Passive Vibration-Control Concept For Truss Frame Structures

T.Takeuchi[1)]

1) Associate Professor, Dept. of Architecture & Building Engineering , Tokyo Institute of Technology, Japan
ttoru@arch.titech.ac.jp

Abstract: Truss frame generally had to be designed elastically even against large seismic force, because of fragile characteristics led by member buckling. In this paper, damage tolerant design for truss frame structures using energy dissipation members in critical positions are discussed. Detailed designs for high-rise rack warehouses and communication towers are studied and tested, followed by simple method for evaluating the effect of these members.

1. INTRODUCTION

In seismic area, plastic design with moment frames have been popular for long time, with the concept of ductile characteristics led by plastic-hinges on bending beam ends. On the other hand, truss frame structures, popularly used for long-span roofs or industrial facilities, had to be designed elastically against design loads, because of less-ductile characteristics due to the axial member's buckling. However, designing truss structures elastically against large earthquake is not economic, not elegant in design, and remaining risks for fragile collapse in the event of seismic level exceeding the design criteria.

Recently, passively controlled buildings have become popular in Japan, and various types of passive energy dissipation devices are put in practical use. Many of them are incorporated within moment frames, achieving damage tolerant design, which keeps main structures in elastic even with great earthquake. The same concept can be applied for truss frame structures by incorporate energy dissipation members in critical positions, plasticize them firstly while other truss members are kept before buckling, and control entire structure ductile (Fig.1). This design concept will enable truss frame structures to be slender, elegant, economic and safe from buckling even in heavy seismic areas.

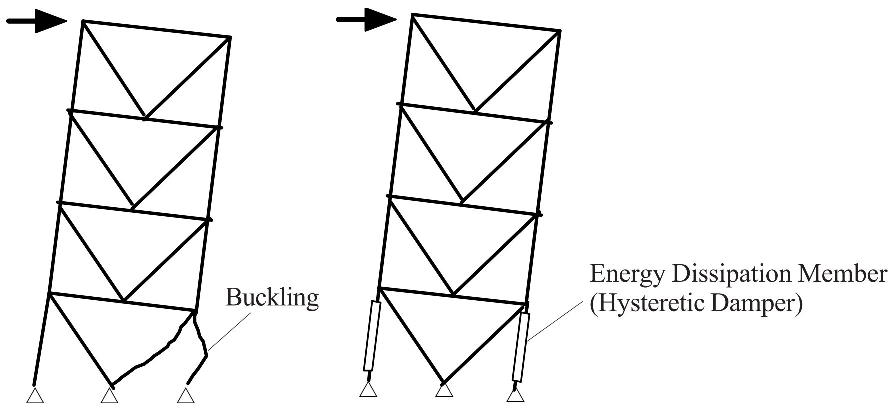


Figure 1. Damage Tolerant Concept for Truss Frame

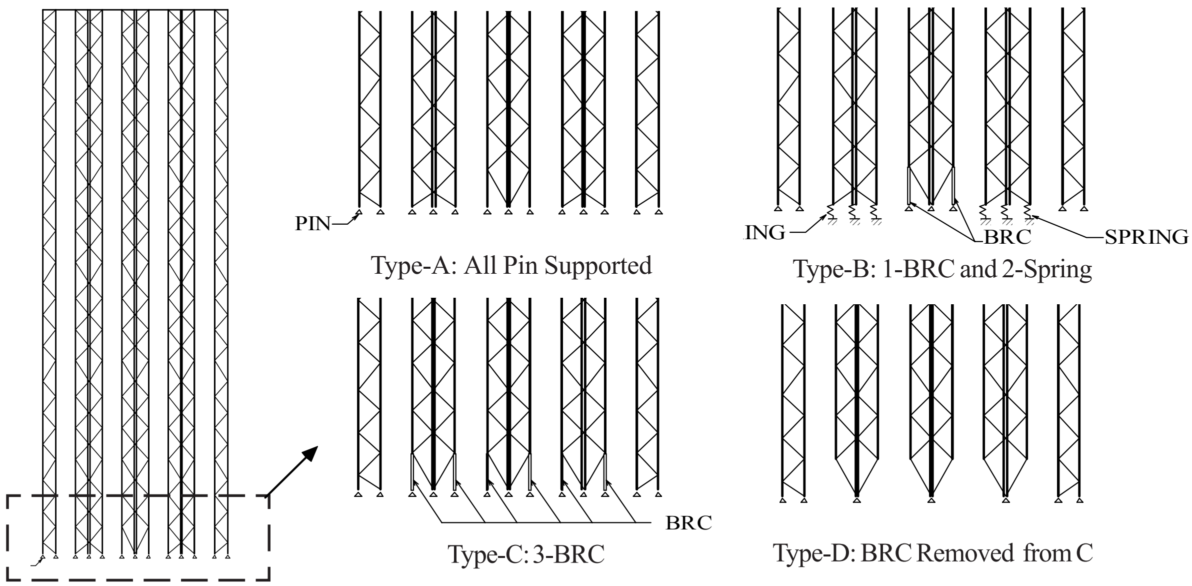


Figure 2. Rack Structure and Supporting Options

In this paper, above design concept is applied to practical design of high-rise automatic rack warehouse and communication tower using hysteretic energy-dissipation members, introducing their details and effects.

2. APPLICATION FOR RACK WAREHOUSES

The high-rise automatic rack warehouse are composed of 52m high, 5 lines of 1.3m-2.6m wide trussed structures connected each other at their top by horizontal beams (Fig.2). Each line of trusses is independent between the ground level and the top to keep spaces for vertical cranes, the aspect ratios of each set of trusses reaches 20 to 40. As diagonal braces stiffen the frames in longitudinal direction, this transverse direction is critical in strength and deflections. Walls are attached on the side trusses and roofs are on top beams, the rack structure itself composes the warehouse frame. The critical members against horizontal forces are the base-chords of the trusses, and buckling of these members lead to the collapse of entire structure. For the sake of ductile design, these chords are replaced by Buckling-Restrained Columns (BRC; e.g. Unbonded Braces), which perform as axial elasto-plastic dampers.

In this study, four types of truss structures as shown in Figure 2 are studied.

Type-A: Normal truss structure whose bases are all pin-supported.

Type-B: Side base chords of the center truss are replaced by BRC, while other chords are supported by pin or spring to concentrate reaction forces into BRC.

Type-C: Side chords of all trusses except for side trusses are replaced by BRC.

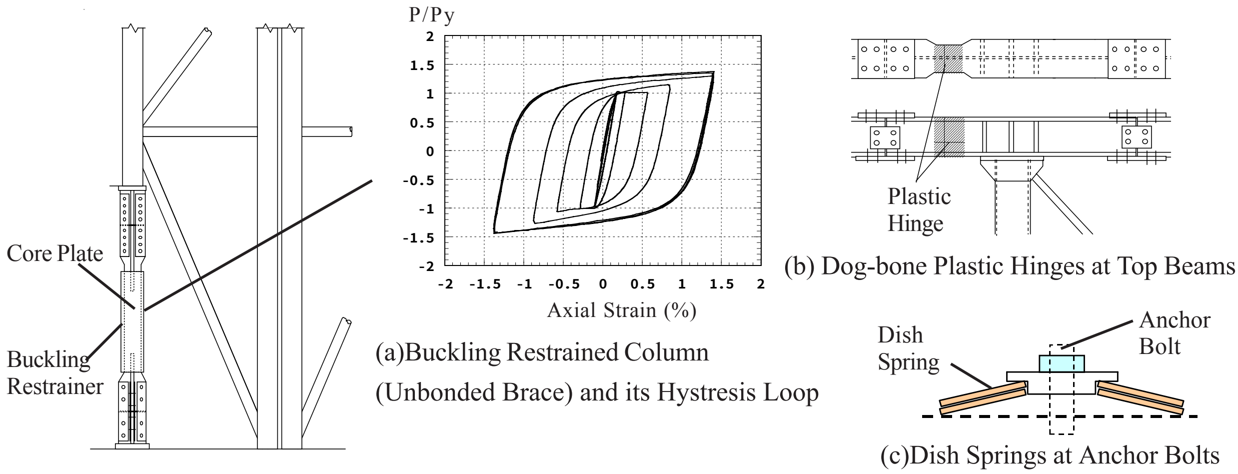


Figure 3 Details of the Rack Structure

Type-D: All BRC are removed from Type-C.

In Type-B and Type-C, axial yielding of BRC and plastic hinges of the top beams decide collapsing mechanism of the entire structure. The side trusses are kept in elastic while BRC are going into plastic, and pull back the structure from residual deformation after earthquake being finished.

Detailed design of each part is shown in Figure 3. Detail of BRC at bases are in Figure 3(a). The base-side chords of the truss are replaced by Unbonded Braces, which are composed of core plates and concrete-filled tube restrainer (Fujimoto et.al. 1990). Because of restrainer keep core-plate from buckling even in plastic zone, they show symmetrical hysteretic loops and excellent characteristics as energy dissipation members. They are connected to the truss members with bolts, which enables replacement of BRC after earthquake.

The top beams have dog-bone shape at truss sides, which realizes stable plastic hinges (Fig.3(b)). For spring supports in Type-B frame, dish springs are inserted for anchor bolts at column bases (Fig.3(c)).

To verify the performance of these structures against large seismic vibrations, time-history analyses are carried out on each types of structure. The analytical models are two-dimensional composed of each member as beam element, considering axial forces and bending moments. For seismic vibrations, El Centro NS, Taft EW, Hachinohe NS, and Artificial wave using site ground conditions are used for design loads with the level of 50 cm/sec in maximum velocity. The maximum responses for Type-A, Type-B and Type-C are shown in Figure 4. In shear forces, additional forces of each line of trusses are shown. In story drift, horizontal displacement of the side trusses, which are the most critical in each truss, are represented.

The ultimate capacity of the frame is about 450 kN at base, and maximum response at Type-A frame exceeds the ultimate capacity. This means Type-A frame will collapse with foresaid seismic vibration caused by the buckling of base columns. On the other hand, maximum story drift is about 42mm in 3365mm unit

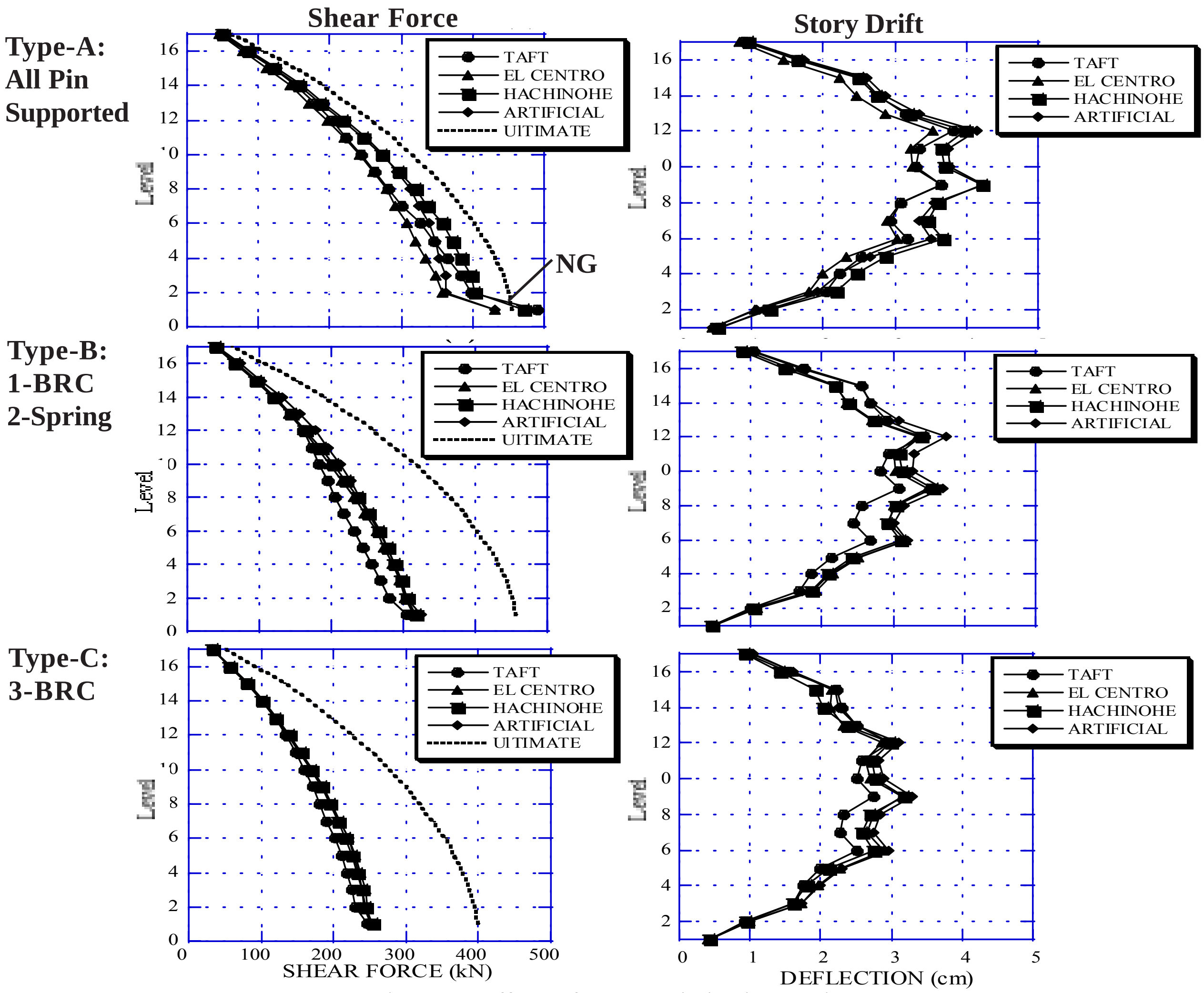


Figure 4 Effects of Energy Dissipation Devices

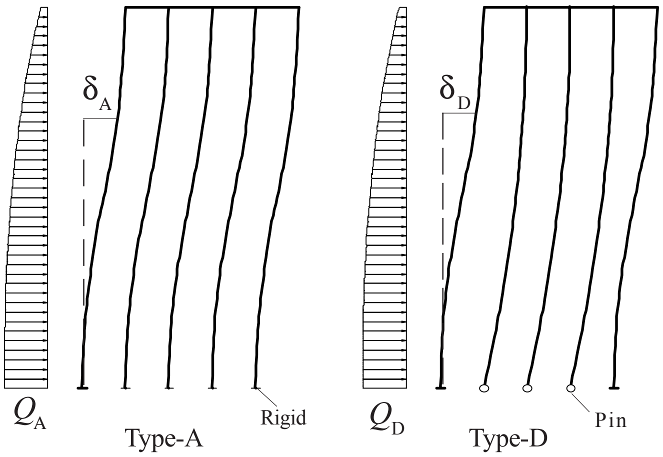


Figure 5 Models for Stiffness Evaluation

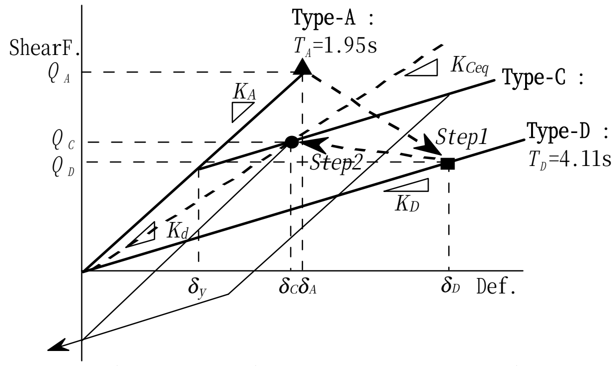


Figure 6 Maximum Response Conversion

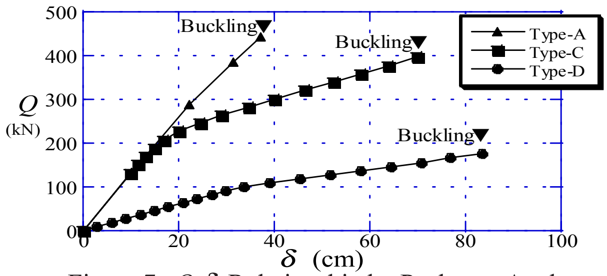


Figure 7 *Q*-δ Relationship by Pushover Analyses

height, which is acceptable for the limit of 1/60 in drift angle.

In type-B, maximum shear forces are reduced to about 75% of Type-A, which secure some margin in strength. Maximum displacement in each story is slightly reduced to about 90%.

In Type-C, the reduction effect of BRC is more significant, and the maximum shear forces are reduced to about 55% of Type-A. Maximum displacement is reduced to about 80%.

Consequently, introduction of BRC is effective to reduce maximum shear forces and avoid local buckling at base chord members. The more BRC are introduced, the more reduction effect is obtained. Although reductions in deflection are not significant, they are also contributes to decrease the deflection of the entire structure.

3. ESTIMATION OF RESPONSE REDUCTION EFFECTS

In this section, a simple method for estimating the maximum response of the frames with energy dissipation members, using linearization techniques are proposed. When each set of truss is modeled as an equivalent fragile single member, a end of truss with energy dissipation elements on side chords can be modeled as a plastic hinge in the equivalent single member. Then the entire truss frames can be modeled as equivalent beam-column frames with plastic hinges as Figure 5. Simplified response estimation methods developed for moment frames with dampers (Kasai et.al. 1998) can be applied to truss frames, which enable easy structural design avoiding try-and-error basis study in preliminary stages. In this section, response of Type-C frame is estimated using stiffness of Type-A and D frames.

When each type of frames are plotted by horizontal forces (Q) and displacement (δ), their relationship can be led as shown in Figure 6. Where, Q is the shear force at the base, and δ is the equivalent displacement defined as following.

$$\delta = \frac{\sum_{i=1}^{N} m_i \delta_i^{\,2}}{\sum_{i=1}^{N} m_i \delta_i} \tag{1}$$

In these frames, δ is represented by the displacement at the height of 68-75% of the total height.

In Fig.6, Type-D has much lower stiffness than Type-A, and Type-C is considered to have the same stiffness as Type-A while BRC is elastic, then transferred to the same stiffness as Type-D after BRC yielding. Push-over analyses in Figure 7 proves this assumption. In this condition, the equivalent stiffness ratio of Type-C/Type-D is the function of $\mu=\delta_c/\delta_y$, where δ_c is the maximum deflection of Type-C, and δ_y is the deflection where BRC start yielding, expressed as following.

$$K_{Ceq}/K_D = 1 + \frac{K_d}{K_D \mu} = 1 + \frac{K_A/K_D - 1}{\mu} \tag{2}$$

The equivalent damping ratio can be estimated as the following equation by taking the average of integral to the maximum amplitude (Kasai et.al 1998).

$$h_{\mathrm{Ceq}} = h_0 + \frac{2(1+K_{\mathrm{d}}/K_D)}{\pi\mu} \ln \frac{\mu + K_{\mathrm{d}}/K_D}{(1+K_{\mathrm{d}}/K_D)\mu^{\frac{1}{1+K_{\mathrm{d}}/K_D}}} = h_0 + \frac{2K_{\mathrm{A}}/K_D}{\pi\mu} \ln \frac{\mu + K_{\mathrm{A}}/K_D - 1}{(K_{\mathrm{A}}/K_D)\mu^{\frac{1}{K_{\mathrm{A}}/K_D}}} \tag{3}$$

where, T_A: own period of Type-A, T_D: own period of Type-D. Consequently, to estimate the maximum response of Type-C, the following two steps are required.

1) Response of Type-A (▲ in Fig.6) is estimated by response spectrums. Where T_A>0.6sec, the response of Type-D (■ in Fig.6) is estimated by Q being multiplied by (T_A/T_D), and δ being multiplied by (T_D/T_A). (*Step1*)
2) Response of Type-C (● in Fig.7) is estimated with the effect of K_{Ceq}/K_D and h_{Ceq}. (*Step2*).

In detail, the response of Type-C/Type-A is expressed by the following equations.

Shear force and displacement response ratio is, respectively :

$$a_C/a_A = D_{\mathrm{h}}\left(T_A/T_D\right)\sqrt{\left(K_{C\mathrm{eq}}/K_D\right)} \qquad \delta_C/\delta_A = D_{\mathrm{h}}\left(T_D/T_A\right)\sqrt{\left(K_D/K_{C\mathrm{eq}}\right)} \tag{4, 5}$$

where, $$D_h = \sqrt{\frac{1+25h_0}{1+25h_{Ceq}}} \qquad T_{A,D} = 2\pi\sqrt{\frac{M_{A,D}\delta_{A,D}}{Q_{A,D}}} \qquad M = \frac{\left(\sum_{i=1}^{N} m_i\delta_i\right)^2}{\sum_{i=1}^{N} m_i\delta_i^{\,2}} \tag{7, 8, 9}$$

The evaluated responses by these equations are calculated in spread sheets, and maximum shear forces and story drift led by this results are plotted with time-history analyses in Figure 8. In these figures, dotted line is evaluated response without dampers (Type-A), and solid line is response with dampers (Type-C), both led by the proposed method, and marks are the results of time-history analyses.. From these comparison, the results of proposed evaluation method is considered to be consistent with the results of time-history analyses.

In Fig.7, the critical deflections of Type-A,C, D frames are 35cm, 87cm, and 70cm respectively, and the rate of safety in each structure estimated by proposed evaluation is 1.1 in Type-A, 1.2 in Type-D, and 2.3 in Type-C. This means the frame with almost collapse limit (Type-A) is improved in performance to more than double-safe frame just by replacing base chords to BRC (Type-C).

From these studies, proposed evaluation method is considered to be effective for deciding the suitable volume and position of energy dissipation members in preliminary design stages. Advantage of this method includes that no elasto-plastic analyses or time-history analyses are required, and suitable amount of energy dissipation members is directly given without try-and-error analyses.

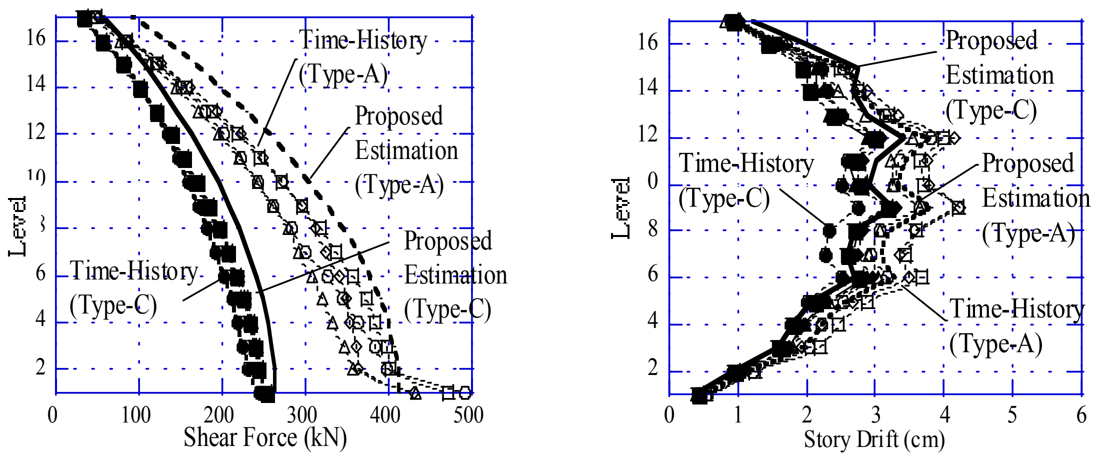
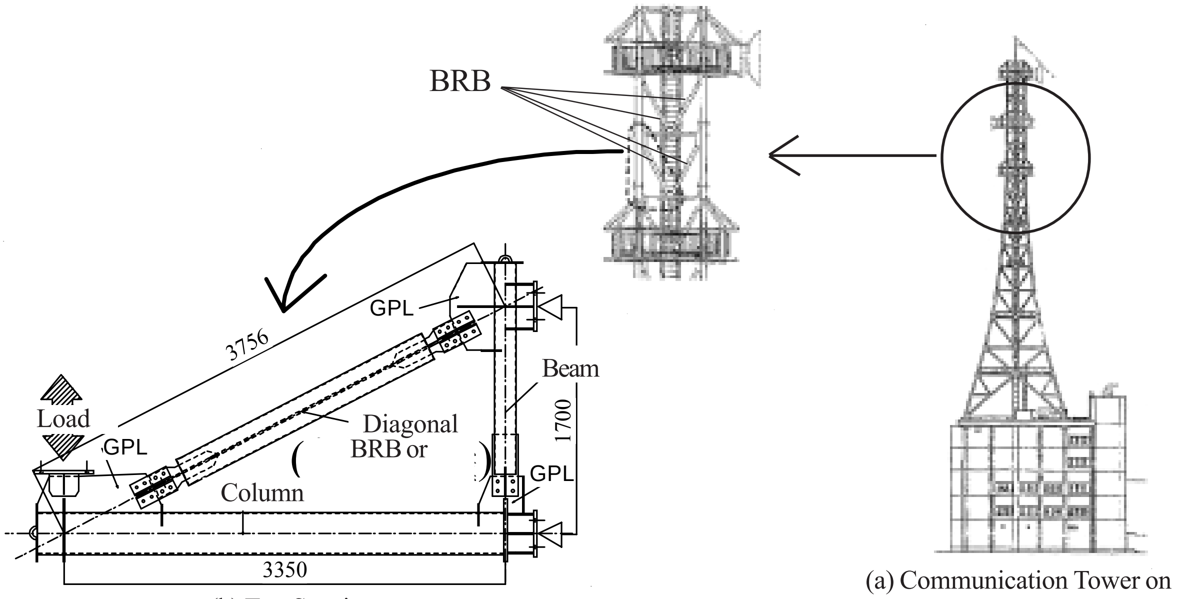


Figure 8 Comparison of Proposed Estimation and Time-History Analyses

4. APPLICATION FOR COMMUNICATION TOWERS

Instead of replacing column, replacing diagonal members to buckling restrained members is another way to reduce seismic responses. In this section, application for communication towers are discussed and tested. The concept of structural system is shown in Fig.9, which is planned as seismic refinement. The towers are usually designed against wind forces, however, seismic response accelerated by standing buildings can be exceed the wind forces. In this study, critical diagonal members are replaced by Buckling Restrained Braces (BRB), aims at saving all other members from buckling.



(b) Test Specimen

(a) Communication Tower on Building

Figure 9 Application for Communication Towers

To verify the reality of energy dissipation mechanism, three types of real-size mock-up tests are carried -out using single truss elements consists of a chord, a beam, and a diagonal member. In the existing tower, all truss members have pipe (CHS) sections (Type-TO). In strengthen reinforcement option, concrete is in-filled within the diagonal member (Type-TC). In BRB reinforcement option, diagonals are replaced by BRB whose yield strength are meeting with the buckling strength of original pipe diagonals (Type-TA). Each truss is cyclically loaded up to 1/25 story drift . The configuration of the test is shown in Fig.10, the detail of BRB in Fig.11, and loading program is shown in Fig.12.

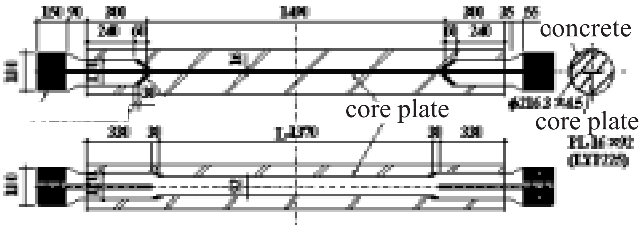


Figure 11 Detail of BRB

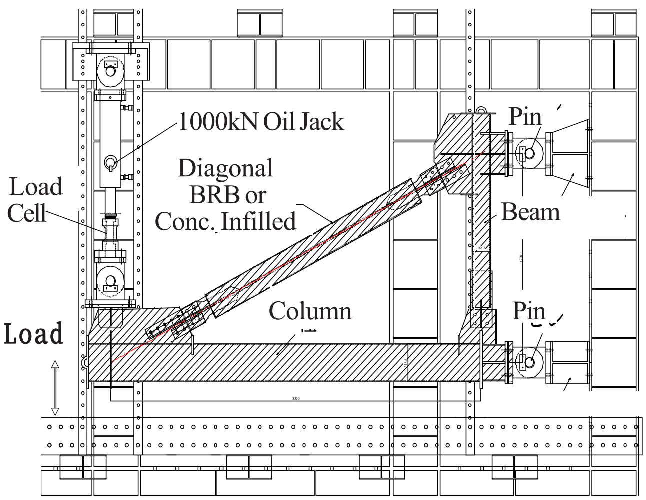


Figure 10 Test Configuration

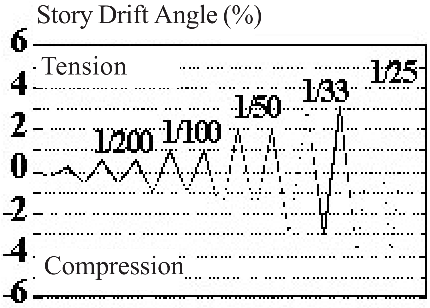


Figure 12 Loading Program

The test results for Type-TO, Type-TC, Type-TA are shown in Fig,13, 14 and 15, respectively. In each figure, (a) are load-relationship of truss structures, (b) are axial force-deformation of diagonal members, (c) are figures at the end of tests.

For Type-TO, story drift up to 1/100 is accepted by slip of connection bolts, then the diagonal started buckling at 1st 1/50 cycle in compression. elbow was created at 2nd 1/50 cycle in compression, and torn off at 1st 1/25 cycle in tension. The maximum axial force to start buckling was about 800kN.

For Type-TC, buckling strength was increased more than 1000kN, the gusset plate at diagonal-to-beam connection was buckled to out-of-plane direction. From this result, it is found that only strengthen diagonal member will cause collapse in other parts as connections.

For Type-TA, BRB keeps its maximum force between 400 kN and 600kN, and shows quite stable and symmetrical hysteresis loop up to 1/25 story drift both in Fig.15(a) and 15(b). After 3rd 1/25 cycles, BRB fractured at its core plate. Their cumulative plastic deformation and dissipated energy until its fracture are shown in Fig.16 and Fig.17, respectively. The cumulative equivalent strain (axial deformation / length) capacity in BRB diagonal (Type-TA) was about 5.5 times of normal pipe tests (Type-TO). The dissipated energy until fracture is also has the same tendency.

From these results, replacement of diagonals to BRB is considered to have the best performance as seismic refinement and saving other members.

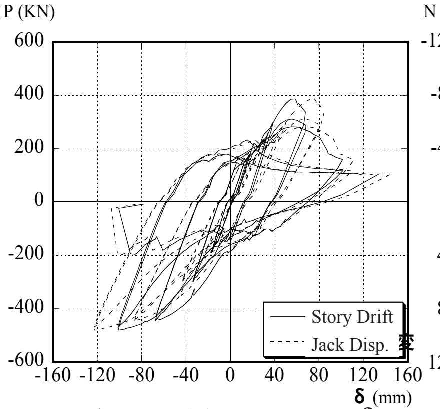


Figure 13(a) Type-TO P-δ

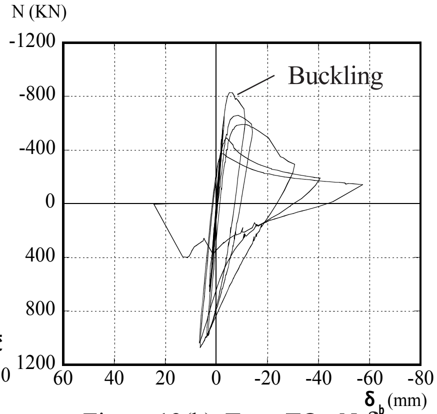


Figure 13(b) Type-TO N-δ_B

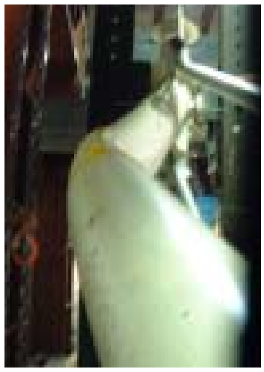


Figure 13(c) Diagonal buckling

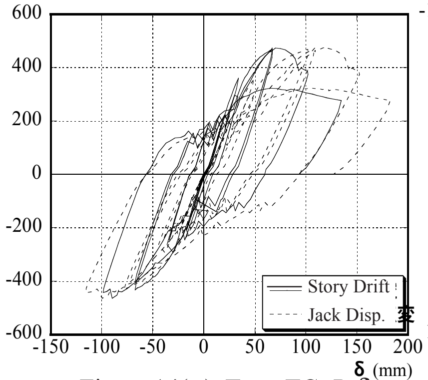


Figure 14(a) Type-TC P-δ

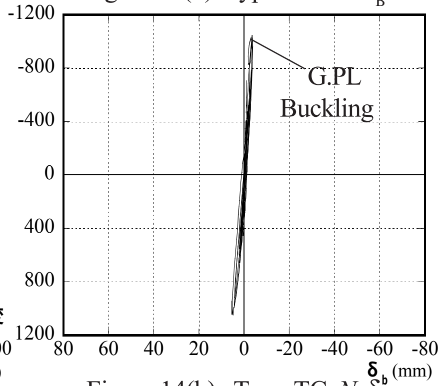


Figure 14(b) Type-TC N-δ_B

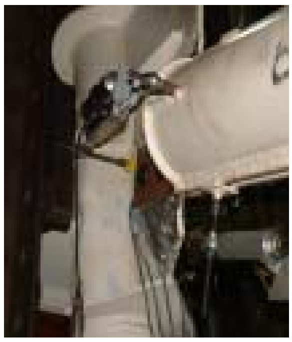


Figure 14(c) G.PL buckling

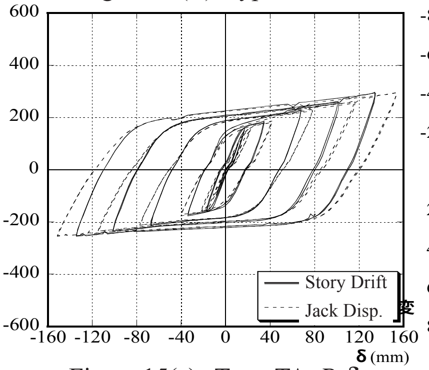


Figure 15(a) Type-TA P-δ

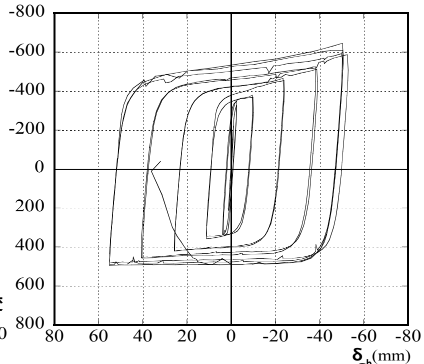


Figure 15(b) Type-TA N-δ_B


Figure 15(c) Diagonal at Test End

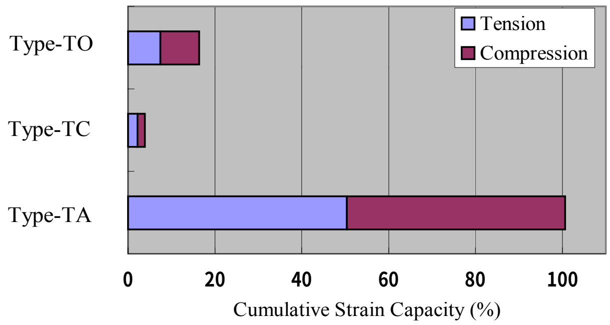


Figure 16 Cumulative Equivalent Strain Capacity until Fracture

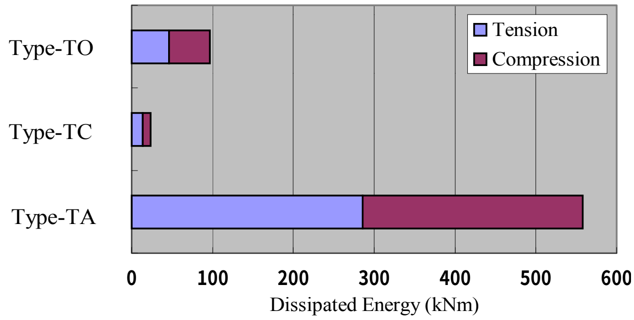


Figure 17 Dissipated Energy until Fracture

5. CONCLUSIONS

Concept of damage-tolerant design for truss frames with elasto-plastic energy dissipation members are introduced, and applications for high-rise automatic-rack warehouse and communication towers are described. Using linearization techniques, the simple method for evaluating maximum response of rack warehouse structures against large earthquake is proposed, and compared with time-history analyses. Also cyclic loading tests for the truss for communication towers are carried out with real-size mock-ups. By these studies, the following points become clear.

1) In truss frames, incorporating energy dissipation members (e.g. buckling restrained members) into critical positions in truss frames are effective to improve the performance of the rack warehouse structure against large earthquake. In the case of 52m high rack warehouse structures, replacing 2/13 base chords to BRC reduces the maximum shear forces to 75%, and replacing 6/13 base chords to BRC reduces the maximum shear forces to 55%.

2) The above reduction effects evaluated by proposed evaluation method are consistent with time-history analyses, and effective to predict maximum responses. For the proposed method requires only elastic analyses, it is useful for deciding the volume of energy-dissipation members in preliminary design stages.

3) From the real size cyclic-loading tests modeling communication towers, replacement of diagonals to BRB is considered to have the best performance as energy dissipation system. strengthening diagonal pipes by infilling concrete will increase the buckling strength, however, other members or connections might be critical instead. BRB frame has stable hysteresis loop, and has over 5 times of cumulative strain than normal pipe diagonal.

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