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State-of-Arts Views on Response Control Technologies on Metal Space Structures

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Abstract

Recently various kinds of passive response control techniques are taken into practice for buildings in seismic areas, and they start affecting on the architectural design itself. These trend affects also on spatial structures, and application of such techniques to domes, truss structures, or tension structures are recently increasing. Folllowing previous descussions [1], this paper firstly present the recent varieties of architectural expressions using seismic response controll technologies in buildings. Then, several important progresses in the application of passive control technology for metal spatial structures are introduced by refering recently realized projects, followed by discussions on their response characteristics and easy response evaluation methods.

Keywords: Response control, Seismic isolation, Damper, Dome, Cylindrical Shell

1. Introduction

Recently various kinds of passive response control techniques are taken into practice for buildings in seismic areas, and more than 80% of high-rise buildings designed after 1990 in Japan, for example, employs energy dissipation devices for reducing response against severe earthquake or heavy winds. Also the number of seismically isolated buildings exceeds 2,000. In normal buildings, they are divided into two categories. The first is seismic isolation, which 1) increases own periods by supporting the base of the buildings by rubber bearing, and 2) reducing response by adding dampers at the base. The second is passive controlled techniques realized by 1) designing main frames relatively soft and 2) adding dampers in each story. However, their border is becoming unclear along the developments of variety in architectural design using these techniques. Such trend also affects on spatial structures, and application of various kinds of response control techniques to domes, truss structures, or tension structures are recently increasing as introduced in previous paper (Takeuchi *et al.* [1]). This paper firstly present the recent varieties of architectural expressions using seismic response controll technologies in buildings. Then, several applications of passive control technology for metal spatial structures are introduced by refering recently realized projects, with their easy response evaluation methods proposed bu authors.

2. Architectural expressions using seismic response control techniques

Expending the numbers of applications, varieties of architectural expressions using seismic control techniques are also expanded. For example, designs as followings are appearing.

2.1. Application for fragile structures

Seismic isolation system enables to realize fragile structures which have not been possible in heavy seismic zones. Figure 1 is the boutique building constructed in Tokyo, in which the façade glass frames are supporting each floors (Nakai [2]). They also designed to resist elastically against the horizontal force produced by heavy seismic inputs. For reducing the seismic response and minimizing the frame member sizes, seismic isolation system was introduced. Figure 2 is a traditional brick building recently re-constructed with seismic isolation system, after its original construction in 115 years ago and demolished because of the poor seismic performance. Figure 3 is the flat-slab frame with slender columns also realized by introducing seismic isolation. As such, design freedom is expanding using these techniques.

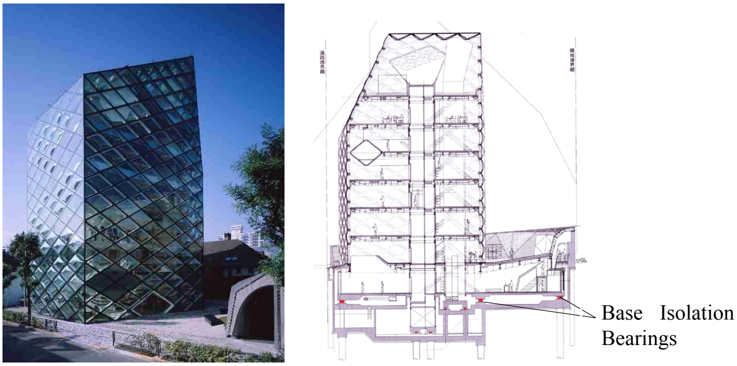


Figure 1: Prada boutique Aoyama

Figure 2:Mitsubishi Ichigokan

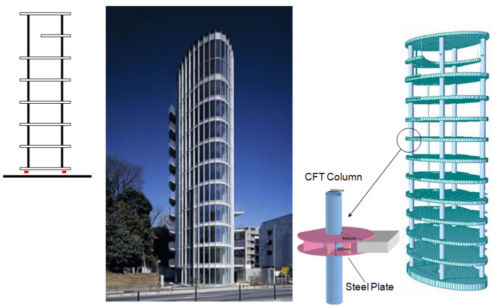


Figure 3: Igrek Shirokane

2.2. Mid-story isolated structures

Applications inserting seismic isolation bearing at mid-level of the structure, not at the base, are also appearing. Figure 4 is an example which piling the office block above hotel block. Large open atrium is placed at the lower part, and office block is supported by slender columns beside the atrium. Seismic bearings placed between the two blocks reduce the shear-force working on the office block, realizing this structure. Figure 5 shows mid-story isolated structure is used for retrofitting. This building is originally constructed with 2-stories, and upper 5-story steel structure is added with inserting isolation bearings in-between. Although the response characteristics of mid-story isolated structures are complicated because of the amplification produced by lower structure, architectural merits which enables different modules placed vertically on line leads to the increase of applications.

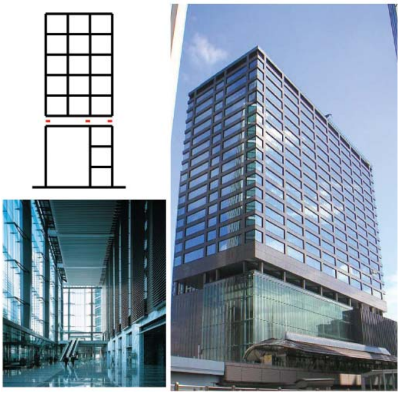


Figure 4:Shiodome Sumitomo building

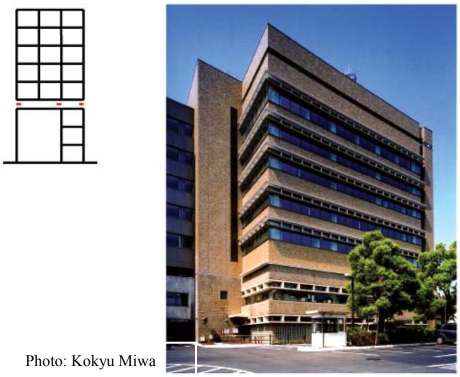


Figure 5: Musashino disaster mitigation center

There are another investigations on “step-over isolation”, in which extension structures are placed over the existing structures with slender columns as shown in Figure 6, producing long natural periods around 3 sec. without isolation bearings. It is confirmed that the shear-force introduced into existing lower structures are not increased but slightly reduced because of the energy-dissipation effect of dampers placed in-between (Takeuchi *et al*. [3]).

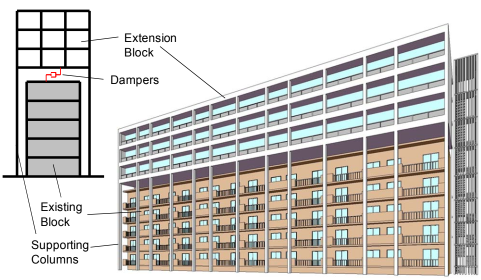


Figure 6:Step-over isolated extension

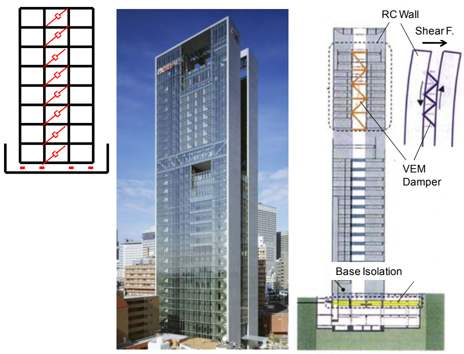


Figure 7: Yoyogi-seminar tower

2.3. Combined systems and other new structural forms

Some structures start to hire both of base-isolation system and passive control. Figure 7 is a school buildings with base isolation system, also introduces visco-elastic dampers at higher levels to reduce vibration produced by wind forces. In these structures, the border between the two systems is becoming unclear. New structural forms as in Figure 8, 9 are also appearing. Structure in Figure 8 places isolated bearing at the top of concrete core-wall, suspending all floors from the top allowing pendulum isolation mechanism (Nakamura *et al.* [4]). This system can realize much longer natural periods than ordinary base-isolation systems, and obtain stable reaction mechanism using *PΔ* effects. Figure 9 shows the building using the accommodation floors as mass-damper system, allowing floor structures slipping against the main structures and dissipating the vibration energy by the frictions and the damping of the bearings. As such, many varieties of structures are investigated using seismic response control techniques.

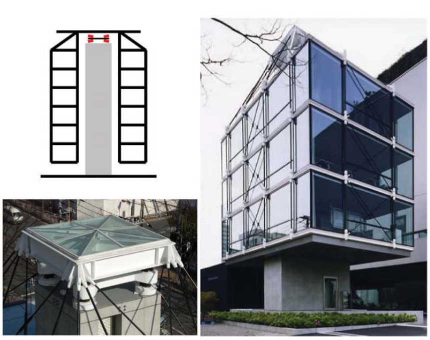


Figure 8:Core-suspended isolation system

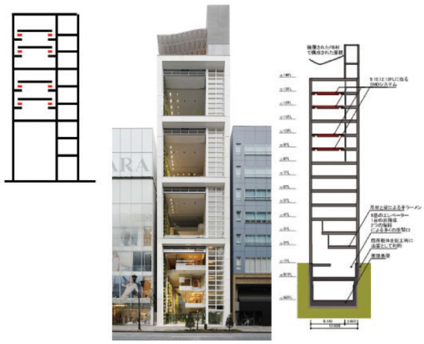


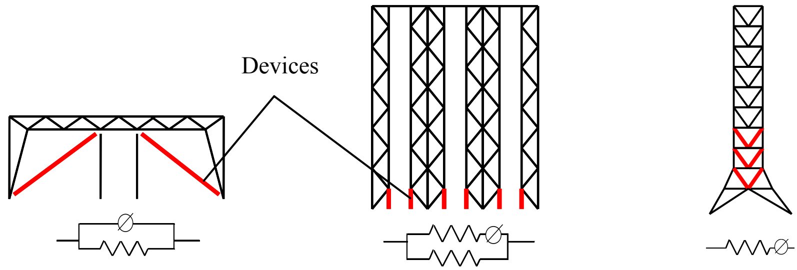
Figure 9: Floor mass dampers

3. Applications to spatial structures

3.1. Categories for structural types and typical examples

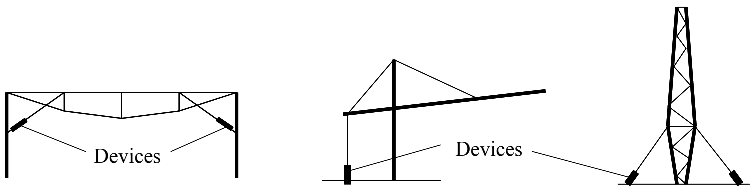
These design variations affects on spatial structures application examples of seismic response control techniques for latticed domes, shells, trusses, and cable structures are increasing. In previous paper [1], they are divided into a) truss structures, b) tension structures, and c) dome and shell structures, and categorized these structural types as in Figure 10, 11, and 12 respectively. In Figure 10, the layout patterns of the seismic energy dissipation devices for truss structures are shown. In (T-1), the devices are installed parallel to the main frame, while in (T-2), each chord member is replaced by a corresponding device in each truss column. (T-3) shows another layout for replacing the diagonal members to energy dissipation devices.

Figure 11 shows the typical layouts for cable structures. Parallel layouts (C-4) are often used as cable stays of bridges, and viscous or visco-elastic materials are used in the energy dissipation devices. For the series layout (C-1,2,3), devices are often required to maintain

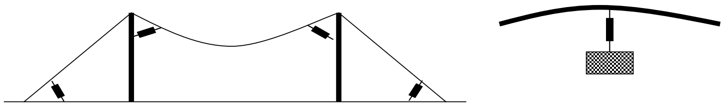


(T-1) Parallel layout (T-2) Series layout (Chord) (T-3) Series layout (diagonal)

Figure 10: Structural types for response-controlled truss structures



(C-1) Diagonal braces (C-2) Vertical anchors (C-3) Tower stays



(C-4) Cable vibration control (C-5) Pendulum control

Figure 11: Structural types for response-controlled cable structures

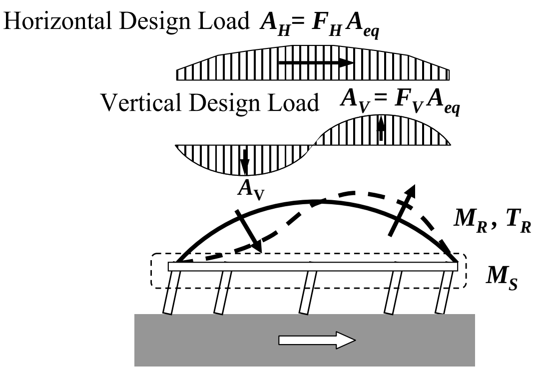


Figure 12: Seismic response of raised roof structures

Figure 13: Damages of non-structural elements in Hanshin earthquake 1995

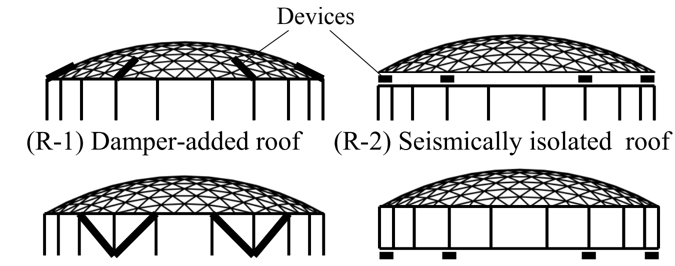


Figure 14: Structural types for response-controlled roof structures

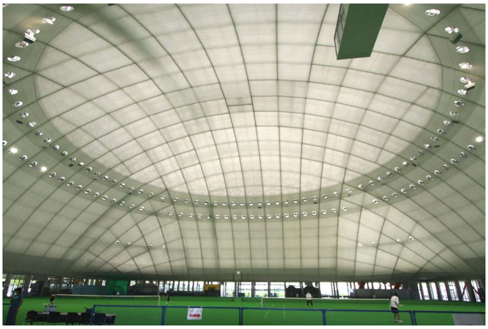
the pre-tension forces introduced in the cable. Devices layouting viscous or visco-elastic material with elastic springs in parallel are proposed and applied for sevaral realized projects (Takeuchi *et al*. [5]).

For roof structures, it is well-known that the seismic responses of raised lattice domes or shells amplify vertical vibrations even when subjected to a horizontal input. This acceleration can be roughly modeled as a combination of horizontal and vertical distributions, as shown in Figure 12. They can be a factor of seismic damages, not only structural, but non-structural elements as ceiling and suspended lightning equipments (Figure 13). Introducing seismic reponse control techniques as shown in Figure 14 are reoprted to be effective to reduce such damages. Especially the seismically isolated roofs, as shown in Figure 7 (R-2), has been practically implemented in many projects. In Japan, this system has been implemented in the Saitama Super Arena, Yamaguchi Kirara Dome (Hitomi *et al*. [6], Figure 15), Hiraga Dome (Figure 16), and Kyoto Aqua Arena etc. Laminated rubber bearings are used for supporting these roofs, reduction in the seismic responses and thermal stresses. On contrary, friction pendulum bearings are often used in United States, and applied to projects as terminal roof of the Istanbul Ataturk International Airport. In China, various kinds of seismic isolation bearings are under developping, and the Ovsavatory building in Shanghai International F1 Circuit is one of representative application examples.

Figure 15: Yamaguchi Kirara dome

Figure 16 Isolation bearings at Hiraga dome

(R-3) in Figure 14 is also one of the most popular application type. Figure 16 shows the Symokita Dome (Hitomi *et al.* [6]); in which, hysteretic damper braces are inserted into the supporting structure as well as the in-plane of the roof. (R-3) types are also expected to be used for the seismic retrofit of aged school gymnasium buildings with poor seismic performances, by replacing the existing diagonal braces into energy dissipation braces.



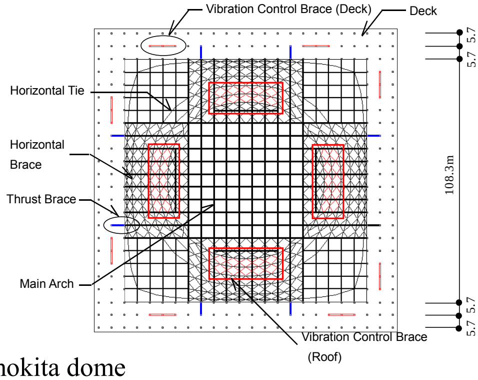


Figure 16: Shimokita dome

3.2. Response Reduction Effects and Easy Evaluation Methods

Response reduction effects of seismically isolated roofs (R-2 types) have been parametrically studied by (Kato *et al.* [8], Xue *et al.* [9], Takamatsu *et al.* [10]) and many other researchers. Roofs with damper-added substructure (R-3 types) have also been parametrically studied by (Yamada *et al.* [11], Takeuchi *et al.* [12]) and others. Some of their research includes easy evaluation methods for seismic response with these techniques, and they are helpful for preliminary design. On the followings, general findings for seismically isolated roofs (R-2 types) and roofs with damper-added substructure (R-3 types) are summarized and easy evaluation methods for these structures proposed by authors are introduced.

3.2.1. General Characteristics and Response evaluation for seismically isolated roofs with substructures

Figure 17 shows a typical analytical model of raised lattice domes (60m span, half-subtended angle θ=30°) with substructure. Seismic isolation bearings composed of isolator (horizontal elastic spring) and dampers (viscous dampers or elasto-plastic dampers) are inserted between the dome and substructure. Natural period of the asymmetrical one-wave mode for the dome T_D is about 0.3 sec, and isolators are designed as T_f=3.0 sec without dampers. Response accelerations distributions of this model along the ridge of the roof (AOA' in Figure 17) obtained from time-history analyses are compared in Figure 18 between ordinary domes without isolator, isolated with viscous dampers, and elasto-plastic dampers. As shown in the Figure, maximum acceleration responses of isolated roofs are reduced to 1/3 to 1/7 of ordinary roofs both in horizontal and vertical accelerations, and their response reduction effects are significant.
To obtain effective response reduction, the following points are found to be important.

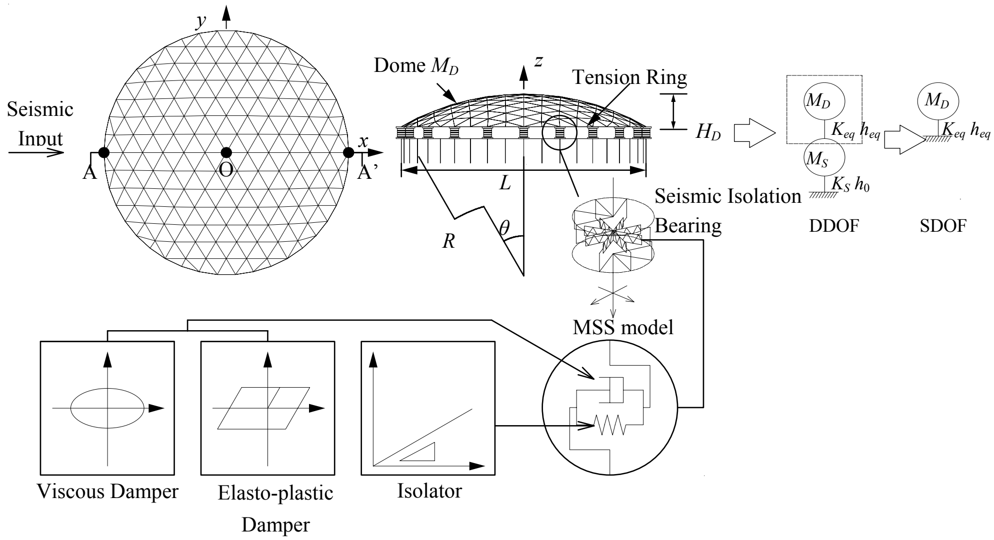


Figure 17: Seismically isolated dome with substructures

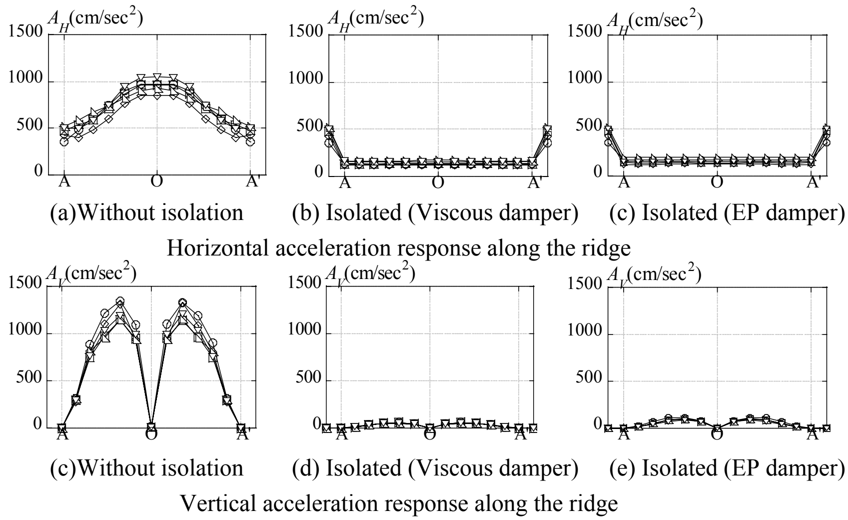


Figure 18: Response Reduction Effects by Seismic Isolation System

1) Designing natural period of isolation bearings much longer than the first natural period of the roof (generally asymmetrical one-wave vibration modes) is essential.
2) If the substructure stiffness is 5 times larger than isolator and substructure mass is less than 1.2 times of roof mass, the effect of substructure is negligible.
3) In case of applying elasto-plastic dampers, response should be estimated by the bearing stiffness avaraged between elastic stiffness and plastic stiffness.
4) Also for elasto-plastic dampers, vertical response is excited by first stiffness and effects of additional damping on vertical response are not remarkable.

Reflecting the above findings, seismic responses of seismically isolated raised lattice domes and cylindrical shells with supporting substructures can be evaluated with the following process. These process is based on the same apploach as the response evaluation methods proposed by authors for raised lattice domes and shells using amplification factors ([13],[14]).

1) Firstly, estimate the roof and substructures as rigid body and model the structure as SDOF model composed with roof mass M_R and isolator stiffness K_{eq} as shown in Figure 17, then estimate the natural period T_{eq} of a seismic isolation system with the following equation (1). For the elasto-plastic damper, assume the elastic natural period ${}_1T$, and equivalent natural period $T_{\max}$ by the equivalent stiffness at the point of maximum amplitude, then calculate T_{eq} as the avarage of those by the following equation (2).

$$T_{eq} = 2\pi\sqrt{M_R / K_{eq}} \qquad T_{eq} = (T_1 + T_{\max})/2 \ \text{(for EP damper)} \qquad (1)\,(2)$$

2) Calculate the total mass M_{eq}=M_R+M_S, where M_S is mass of the upper part of the substructures. Then calculate the natural period for substructure T_S using M_{eq} and substructure horizontal stiffness K_S. Also estimate the mass ratio R_M = M_{eq}/M_R = $(M_R+M_S)/M_R$. If R_M<1.2 and T_{eq}/T_S > 5.0, effects of substructure can be negligible, and roof response can be estimated using SDOF model. If not, estimation by DDOF model is required and detailed procedure is explained in [10].

3) Esimate the maxmimum acceleration response A_{eq} for SDOF model with design spectrum. Equivalent natural period T_{eq} shall be used for estimation. Additional equivalent damping by dampers can be included; however, additional damping effect of elasto-plastic damper should be ignored for vertical response evaluation.

4) Calculate the natural period ratio R_T= T_{eq} /T_R, where T_R is the natural period of asymmetric one wave mode of the roof. This value can be roughly estimated by the span and finishing as T_R=0.007L (Tatemichi [15]), however, detailed estimation by eigenvalue analysis is recommended.

5) Esimate the maximun acceleration amplification factors in horizontal and vertical directions using the following equations ($0 \le \theta < \pi/2$).

(Latticed Domes)

$$F_H = \begin{cases} C_H(\theta) & \left(0 < R_T \le 5/4\left(C_H(\theta)\right)^2\right) \\ \sqrt{5/4R_T} & \left(5/4\left(C_H(\theta)\right)^2 < R_T \le 5/4\right) \\ 1 & \left(5/4 < R_T\right) \end{cases} \qquad (3)$$

$$F_V = \begin{cases} 3C_V(\theta) & \left(0 < R_T \le 5/32\theta\right) \\ \left(\sqrt{5/2\theta R_T} - 1\right)C_V(\theta) & \left(5/32\theta < R_T \le 5/2\theta\right) \\ 0 & \left(5/2\theta < R_T\right) \end{cases} \qquad (4)$$

Where, $$C_H(\theta) = 2.47\sin^2\frac{3}{4}\theta - 1.33\sin\frac{3}{4}\theta + 3.0 \qquad C_V(\theta) = 2.47\sin\frac{3}{4}\theta\cos\frac{3}{4}\theta \qquad (5)(6)$$

(Latticed Cylindrical Shells)

$$F_H = \begin{cases} C_H(\theta)/2 & (0 < R_T \le 1/(C_H(\theta)-1)^2) \\ \left(\sqrt{1/R_T}+1\right)/2 & (1/(C_H(\theta)-1)^2 < R_T \le 5/4) \\ 1 & (5/4 < R_T) \end{cases} \tag{7}$$

$$F_V = \begin{cases} 3C_V(\theta) & \left(0 < R_T \le 5/32\theta\right) \\ \left(\sqrt{5/2\theta R_T}-1\right)C_V(\theta) & \left(5/32\theta < R_T \le 5/2\theta\right) \\ 0 & \left(5/2\theta < R_T\right) \end{cases} \tag{8}$$

Where, $C_H(\theta) = 2.47\sin^2\frac{3}{4}\theta - 1.33\sin\frac{3}{4}\theta + 3.0 \qquad C_V(\theta) = 1.77\sin\frac{3}{4}\theta\cos\frac{3}{4}\theta$ (9,10)

5) Calculate the maximum acceleration distributions using the following equations with A_{eq}, F_H, F_V and horizontal coordinates x, y. For high-rise roofs, spherical cordinates should be used refering [14].

(Latticed Domes)

Horizontal acceleration: Vertical acceleration

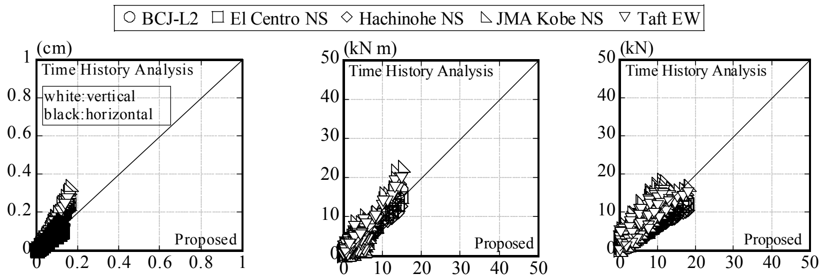
$$A_H(x,y) = A_{eq}\left\{1+(F_H-1)\cos\frac{\pi\sqrt{x^2+y^2}}{L}\right\} \qquad A_V(x,y) = A_{eq}F_V\frac{x}{\sqrt{x^2+y^2}}\sin\frac{2\pi\sqrt{x^2+y^2}}{L} \tag{11,12}$$

(Latticed Cylindrical Shells)

Horizontal acceleration: Vertical acceleration:

$$A_H(x,y) = A_{eq}\left\{1+(F_H-1)\cos\pi\left(\frac{x}{L_x}\right)\cos\pi\left(\frac{y}{L_y}\right)\right\} \qquad A_V(x,y) = A_{eq}F_V\sin\pi\left(\frac{2x}{L_x}\right)\cos\pi\left(\frac{y}{L_y}\right) \quad (13,14))$$

6) Maximum deformations and member forces are estimated using static analyses using equivalent loads derived from multiplying unit mass with acceleration distributions given by equations (11)-(14).



(a) Displacement (b) Axial forces (c) Bending morment

Figure 19: Validity of proposed evaluation method

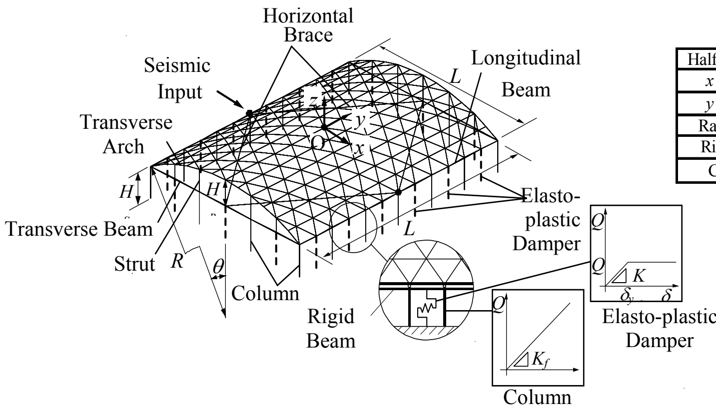
Obtained results by the method described in above 1)-6) process for the sample model are compared with the results of time-history analyses in Figure 19. They show general agreements, and these effects can be roughly estimated with the proposed method.

3.2.2. General Characteristics and Response evaluation for roofs with damper-added substructures

For roofs supported by damper-added substructure (R-3 types), similar findings as isolated roofs are obtained as follows.

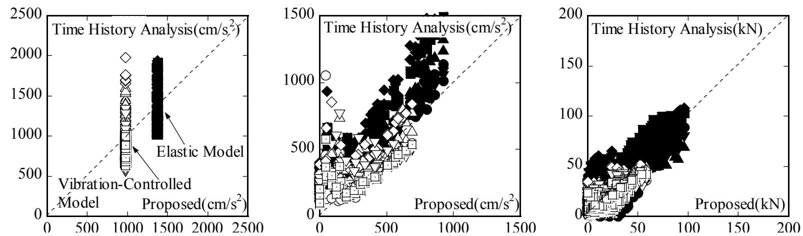
1) Maximum response acceleration distributions of the roofs supported by damper-added substructure are generally evaluated by SDOF model with equivalent stiffness and equivalent damping produced by substructures and dampers, and then multiplied by amplification factors.
2) As same as isolated roofs, effect of additional damping by elasto-plastic dampers on the vertical response is not remarkable.

The response evaluation process is almost the same as 1)-6) process in previous section; however in the process 1) and 3), equivalent stiffness and damping of substructure with dampers should be used for estimating T_{eq}.and A_{eq}, instead of isolator.



Half Subteded Angle θ (deg.)	30
x-dirention span L_x (m)	36
y-direction span L_y (m)	48
Radius of the Roofs R (m)	36
Rise of the Roofs H_R (m)	4.82
Column Length H_s (m)	6.0

Figure 20: Cylindrical shell with damper-added substructures



(a)Horizontal Acceleration (b) Vertical Acceleration (c) Member Axial Force

Figure 21: Validity of proposed evaluation method

Obtained results on the model as shown in Figure 20 are compared with the results of time-history analysis in Figure 21. In the figures, black marks are response with ordinary elastic substructures, and white marks are substructures with elasto-plastic dampers. It is observed that the response reduction ratios are around 60%. Also the proposed evaluation methods seem to be generally valid, although there are some errors.

4. Conclusive Remarks

This paper oveview the recent varieties of architectural expressions using seismic response controll technologies in buildings and metal spatial structures by refering recently realized projects, and their characteristics and response evaluation methods are summerrized.

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