Seismic Isolation of the Roof over a Large Space
Design of the Roof for Kyoto Aquarena

KANEDA.Katsunori [1]

[1] *Shibaura Institute of Technology. Structural Design Office Plus One Co.,Ltd. Japan*

Email: k-kaneda@sp-plusone.co.jp

ABSTRACT :

This indoor swimming pool has approximately 57-meter span at the maximum. The supporting structure is made of pre-cast pre-stressed concrete and the roof structure is composed of beam string structure (BSS). The base isolated system, the combination of laminated lubber and hysteric damper is placed on the connection between the supporting part of BSS and lower structure. Therefore the vibration response against seismic and wind force is reduced as well as the thrust caused by thermal stress and snow load is reduced evidently.

KEYWORDS: Seismic Isolation of Roof, Pre-cast Pre-stressed Concrete, Beam String Structure

1. OVERVIEW OF ARCHITECTURE

1.1 Summary of Architecture:

Kyoto Aquarena is an indoor sports arena neighboring Nishikyogoku Sport Park in Kyoto. These were the key considerations during the design of Kyoto Aquarena:

・Since it was one of the facilities of the Sport Park, it needed to be completely integrated with its surrounding environment.

・In order to preserve the global environment, it needed to be designed to consume a minimum of energy and release the minimum possible CO_2.

The following elements were incorporated into the planned structure, on the basis of those considerations:

・The entire roof was planted with vegetation, except for the portions over the approaches and over the pool.

Solar heating panels were installed on the roof over the main pool to warm the water for the pools, for air conditioning and to heat the floors.

Photovoltaic panels were installed on the roof of the sub-pool (Figures 1, 2).

This complex consists of a sub-pool, parking lot, archery range and other facilities were laid out in a rough circle around the largest wing, which houses the main pool. The main pool is Olympic-sized, consisting of an officially recognized 50 m section and a diving section. It is also designed be used for ice skating in the winter. The entire parking garage and parts of the walls of the pool buildings consist of embankments for planting vegetation (Figures 3, 4).

Figure 1: Aerial view of Kyoto Aquarena

Figure 2: Layout of Kyoto Aquarena

Figure 3: Main pool

2. OVERVIEW OF STRUCTURE

2.1 Summary of structure:

The supporting structure was built of steel-reinforced concrete because it will be in contact with the soil, because exposed steel is vulnerable to chlorine corrosion, and for other reasons. The lobby and other large spaces were built of pre-stressed concrete, cast in-situ, to provide support for the roof without columns. The roof over the main pool has a rather complicated shape, so pre-cast pre-stressed concrete was used there (Figures 5, 6)

All roof structures over the main pool and sub-pool were made of hot-dip galvanized steel. Isolators, dampers and sliding bearings were installed at the junction between the roof of main pool structure and supporting structure to isolate the roof from seismic vibrations (Figure 7).

Figure 4: Diving pool

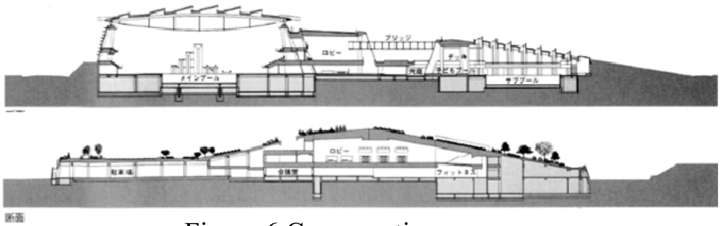


Figure 6 Cross-section

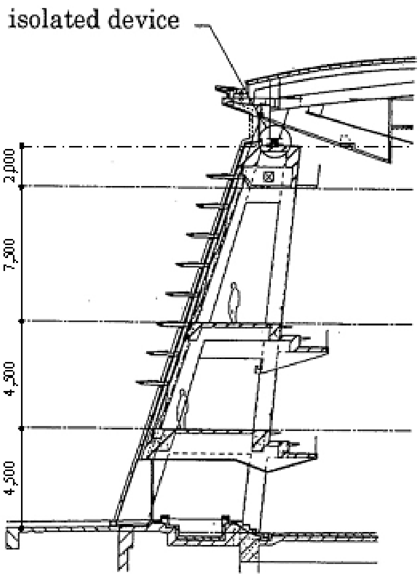


Figure7 Detailed section drawing

Figure 5: Exterior view

2.3 Supporting structure:

The plan form of the building surrounding the 2 pools is bean-shaped, where the exterior walls are composed of four circular arcs. The structure bearing the roof follows the curves of the exterior walls. Supporting columns slope radially toward the arc centers in pairs (Figures 8, 9).

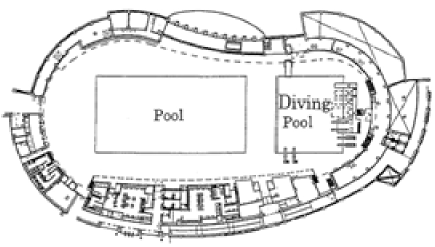


Figure 8 Plan view of main pool

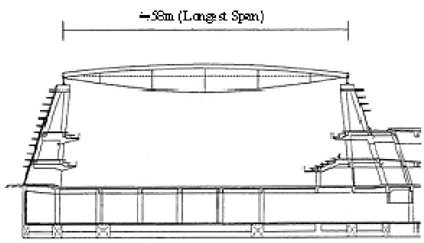


Figure 9 Framing elevation of main pool

The lower portions of the structure bearing the roof were cast in-situ of reinforced concrete. Of the structure above the first-level floor, only the seismic stress-bearing walls were cast in-situ. The remaining columns, beams and floor slabs were all constructed of pre-cast pre-stressed concrete (Figures 10, 11).

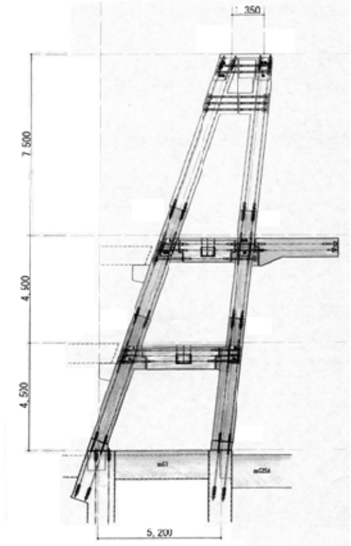


Figure 10: Installation diagram of pre-cast concrete componentwing

Figure 11: Supporting structure (pre-cast pre-stressed concrete)

2.4 Roof construction:

The roof structure incorporates beam string structures that cross the lines of the pools at right angles, providing a 2-dimensional planar structure. The reason for doing this are provided below:

・ High-tension power lines cross diagonally above the roof of the main pool, so the maximum allowed height of the building was 25 m. Work during the construction process was also required to be performed at heights no greater than 25 m.

・ The plan form of the building consists of curves that have reversing curvatures, but are continuous. Therefore, we cannot assume any meaningful resistance to the horizontal thrust forces from the roof structure by beams in the roof periphery acting as a tension ring.

・ Regulations governing the roof structure for indoor competition pools require that structural members that are visible to swimmers swimming on their backs to run either parallel or perpendicular to the pool, so that they do not confuse the swimmers.

・ The relatively complicated plan form of the roof was then simplified (Figure 12).

The main design characteristics of the beam string structures were as follows:

・ The upward force applied by the strut at the midpoint of the beam acts to support the upper chord member at that location; this greatly reduces the bending stresses in the members.

・ Since this results in a self-balanced structure, where the compressive stresses arising in the upper chord member at the supported end of the beam come to equilibrium with the tensile stresses in the lower chord member, The overall weight of the roof structure can be reduced while avoiding horizontal thrust loading on the supporting structure.

Due to the height restrictions beneath the high-tension power lines, it was not possible to use large equipment during construction; Therefore, the beam string structures for the large roof were assembled away from the power lines, brought to the site and slid horizontally into position (the sliding stage construction method). No special jigs or other components were needed for the horizontal shift of the self-balanced beam string structure. This design simplified construction of the Aquarena.

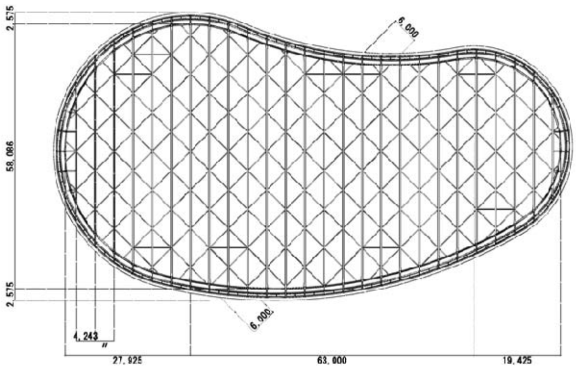


Figure 12: Roof frame plan

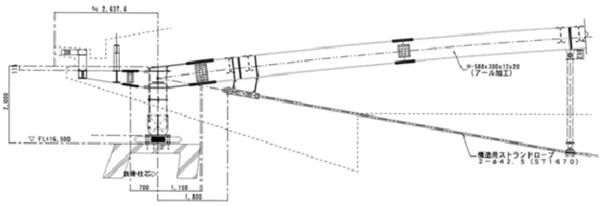


Figure 13: Beam string structure detail
Dotted lines show final locations of roof and ceiling surfaces

2.5 Objectives of Seismic Isolation of Roof:

As described above, the self-balanced beam string structure design exerts no horizontal forces on the supporting structure during ordinary loading conditions. In other words, under ordinary use conditions, the roof only exerts loads in the vertical direction on the support points. Accordingly, seismic protection was placed at the junction between the roof structure and the supporting structure in order to isolate the roof from seismic shaking (Figure 13).

By isolating the roof from earthquake motions, the horizontal forces on the supporting structure during quakes are reduced, greatly reducing the quake-induced vibration susceptibility of the various portions of the roof structure. This allowed the designers to lighten the materials used in the roof structure itself, and in the supporting structure, and to provide increased safety factors against detachment and collapse of both the solar panels mounted on the roof and the internal components mounted on the ceilings.

After construction, the isolating components, which have a low stiffness in the horizontal direction, are quite effective at maintaining the mechanical self-balancing of the beam string structures and for restraining the horizontal location of the roller bearings under the roof supports, which is an essential design condition.

2.6 Seismic protection components:

Three kinds of roof bearings were combined to provide the longest period of natural vibration possible. The first bearing was a laminated natural rubber isolator incorporating a U-shaped steel hysteretic damper (Type A bearing). The second was a bearing consisting only of an isolator (Type B bearing). The third was a sliding bearing (Type C bearing); see Figures 14 – 16.

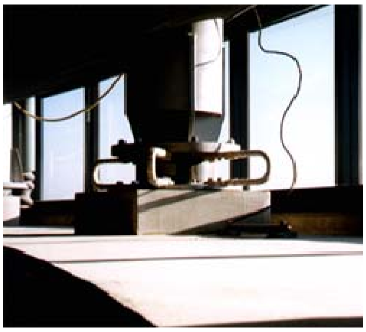


Figure 14: Bearing A

Figure 15: Bearing B

Figure 16: Bearing C

Table 1: Specifications of laminated rubber isolators

Rubber type	Natural rubber
Rubber O.D.	350mm
Rubber thickness	2.63 mm, 26 sheets
Shear modulus	0.34 N/mm^2
Horizontal modulus	0.49 KN/mm
Steel plate thickness	3.2 mm, 25 sheets
Steel mounting plate	12 mm (SS 400), zinc electroplating
Flanges	22 mm (SS 400), zinc electroplating

Table 2: The first rigidity and surrender shearing power of U-shaped hysteretic dampers

	A direction	B direction
Primary stiffness	4.2(+7.5%)	3.6(-7.5%)
Yield shear force (kN)	55.9(+9.6%	46.1(-9.6%)

In order to obtain the longest possible natural period in the vibrating system during a major earthquake, isolators were chosen with the lowest available shear modulus (after yielding of the damper). The shape factor of the primary isolator was 33.3 and that of the secondary isolator was 5.3 (Table 1).
The U-shaped hysteretic dampers are made of ordinary steel (SN490B) formed into the desired shape. Tests of the dampers showed a high capacity for energy absorption that had almost no dependence on the magnitude of deformation, direction of deformation, temperature or other parameters (Table 2, Figures 17 – 19).

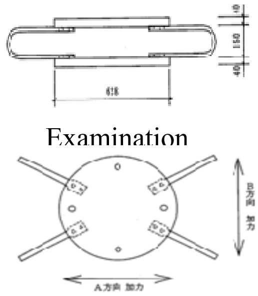
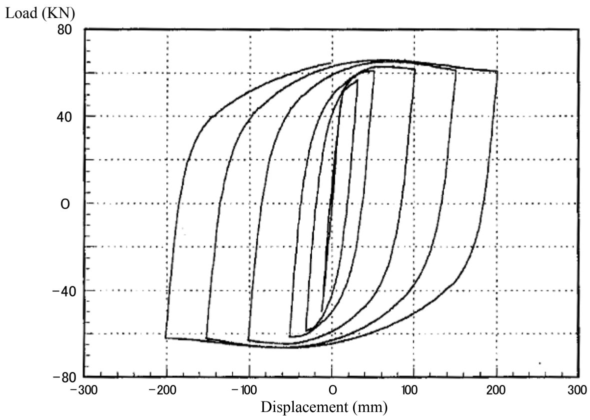
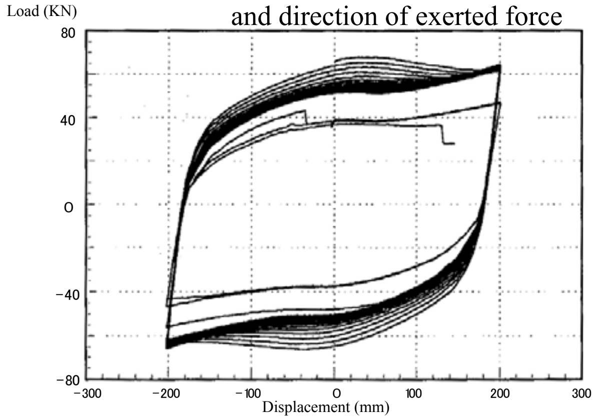


Figure 17: U-shaped damper and direction of exerted force

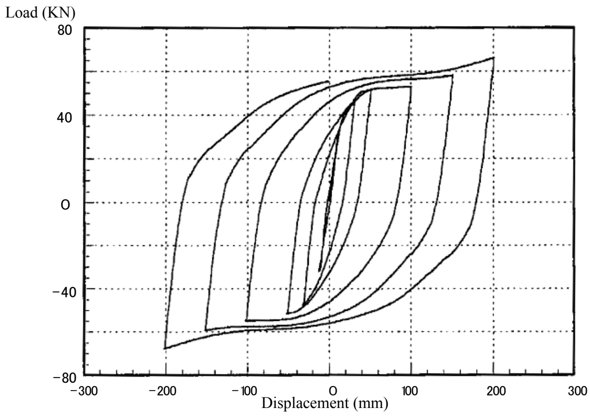


Results of test under slowly increasing force in direction A

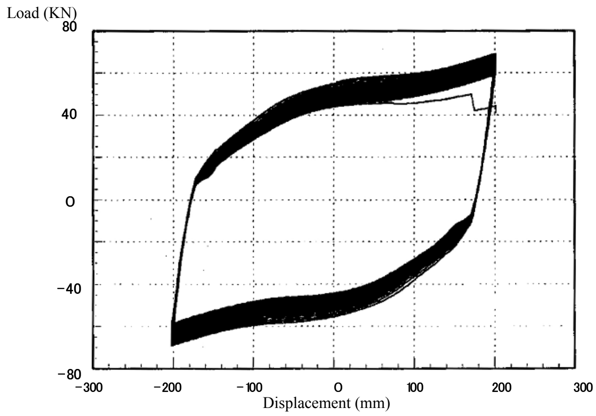


Results of test under constant reversing displacement in direction A

Figure 18: Results of test in direction A



Results of test under slowly increasing force in direction B



Results of test under constant reversing displacement in direction B

Figure 19: Results of test in direction B

The bearings were installed in locations that had been calculated to keep damper displacements under wind loads lower than the plastic range and to provide the longest possible natural periods (Figure 20).

3. ANALYSIS OF RESPONSE TO VIBRATIONS

3.1 Analytical Model and Characteristic Values:

A simple model was made for analysis of the supporting structure, with components having equivalent cross-sections to those of the beams and the assembled columns. The model of the roof structure previously used for static analysis was used same model for the dynamic analysis (Figure 21). The characteristics of the hysteresis characteristic in the isolating devices were as shown in Figure 22, on the basis of the test data.

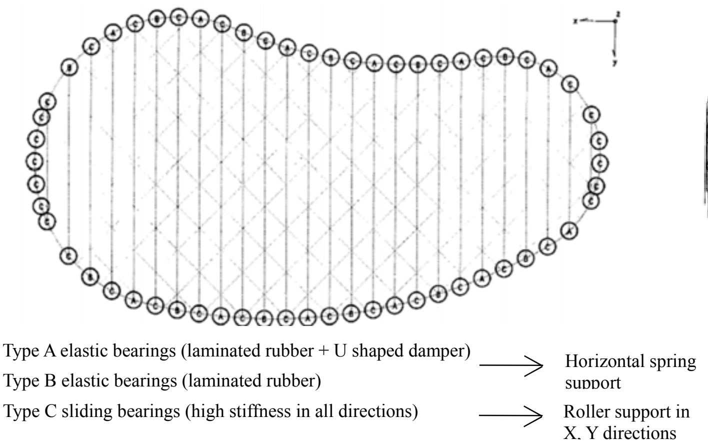


Figure 20: Installation locations of seismic protection

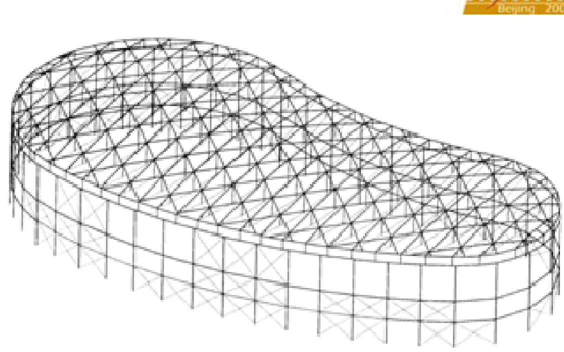


Figure 21: Model for analysis of response to vibrations

In order to compare the data obtained for the vibratory response, analyses were performed on both the isolated and non-isolated structures. As can be seen in the vibratory modes of the two models in the horizontal direction, the most marked difference between them was that the isolated structure showed almost no deformation of the roof structure as it rocked in the horizontal direction. In contrast, the non-isolated structure showed large deformations of the roof structure in the vertical direction as it rocked horizontally (Figure 23).

The calculated value of the primary vibration period was 1.3 s during small deformations and 2.2 s during large deformations following yielding of the dampers.

Real earthquake time histories, which are typically used in Japan, were employed as the input waveforms (Table 3).

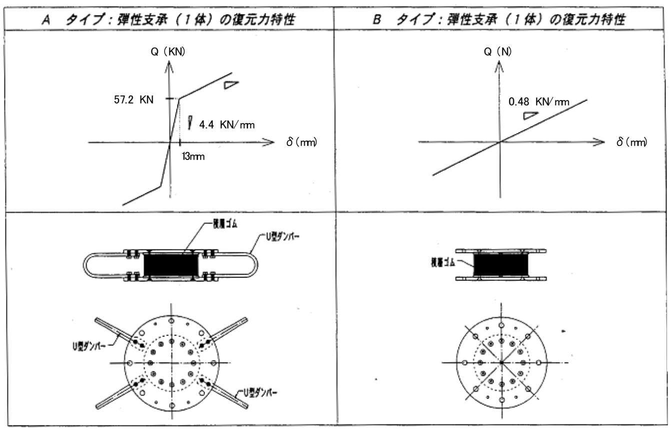


Figure 22: Restoring forces in seismic devices

Table 3: Earthquake time histories

Input Earthquake wave	V_{max} (cm/sec.)	a_{max} (cm/sec.^{2})
Kobe 1995 NS	50	444
El_Centro 1940 NS	50	511
Taft 1952 EW	50	497
Hachinohe 1968 NS	50	330
Kobe 1995 UD	25	204
El_Centro 1940 UD	25	317
Taft 1952 UD	25	240
Hachinohe 1968 UD	25	139

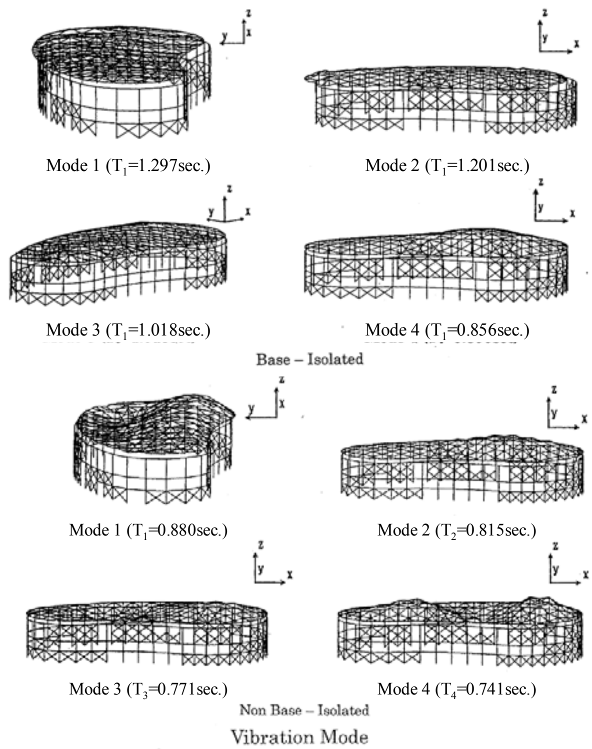


Figure 23 Model for Vibration mode

3.2 Analytical Conditions:

The conditions for analysis of the response to the seismic time histories were as follows:

Mass: The mass of the roof surface was modeled as concentrated masses fixed to each node.

Boundary conditions: The lower ends of the columns of the supporting structure (at the 1st floor level) were fixed.

Analytical method: A numerical approach based on the Newmark β method was used ($\beta = 1/4$).

Damping coefficient: Rayleigh internal viscous damping was assumed, primary damping coefficient of 0.02

$[C] = a_0[M] + a_1[K]$

$a_0 = 0.1245$, $a_1 = 0.0030$

Employed software and equipment: ADINA Ver. 7.1, ULTRA1 (Sun Microsystems)

3.3 Results of Analysis of Response:

Table 4 shows the results of analysis of responses to horizontal earthquake time histories. No marked differences were found in the magnitudes of response to the each earthquake time histories. The maximum deformation of the laminated rubber was 19.4 cm; the corresponding shear deformation ratio was 285%. Figure 24 presents an example of a typical deformation history.

The chief differences between the isolated and non-isolated structures were as follows (Figure 25):

・ In the base-isolated structures, not only is the response acceleration dramatically reduced, the difference in the response due to variations in the seismic waveforms is reduced by the high damping coefficient.

・ No large differences among response s by the supporting structure were observed with different earthquake time histories. However, in the non-seismic structures, the storey shear coefficient increased with distance up the laminations, while the coefficient was roughly constant throughout the damper body in the seismic structure.

・ The vertical deformation of the roof structure in response to horizontal shaking was reduced to about 1/3 the magnitude at the beam ends and about 1/5 the magnitude at the center of the beam by seismic isolation(Figure 26).

Table 4: Predicted results of horizontal motions resulting from earthquakes

Seismic wave	Direction	Max. shear stress coeff.	Max. rel.displacement.
Kobe NS	X	0.16	170.8 mm
	Y	0.17	172.0 mm
	45	0.16	172.6 mm
	-45	0.17	166.0 mm
El-Centro NS	X	0.17	175.2 mm
	Y	0.17	167.8 mm
	45	0.16	158.1 mm
	-45	0.19	191.6 mm
Taft EW	X	0.13	126.0 mm
	Y	0.14	139.8 mm
	45	0.13	136.2 mm
	-45	0.13	135.6 mm
Hachinohe NS	X	0.15	163.2 mm
	Y	0.14	144.8 mm
	45	0.14	143.4 mm
	-45	0.15	169.0 mm

Table 5: Predicted vertical motions

Seismic wave	Max. shear stress	Max. vert.
Kobe UD	0.3	140.1 mm
El-Centro UD	0.21	70.2 mm
Taft UD	0.13	74.7 mm
Hachinohe	0.15	96.1 mm

Since the seismic isolation was only applied in the horizontal direction, almost no benefit of the isolation was seen in response to vertical seismic shaking.

These results indicate that the design value for the storey shear coefficient of the roof should be about 0.2 for the horizontal direction and 0.3 for the vertical direction(Table 5).

4. SUMMARY

The main benefits from installation of seismic isolation in the roof of the Kyoto Aquarena are as follows:

・ The seismic safety factors of the structure were raised for horizontal seismic motions.

・ A large reduction was realized in both the vertical and horizontal motions of the roof body in response to horizontal seismic shaking. This resulted in increased safety factors with respect to falling of or damage to roof trim and internal furnishings in the facility.

・ There was a lower scatter in the magnitude of vibratory response due to differences between seismic waveforms, and the relation between earthquake motion magnitude and quake-induced loads on the structure was greatly simplified.

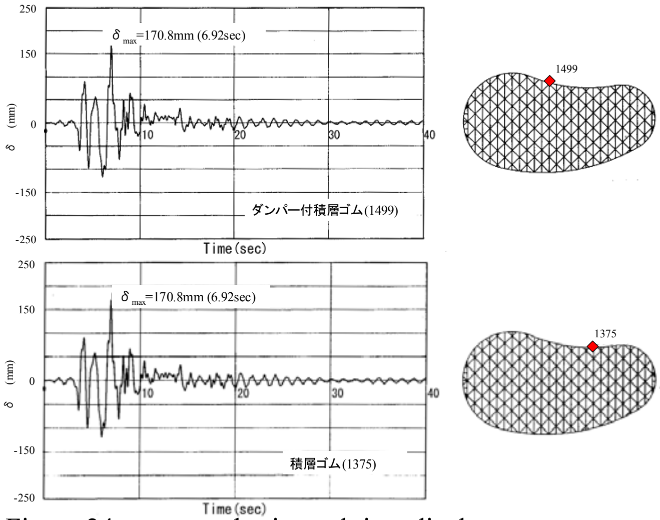


Figure 24: response horizontal time displacement
(Kobe NS:X direction)

・ At the support points for the roof structure, there are nearly zero horizontal thrust loads from external forces, i.e., from forces other than thermal loads in the roof surface, vertical seismic motions, snow loads and wind loads.

・Skeleton costs were reduced.

Usually, it is more difficult to design seismic isolation systems for roofs with long natural periods than to do so in systems for foundations. This is due to the relatively light weight of roofs and is usually considered one of the chief drawbacks of roof systems. However, from the viewpoint of cost, roof isolation systems are less expensive or, at least, no more expensive than foundation systems. It was not possible to make such comparisons in this design, but this method allowed the designers to "tune" the natural periods of the roof structure and supporting structure. This appears to be a promising approach for controlling the effects of seismic shaking of roof structures.

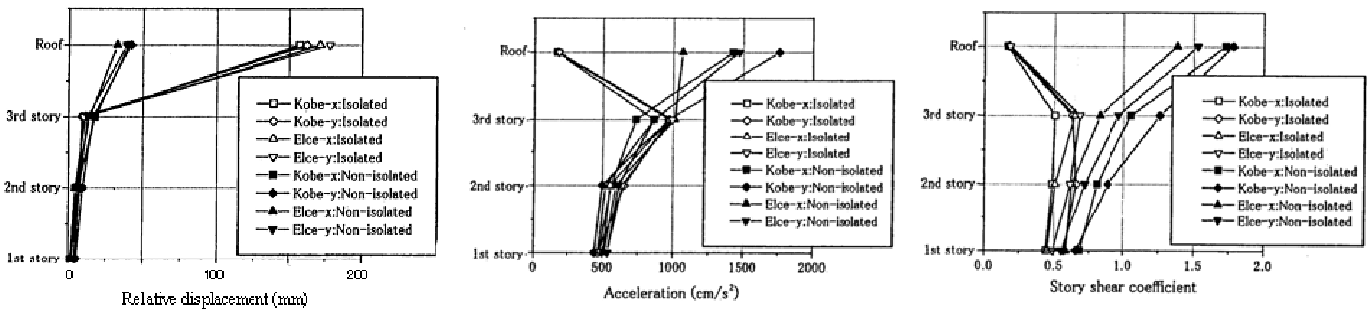


Figure 25: Comparisons of horizontal response displacements in seismic and non-seismic structures

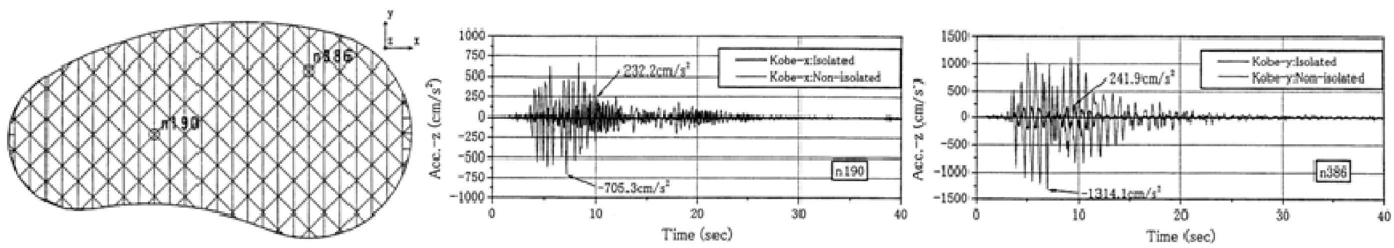


Figure 26: Comparison of vertical response displacement velocities during horizontal seismic waves