Paper:

Recent Design Approaches for Passively Controlled Structures

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This study reviews and discusses several recent design approaches for passively controlled structures. First, an optimized arrangement method for the energy dissipation members (EDMs) of various infrastructures is introduced. Next, seismic isolation and passive control techniques for freeform space structures are discussed. Finally, research on various spine-frame concepts with EDMs is reviewed. All these approaches are being introduced to actual structural design, and recent examples are reported here.

Keywords: passive control, seismic isolation, spatial structures, spine frame, rocking frame

1. Introduction

Passively controlled structures and seismically isolated buildings are gaining prevalence in seismic design. In Japan, more than 3000 buildings are seismically isolated, and over 1000 buildings utilize energy dissipating members (EDMs) such as dampers or fuses. Although the understanding of passive control has matured, many unsolved issues remain, and recent concepts are under investigation for application in design methodologies. This study introduces three new approaches for the design of passively controlled structures: the optimized arrangement method for EDMs in various infrastructures, the application of seismic isolation and response control to freeform space structures, and various spine-frame concepts with EDMs. All these approaches are being introduced to actual structures, and recent examples are reviewed.

2. Damper Arrangement Optimization

With the expanding application of passive control techniques to various structures, arrangement methodologies for EDMs in non-ordinary multistory structures are under investigation. The application of EDMs in truss towers for communication facilities are discussed here. Communication towers owned by electric companies have recently been recognized as prone to failure against large seismic forces; therefore, seismic retrofitting is being implemented. However, conventional strength-based retrofitting procedure is not feasible because the other construction components or connections may fail after the weak members are reinforced, which would necessitate the reinforcement of all members. A seismic retrofitting method has been proposed in which the critical truss members are replaced with EDMs [1]. This approach was implemented after the efficiency of the method was confirmed both analytically and experimentally, and approximately 20 communication towers have been retrofitted with this method.

However, determining which members should be replaced with EDMs remains unclear. Two empirical methods are often employed in practice:

(*Method A*) Arrange an EDM to connect two points with relatively large displacements.

(*Method B*) Perform pushover analysis and replace the buckling members with EDMs.

Recently, various optimization techniques have been applied to examine EDM arrangement [2]. In such studies, optimization techniques based on genetic algorithms (GA) are used to optimize the arrangement of EDMs in communication towers under various simulated seismic inputs to minimize the deflection response or risk of buckling. However, performing a time history response analysis for each damper arrangement is time-consuming and not feasible for practical design. Takeuchi et al. [3] proposed a method in which the response of a structure is directly estimated from the response spectrum taken from the equivalent natural periods and damping factors of the structure (**Fig. 1**). To evaluate the equivalent damping factors ${}_m h_{eq}$ from EDMs distributed on each floor, the following equation is used for multistory structures in the m^{th} vibration mode:

$$ {}_m h_{eq} = \frac{\sum_{i=1}^{N} {}_m h_{eqi} \cdot W_i}{\sum_{i=1}^{N} W_i} = h_0 + \frac{1}{4\pi} \frac{\sum_{i=1}^{N} {}_m E_{di}}{\sum_{i=1}^{N} W_i} \quad . \; . \; . \quad (1) $$

where W_i is the elastic strain energy in the i^{th} floor, and ${}_m E_{di}$ is hysteretic energy dissipated by EDMs in the i^{th} floor for m^{th} mode. In truss structures, where EDMs and the surrounding elastic members are connected in series, the following equation is used instead to evaluate the equivalent damping, as in **Fig. 2**.

$$ {}_m h_{eq} = h_0 + \frac{1}{4\pi} \frac{\sum_{i=1}^{N} \left\{ {}_m E_{di} / \left(1 + {}_m \eta_{di}^2\right) \right\}}{\sum_{i=1}^{N} {}_m W_{bi} + {}_i \sum_{i=1}^{N} \left\{ {}_m W_{di} / \left(1 + {}_m \eta_{di}^2\right) \right\}_i} \quad (2) $$

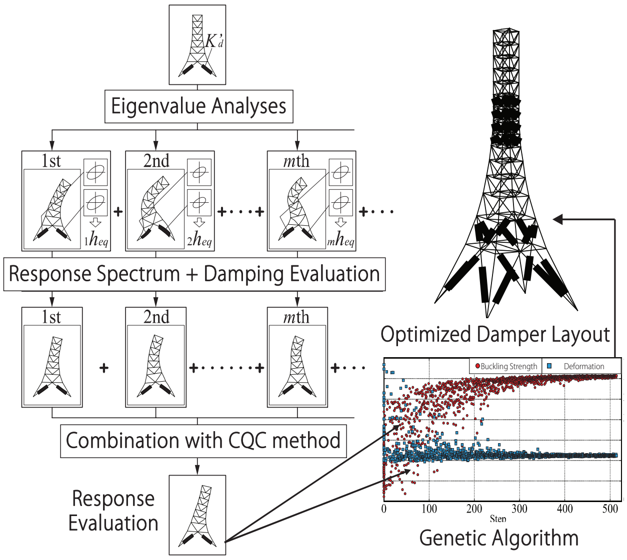


Fig. 1. Optimization process of EDM arrangement.

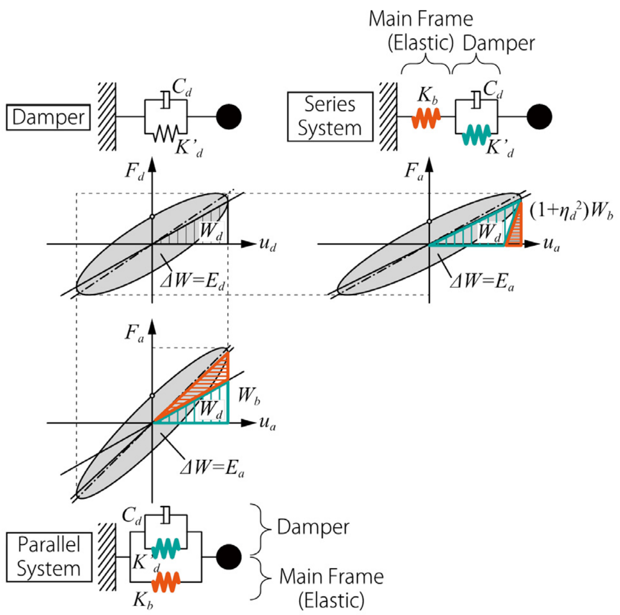


Fig. 2. Evaluation of equivalent damping.

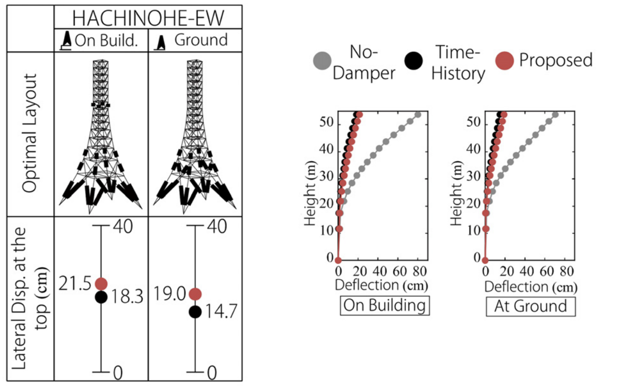


Fig. 3. Optimized EDM arrangement including chord members.

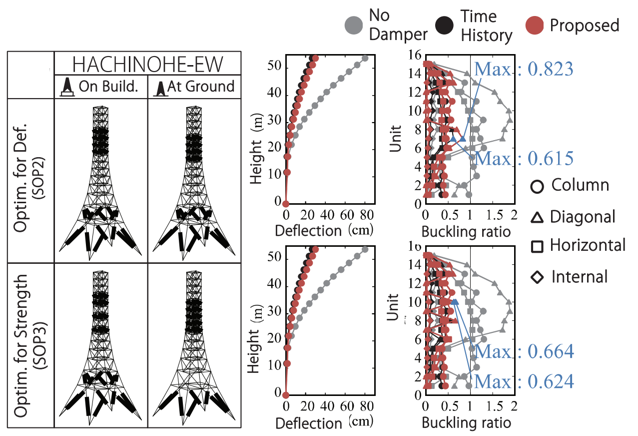


Fig. 4. Optimized EDM arrangement excluding chord members.

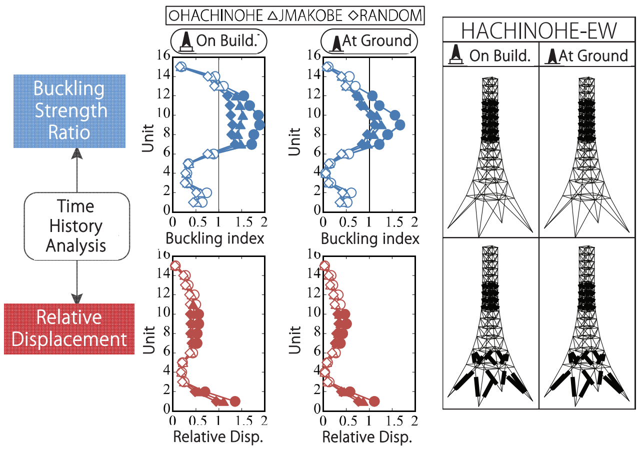


Fig. 5. Comparison against empirical methods.

The response of a structure with a certain EDM arrangement is estimated from the response spectrum with the natural periods of major vibration modes taken from the results of eigenvalue analysis, and the equivalent damping is determined using Eq. (2). Analyses were performed for towers on a supporting building structure or directly on the ground, and the optimized EDM arrangement was analyzed with a simple GA (tournament method, uniform cross) against the expected Tokai seismic input. **Fig. 3** shows the optimal solution in which the replacement of any member, including the chords, with 24 viscoelastic EDMs is allowed (SOP1). The maximum response displacement distribution is shown on the right. The maximum response displacement without damping exceeded 70 cm at the top of the structure; it decreased to around 20 cm in the structure with the optimal arrangement of EDMs. Thus, the results of the time history response analysis and the proposed method showed good agreement. **Fig. 4** shows the results when member replacement was limited to diagonal members with the objective of either minimizing displacement at the top (SOP2) or maximizing the buckling strength margin (SOP3). In both cases, arrangements of EDM substituents at the base and at $\sim 3/4$ of the total height of the structure were found to be optimal. The maximum displacement and buckling strength margin were compared, and the results of the time history response anal-

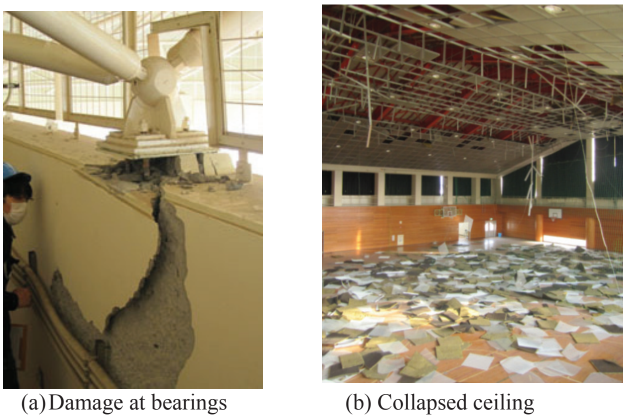


Fig. 6. Typical earthquake damage in spatial structures.

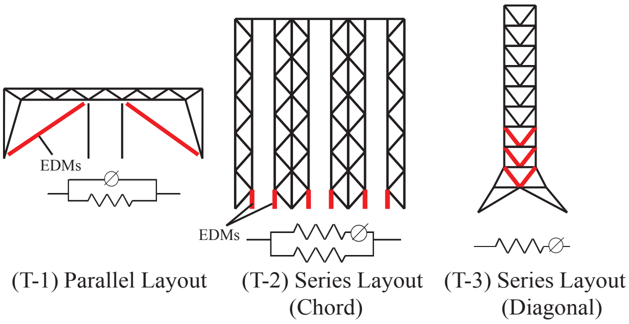


Fig. 7. Energy dissipation for truss structures.

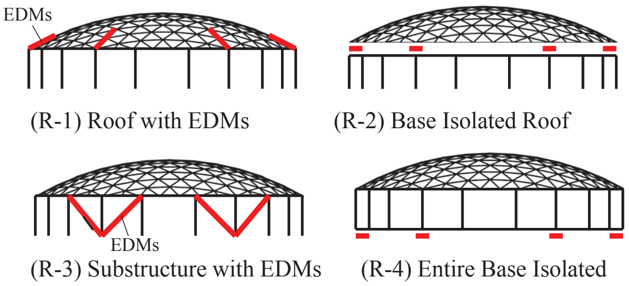


Fig. 8. Energy dissipation for latticed roofs.

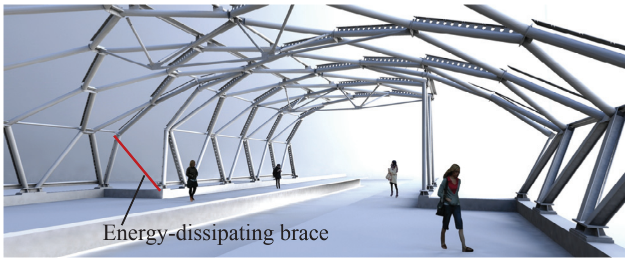


Fig. 9. Grid roof structure for train station with EDMs.

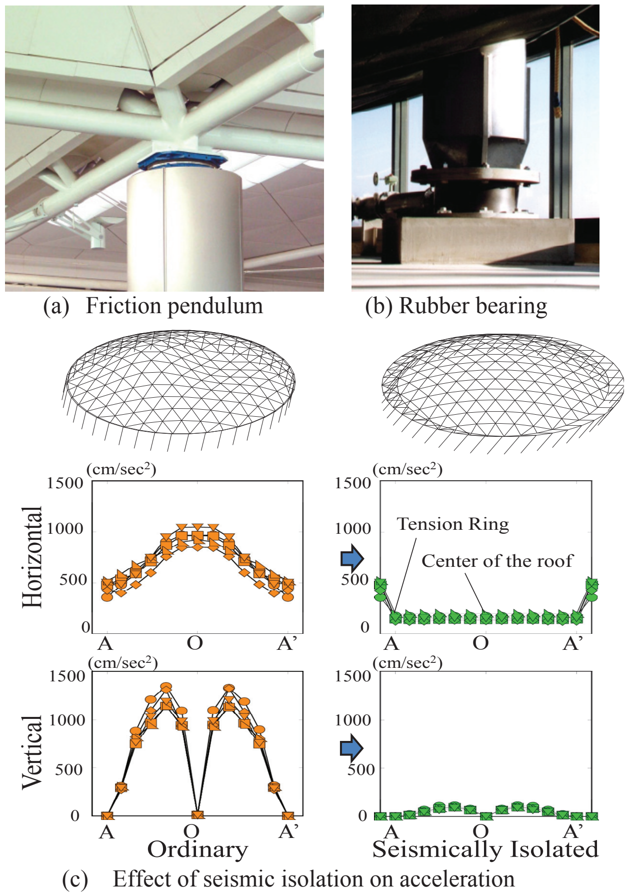


Fig. 10. Energy-dissipating roof bearings.

ysis and proposed method again showed good agreement. **Fig. 5** compares the above empirical approaches with the optimization analyses. While replacing buckling members (*Method B*) produced only an upper arrangement without base zones, connecting larger displacement joints (*Method A*) produced results relatively similar to those of the optimization analyses. In the near future, such optimization techniques will be applied more widely in the preliminary designs of passively controlled structures.

3. Response Control for Free-Formed Spatial Structures

The 2011 Tohoku earthquake seriously damaged many spatial structures such as gymnasia and indoor stadiums [4]. Typical damage involved buckling and fracture of the vertical and roof braces, as well as damage to the roof bearings (**Fig. 6(a)**) and non-structural members, such as ceilings (**Fig. 6(b)**). Vertical excitation of the roof structures and out-of-plane responses of cantilevered reinforced concrete (RC) walls are believed to have caused such damage [5]. Energy dissipation concepts were investigated to prevent damage of this scope to various space structures. **Figs. 7** and **8** show application patterns for truss and latticed roof structures, respectively [6]. The use of these methods has increased recently. In particular, Buckling-Restrained Braces (BRBs) have been introduced into truss structures such as (T-1)–(T-3), and seismic isolation has been introduced for latticed roofs such as (R-2). **Fig. 9** shows a practical example of (R-3). The grid shell roof for a train station was placed on a heavy concrete frame, and excitations of the supporting structure were expected to cause the response of the roof in

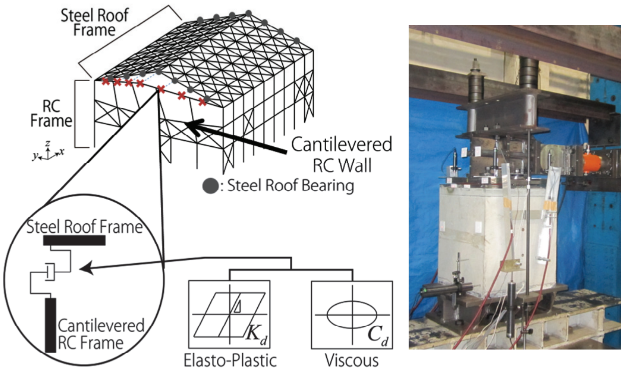


Fig. 11. Damper bearing controlling cantilevered RC walls.

Fig. 12. Free-formed structures.

the transverse direction. Therefore, small EDMs were inserted at the edges of the structure to control the natural period and add damping. In an analytical study, the maximum story drift angle of 3% was reduced to 0.3% by the addition of two energy-dissipating braces.

As examples of (R-2), introducing isolation bearings (**Fig. 10**) are also gaining popularity as components of spatial structures. Seismic isolation reduces the response accelerations on a roof in both horizontal and vertical directions, as shown in **Fig. 10(c)** [7]. Friction damper bearings also reduce the response of cantilevered RC walls supporting steel roof structures (**Fig. 11**, [8]), which aids in preventing damage, as shown in **Fig. 6(a)**.

In such space structures, aesthetic and functional design requirements often lead to the use of freeform and irregular shapes. Such freeform roof structures have recently been produced and designed through the use of various optimization algorithms, as shown in **Fig. 12**. Form-finding against the seismic response has also been investigated [9]. The EDM distribution for such freeform structures should also be considered with the methods using the optimization algorithm.

4. Concepts of Controlled-Spine Frames

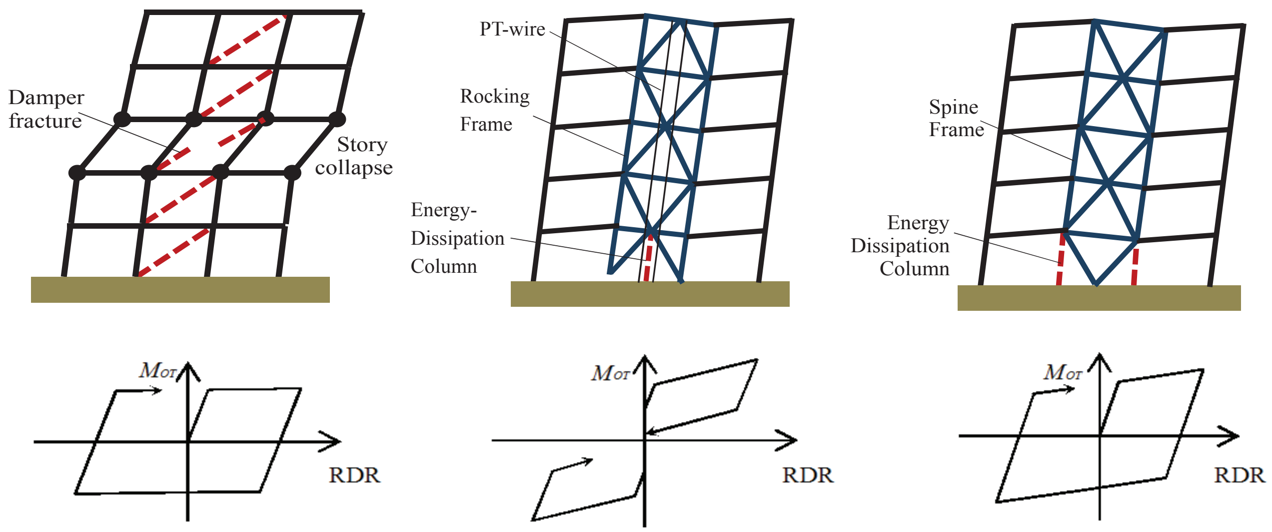
Although distributed EDMs in a multistory structure are effective in proofing the structure against seismic shocks, some risk remains of EDM fracture under more severe earthquakes. The introduction of spine frames throughout the stories is one effective means of minimizing these risks. **Fig. 13** shows three structural systems and the relationships between the overturning moment (M_{OT}) and roof drift ratio (RDR) for each system. A conventional frame with shear dampers, such as BRBs (**Fig. 13(a)**), generally shows excellent stability as long as the main structure is well balanced regarding stiffness and remains elastic. However, when a structure with irregularities in stiffness and strength is subjected to a large earthquake, damage is concentrated in the weak stories, and residual deformations occur. To decrease the risk of such failures, a controlled uplifting rocking frame system (**Fig. 13(b)**) has been investigated. The performance of the system was confirmed through actual-size shaking-table tests [10]. The system has been also applied in several actual construction projects. In the tests, a rocking spine frame was utilized to distribute damage uniformly throughout the stories, and post-tensioning (PT) strands were introduced to achieve self-centering (**Fig. 13(b)**). The pre-stressed forces of the PT strands must exceed the expected residual forces of the energy dissipation columns (BRCs) to provide self-centering performance; the pre-stresses can reach several thousand kilonewtons in practice. In addition, the column-base detail must allow an uplifting action. To avoid these requirements, a non-lift-up spine-frame system (**Fig. 13(c)**) can be a viable option.

The non-lift-up system comprises steel-braced frames and replaceable energy dissipation columns (BRCs) without PT strands. Self-centering is achieved with an envelope elastic-moment frame. Unlike the lift-up system, the non-lift-up system is supported with BRCs that are fixed to the foundation.

For earthquakes exceeding specific levels, plastic hinges activate at the bottom of the BRC, and the braced rocking frame rocks around the center. During rocking, the rocking frames remain elastic, while energy is dissipated by the plastic deformation of replaceable EDMs. After the shaking, restoring forces from the envelope moment-resisting frames aid in system centering. The rocking braced frame acts as the spinal element of the entire structure to prevent deformation concentration, even when the envelope frame includes weak stories. With proper design, the self-centering capacity can ensure immediate occupancy of the building.

The proposed structural system was applied to the construction of the MCES building on the Suzukakedai campus of Tokyo Tech. **Figs. 14** and **15** depict the perspective view and structural system of the building, respectively. Most stories of the building are 4 m in height; the first story is 4.2 m. The plan dimensions are 27 m × 27 m, with 4.5 m × 4.5 m bays for the perimeter frame and 9.0 m × 9.0 m bays for the internal frames. The spine frame is located at the center of the building. To evaluate the performance of the proposed structural system, models of the same envelope steel moment-resisting frame (SMRF) using the shear damper system, lift-up system, and proposed non-lift-up system were designed with the same configuration and earthquake inputs [11].

Figure 16 shows the maximum story drift and residual drift ratio of each story in the three models obtained by time-history analyses. The shear damper model tends



(a) Shear damper system (b) Lift-up spine system (c) Non-lift-up spine system

Fig. 13. Controlled-spine frame system concept.

Fig. 14. Tokyo Tech MCES building.

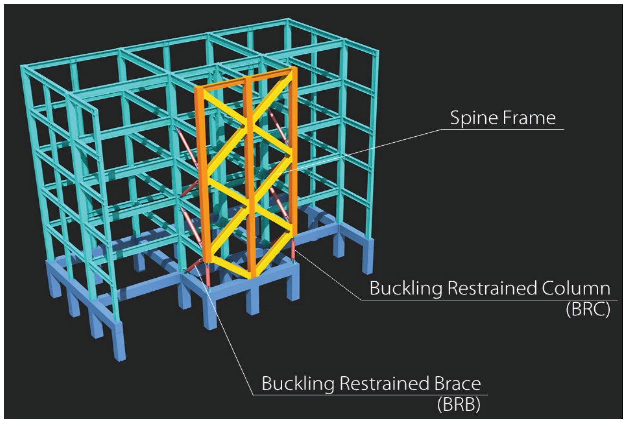


Fig. 15. Structural system for MCES building.

to concentrate deformations in the second story. By contrast, the lift-up model and non-lift-up model distribute the story drift angle more uniformly because of the spine mechanisms of the structures. All residual story drift ratio values are less than 0.12%, and less than 0.05% for the lift-up and non-lift-up models. These models confirm slight yield in the main frames, but almost elastic behavior. The residual drift ratio of both the lift-up and non-lift-up model is smaller than that of the shear damper model, and the damage in the envelope frame is minor in non-lift-up spine system and the shear damper model. This suggests that the elastic reaction forces from the envelope frame of the non-lift-up spine system are sufficiently large to overcome the residual axial force of the EDMs. This also indicates that the proposed non-lift-up spine system possesses excellent resilience when the envelope frame behaves elastically or even yields slightly.

The construction of the MCES building began in 2014 and ended in May 2015. **Fig. 17** shows the detail at the base of the controlled spine frame, and **Fig. 18** shows the construction scene at the base of the spine frame. The steel frame erection was performed in the same manner as that for an ordinary steel frame. Although the spine frame is located at the center of the building in this case, these elements may also be placed on the perimeter of the building to allow them to work in two individual directions. The placement of several controlled spine frames vertically for high-rise buildings will be also investigated in the near future.

5. Conclusions

This study reviewed three recent design approaches for passively controlled structures, and various application examples were discussed. The conclusions are summarized as follows:

1) The optimum arrangement of EDMs was investigated using a genetic algorithm, and the validity of the simulated arrangements was confirmed experimentally. Various optimization techniques can be

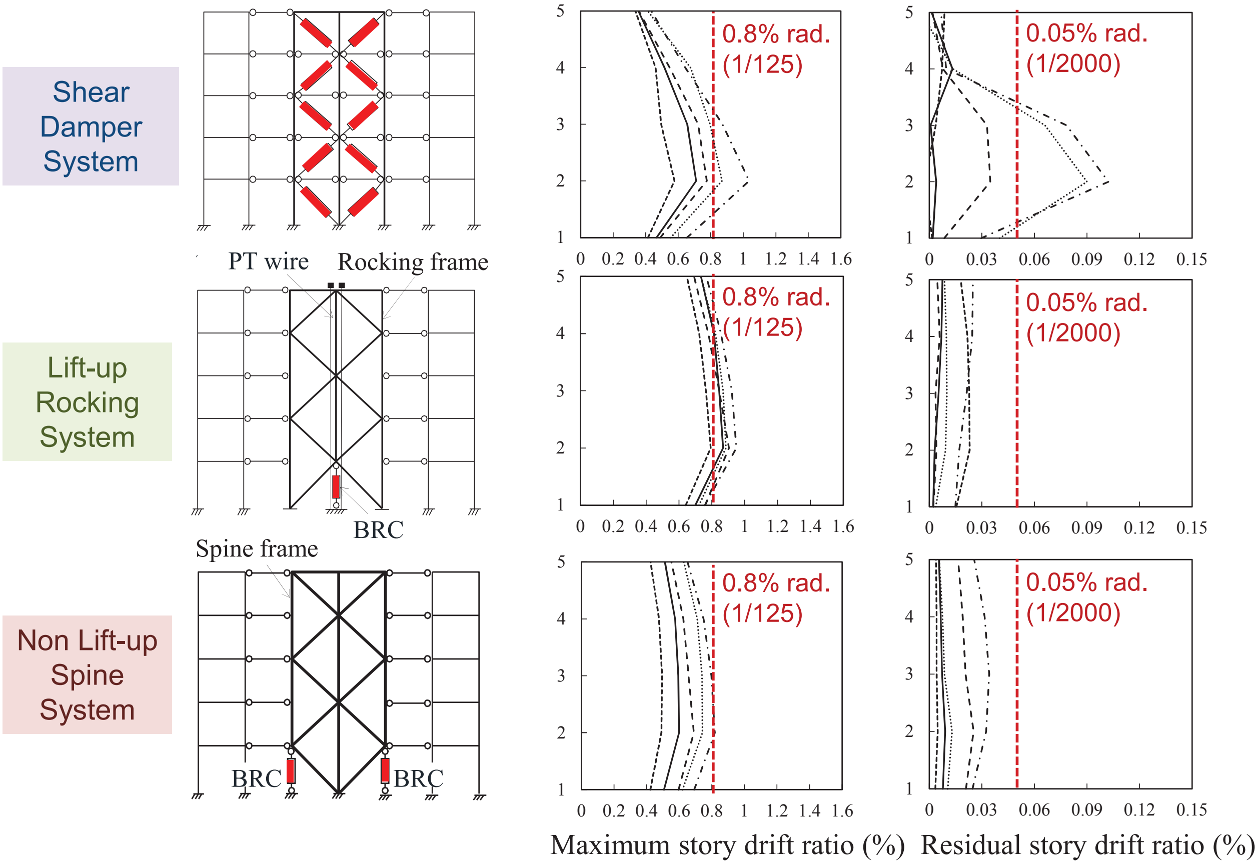


Fig. 16. Comparison of response in each structural system.

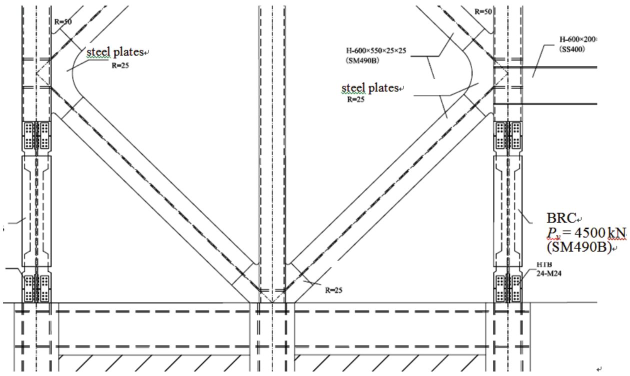


Fig. 17. Detail at the base of controlled spine frame.

Fig. 18. Construction scene at spine frame.

used for wider application in the preliminary design of passively controlled structures.

2) The application of the energy-dissipation concept, including seismic isolation, to spatial structures such as truss and latticed roof structures is considered for many uses, as it may drastically improve the performance of such structures. Providing EDMs between stiff and soft structures as RC and steel frames in composite structures can control structural responses to seismic shocks in various ways. An optimization algorithm can also be applied for EDM arrangement for space structures of freeform design.

3) The introduction of various spine frames with EDMs is an effective means of avoiding possible damage concentration at weak stories and residual drift ratio. In addition to self-centering systems using PT wires, ordinary elastic moment frames can also provide sufficient self-centering. The applicability of these frames to various structures, including high-rise buildings, will be studied in the near future.

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Academic Societies & Scientific Organizations:
• Archirectural Institute of Japan (AIJ)
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• International Association of Shell and Spatial Structures (IASS)