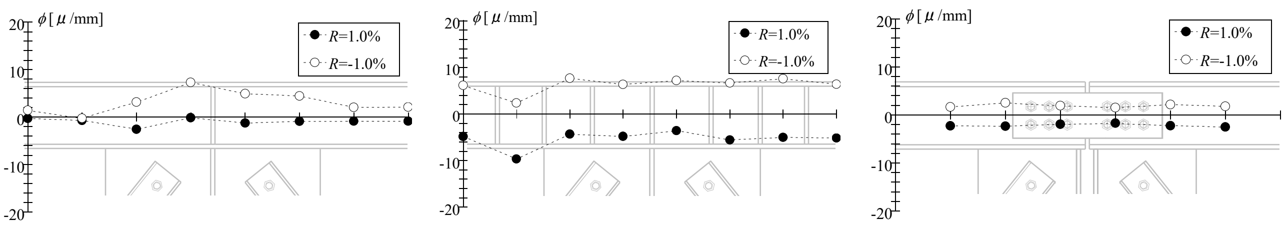


(1) TRUSS1 (2) TRUSS2 (3) TRUSS3

Fig. 8 – Vertical Displacement Distribution of the Chord

3.5 Curvature distribution of the chord around the upper chord connection

The curvature distributions of the chord around the upper chord connection at $R = \pm 0.01$rad for each specimen are shown in Fig.9. The curvatures of each section are calculated by dividing the differences between the strain measured at the upper and lower flanges of the chord section by the depth of chord, and expresses the upper side tension as positive.



(1) TRUSS1 (2) TRUSS2 (3) TRUSS3

Fig. 9 – Curvature Distribution of the Chord around the Upper chord connection

The curvature distribution at R=+0.01rad of TRUSS 1 (marked with ●) in Fig. 9 (1) is almost zero indicating the behavior of truss as axial force member. The curvature at R=-0.01rad (marked with ○) at the center of upper chord connection is slightly larger because of the stress change caused by local buckling of the lower chord. However, the curvatures are within ±5μ/mm and are not significant.

The curvatures of TRUSS 2 in Fig. 9 (2) show relatively constant distribution where the curvatures are about -5μ/mm at R=+0.01rad and about +8μ/mm at R=-0.01rad. It indicates that the upper chord connection is subjected to additional bending moment as plastic deformation concentration on the BR member progress.

The curvatures of TRUSS 3 (Fig. 9 (3)) are small overall and are within ±2μ/mm which is less than a half of curvature of TRUSS 2. The upper chord connection in TRUSS 3 has semi-rigid connection and its rotation stiffness is lower than that of TRUSS 2. As the upper chord connection in TRUSS 3 behaves as pin joint, no significant bending stress occurred after yielding of the buckling restrained member, and curvature distribution is almost same to the behavior in Fig. 8(3).

4. Summary and Conclusions

Based on the experiments of damage-controlled type truss beam using BR member, the conclusions can be summarized as the followings.

(1) TRUSS1 showed strength deterioration due to the global and local buckling of the compression horizontal chord member in R =0.0125rad. The maximum strength of TRUSS1 was 171kN, it was larger than those of TRUSS2 and TRUSS3, however less ductile.

(2) Although the maximum strengths of TRUSS2 and TRUSS3 were about half that of TRUSS1, they showed stable elasto-plastic behavior up to R = 0.02rad. Equivalent plastic deformation capacity of TRUSS2 and TRUSS3 were more than 6 times that of TRUSS1, indicating excellent ductility.

(3) Ultimate state of TRUSS1 was determined due to local buckling and out-of-plane buckling of the upper and lower chord members around the fixed end. On the other hand, TRUSS2 and TRUSS3 using the BR member avoid the buckling in their members. TRUSS2 showed slight plasticity around the BR member, but TRUSS3 showed no damage. TRUSS3, which the upper chord connection behaved as a semi-rigid joint, and it also reduced the damage to the joint.

Seismic performance such as strength and ductility of the proposed truss beam frames can be controlled by designing the BR member. It suggests an excellent seismic performance and restorability that enables continuous use by slight inspection of the BR member even for unexpected large earthquakes.

References

[1] Toshiro SUZUKI, Kikuo IKARASHI, Ichiro IMAI, Tadashi ISHIHARA : A study on structural properties of truss beams with eccentric joints. *Journal of Structural and Construction Engineering (Transactions of Architectural Institute of Japan)*, No.526, 185-192. 1999.12 (in Japanese)

[2] Toshiro SUZUKI, Ichiro IMAI, Tadashi ISHIHARA : A study on structural properties of truss beams with shear panels. *Journal of Structural and Construction Engineering (Transactions of Architectural Institute of Japan)*, No.519, 103-109. 1999.5 (in Japanese)

[3] Motohide TADA : A study on the load carrying capacity of the double-layer space truss incorporating force limiting devices. *Journal of Structural and Construction Engineering (Transactions of Architectural Institute of Japan)*, No.433, 103-112. 1992.3 (in Japanese)

[4] Hidemoto MUKAI, Atsushi WATANABE, Mamoru IWATA, Akira WADA : Experimental study for the small scale truss structures with non-buckling members. *Journal of Structural and Construction Engineering (Transactions of Architectural Institute of Japan)*, No.438, 141-152. 1993.6 (in Japanese)

[5] Morihisa FUJIMITO, Akira TADA, Eiichiro SAEKI, Atsushi WATANABE, Yasuyoshi HITOMI : Study on the unbonded brace detained buckling by the steel pipe concrete. *Journal of Structural Engineering*, Vol.34B, 249-258. 1988.3 (in Japanese)

[6] Shoichi KISHIKI, Satoshi YAMADA, Toru TAKEUCHI, Kazuaki SUZUKI, Ken OKADA, Akira WADA : New ductile steel frames limiting damage to connection elements at bottom flange of beam-ends. *Journal of Structural and Construction Engineering (Transactions of Architectural Institute of Japan)*, No.575, 113-120. 2004.1 (in Japanese)

[7] Satoshi YAMADA, Kyung-Soo CHUNG, Shoichi KISHIKI, : Relationships of damage between steel member and material subjected tobcyclic loading. *Journal of Structural and Construction Engineering (Transactions of Architectural Institute of Japan)*, No.603, 139-146. 2006.5 (in Japanese)