RESTORATION OF AKASHI MUNICIPAL MUSEUM OF ASTRONOMY AND SCIENCE DAMAGED BY THE GREAT HANSHIN EARTHQUAKE IN 1995

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SUMMARY

This building is situated on the Japanese Standard Time meridian, at 135 degrees east longitude. The building, a four-storied structure of reinforced concrete, was built in 1960. A tower of double-tube structure stands about 37 m above the top of the building. It was damaged to a great degree, especially the tower , by the Great Earthquake in the Southern Hyogo District on January 17, 1995. Immediately after the earthquake, close structural examination and design preparation for restoration were conducted. This restoration work was completed in December 1997. This paper reports on the general conditions of the damage to the building and the seismic performance of various sections of the building as well as the restoration method utilized.

The seismic performance of the inner tube of the tower was examined. It was then made clear that the ratio of the effective flexural yield strength to the required flexural yield strength on the 13th to 15th floors in particular where damage was quite serious was merely 25 to 35%. The seismic performance of the outer tube of the tower was examined in two directions, the buttress direction and the other direction at an angle of 45 degrees from the buttress direction. As a result, the seismic performance on the 13th and 14th floors was confirmed to be low in both the buttress direction and the 45-degree direction. The seismic performance in the longitudinal direction was confirmed to be low on the 1st and 4th floors. Additionally, the same performance in the transverse direction was confirmed to be low on the 1st through 4th floors.

Concerning the restoration work, seismic reinforcement work was executed in addition to the repair work. The internal tube of the tower was reconstructed. The external tube of the tower was retrofitted by reinforced concrete cast additionally and the use of carbon fiber sheet. The 1st through 4th floors of the building had to employ a reinforcement method with new steel braces installed.

The seismic performance of the building increased 1.11 to 2.64 times the level recorded before the reinforcement work and a sufficient degree of seismic safety was secured.

INTRODUCTION

This building, AKASHI Municipal Museum of Astronomy and Science, is located at 135 degrees east longitude or on Japan´s standard civil time meridian circle. The building, especially its tower structure, sustained considerable damage at the earthquake which occurred at the southern part of Hyogo prefecture on January 17, 1995. The damage estimation was conducted just after the earthquake. Based on the estimation, a detailed research of the damage, an evaluation of seismic capacity and design for repair/retrofit had been conducted from May to September, 1995. The restoring work was started during December, 1995 and was completed by December, 1997. This paper will discuss the damage situation of the structure, seismic performance of every part of the structure, its restoring plan and method, and its seismic performance following the restoration.

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OVERVIEW OF THE STRUCTURE

This building is a reinforced concrete building (partially steel-frame building) of four stories built in 1955 and there is a tower of 37 m height on top of the building. The tower has a double structure consisting of inner and outer towers and each of these has independent stories for the 5th and higher stories. Inside the inner tower, there is an elevator shaft, and its reinforced concrete spiral stairway is constructed of cantilever slabs. In the 4 directions of OuterTower, buttress is constructtd. In addition, this bulding is built on the cut part of leveled ground by cutting and banking into slope, thus the building has some single sided earth pressure up to second story. An overview of the building is shown in table 1.

Table 1 Outline of the building

a)	USAGE	*ronomical Science Museum*	*anetarium, exhibition room, observatory, observation room)*
b)	DIMENSION	Building Area:	1060.46 m^2
		Total Floor Area:	3124.32 m^2
		Number of Stories:	Low Storied Section: 4 (above the ground) (H:16.42 m)
			High Raised Section: 16 (above the ground) (H:53.46 m)
c)	STRUCTURE	Low Storied Section:	reinforced concrete with frame structure (Only the roof of the Planetarium is constructed with a steel-framed structure)
		High Raised Section:	Cylindrical shaped double structure (inner and outer) reinforced concrete tower (the outer tower is buttressed)
d)	FOUNDATION	Direct Foundation of Independent, Continuous Footing and Mat Type (prescribed bearing capacity of soil 25t/m^2)	

OVERVIEW OF THE DAMAGE

Damage situations affecting each section are shown in figure 1. An outline of the damage for each section of the building is as follows:

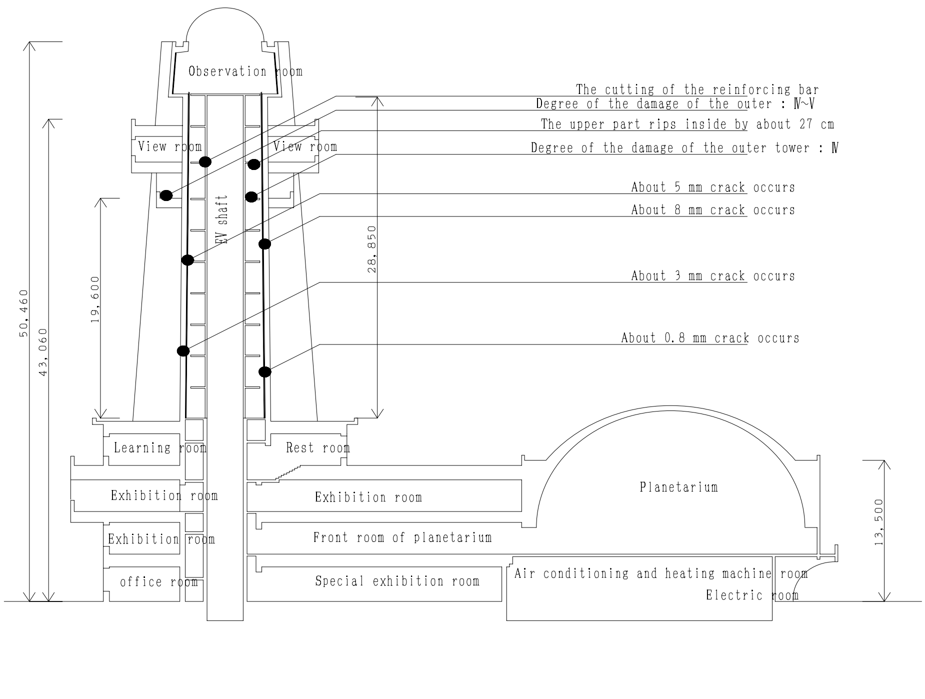


Fig.1 The main damage of the building

Tower Section:

Damage to the tower is especially significant around the 13th and 14th stories. Particularly, around the 14th story of the inner tower, there was clearly visible structural damage. Thus, the upper part of the tower penetrated into its lower part at the north-east side by 27 cm approx. and inclined about 2.5 degrees (0.044 rad.) on the north-east side. In contrast, the tower was raised by 22 cm at the south-east side and thus the main reinforcement was ruptured in this part. This inclination was approximately 4.5 degrees (0.079 rad.). In the outer tower as well, the damage around the 13th story was mostly significant (damaged degree: 4-5). At the 8th to 12th stories, there was shear crack of 3.0-8.0 mm width occurring from the corners of the opening area. The 7th or lower stories of the outer tower experienced only minor damage. In addition, at around the 15th story, there was damage from a collision between the inner and outer towers.

Low Storied Section:

Concerning the low storied section, there was shear and flexural crack (Maximum 2.0 mm width approx., all in the single direction) occurring on the wall of first story. This damage occurred because of the one sided soil pressure rather than vibration of the earthquake.

Ground:

From a measurement of floor levels for one to three floors, it was clear that the building was wholly inclined to the south side by 1/389 – 1/1514. In addition, from the front yard being inclined to the south side and the damage situation affecting the low storied section, it is assumed that there was some soil movement or ground subsidence from north to south affecting the whole building site.

SEISMIC PERFORMANCE

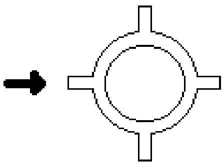
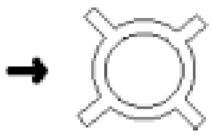
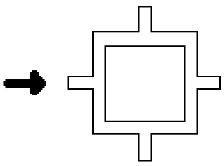
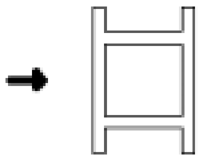
Inner Tower Section:

The seismic performance of the inner tower section was calculated for the 6th or higher stories in which the inner tower was separated from the outer tower. This calculation was conducted in compliance with a secondary diagnosis of an “evaluation of durability and seismic capacity for an existing RC stack [1].” For input acceleration, maximum acceleration 481gal measured at Japan Rail AKASHI station on the occasion of this earthquake was used. Key conditions of calculation are listed in table 2. From that, all of the average bending stress was 20 kg/cm^2 or higher. However, the prevalence flexural capacity was small in comparison with

Table 2 Calculated condition at inner tower and outer tower

Inner tower	Height	Diameter	Thickness of highest point	Thickness of bottom point
	45.46m	334cm	12cm	25cm
Outer tower	Height	Diameter	Thickness of highest point	Thickness of bottom point
	50.46cm	665cm	12cm	25cm
Commonness	Compressive strength of concrete	Yield strength of reinforcing bar	Natural period	
	250kg/cm^2	3.0t/cm^2	0.75s	

Table 3 Model to calculate strength at outer the tower

	The direction of the buttress	The 45 degree direction of the buttress
Actual section		
Model section		
Shear strength	Effective length of rectangular wall are tensile side 0.2 h and compressive side 2 t (h Shear span of wall t thickness of rectangular wall)	One of small value in shear strength to be considered rectangular wall and shear strength of web
Bending strength	To calculate bending strength of the basis floor (6^{th} stories) of tower by the virtual work method	

the required flexural capacity at the 7^{th} to 15^{th} stories as shown in figure 2. At the 13^{th} to 15^{th} stories in which the damage was especially significant, prevalence flexural capacity was 25% - 35% of the required flexural capacity.

Outer Tower Section:

A seismic capacity (Is: Structural Seismic Index) of the outer tower (6^{th} and higher stories) was calculated in compliance with the "Overall Earthquake Resistant Design Code for Government Facilities, Earthquake-Proof Inspection/Workover Instruction of Government Facilities [2]." The Earthquake Index ($I*I_E$) for determining seismic performance will be $I*I_E = 1.31$ by considering the Standard Shear Force Coefficient (C_o=1.0), the Vibration Characteristic Coefficient (Rt=0.99), the Earthquake Index Correction Coefficient per Geography (G=1.1) and Important Coefficient (I=1.2). The calculation of the Structural Seismic Index was conducted by cross-section model listed in tables 2 and 3 for buttress direction and buttress 45 degrees direction [3]. As a result (refer to Fig.4 and 5), for buttress direction for the 6^{th} – 14^{th} stories and for buttress 45 degrees for all stories, the Structural Seismic Index**,** Is were less than $I*I_E$. Particularly, both for buttress and for buttress 45 degrees, the seismic capacity of the 13^{th} and 14^{th} stories were poor.

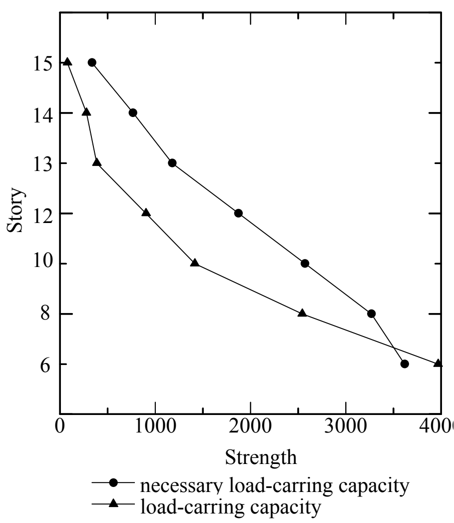


Fig 2 Relationship between load-carrying capacity and required capacity

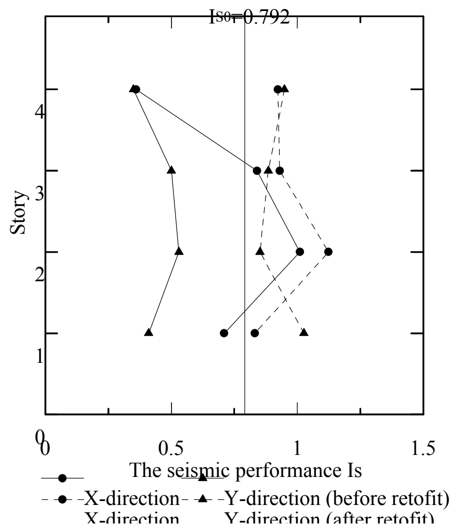
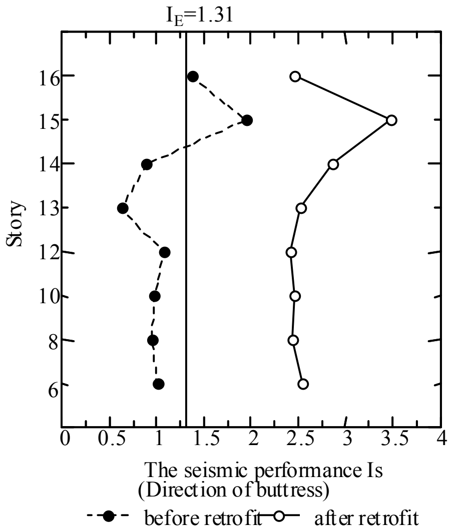


Fig 3 Is value before and after retrofitting of the lower stories



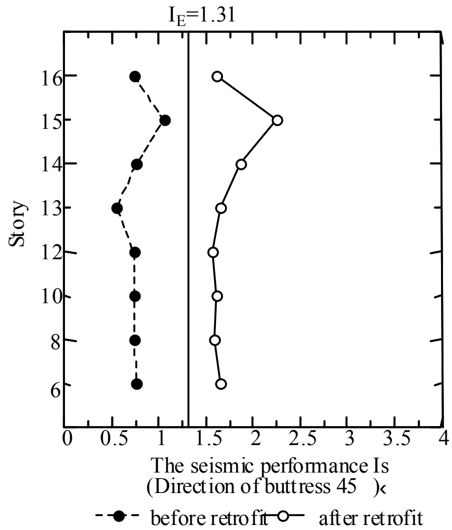


Fig 4 Is value before and after retrofitting (Outer tower buttress direction)

Fig 5 Is value before and after retrofitting (Outer tower buttress 45 deg. Direction)

Low Stories Section:

A seismic capacity (Is: Structural Seismic Index) of the lower stories was calculated in compliance with a secondary diagnosis of an "Evaluation of Seismic Capacity For Existing Reinforced Concrete Building, Revised Version [4]." The Structural Seismic Judgement Index (I_{SO}) for determining seismic performance will be I_{SO} = 0.792 by considering the Standard Structural Seismic Judgement Index (E_S=0.6), Ground Index (G=1.1) and Usage Index (U=1.2). The result of calculation for Is is shown in Fig.3. The value of Is is within the range of 0.351 – 1.013 for ridge direction (X) and within the range of 0.359 – 0.534 for span direction (Y), and lower than the I_{SO} value in direction X for the 1st and 4th stories, and in direction Y for all stories.

RESTORING PLAN

We evaluated the seismic performance of the buildind. As a result, we judged some seismic strengthening required in addition to reconstruction and repair of the damaged part. The defined targeted retrofitting level is same as the seismic performance of any structure constructed under the current seismic standard or higher. Restoring plan for each section are as follow:

Tower Section:

Since the inner tower sustained severe damage and/or collapse around the 13th story, the determination was for reconstruction rather than repair and retrofit.

Additional reinforced concrete placing, reinforcement by steel plate wrapping, reinforcement with affixing of carbon fiber sheet, etc., are planed as strengthening methods. Among these methods, additional reinforced concrete placing was anticipated to provide considerable stiffening effect by increasing of wall thickness and amount of steel reinforcement even though it has the problem of weight increase. In addition, the ease of working on reinforcing materials and the overall workability of the entire project were examined when choosing a methods of reinforcement work since the surface to be reinforced had a three-dimensional surface. From that evaluation, suitability of reinforcement with affixing of carbon fiber sheet or additional reinforced concrete placing was considered better than reinforcement by steel plate wrapping.

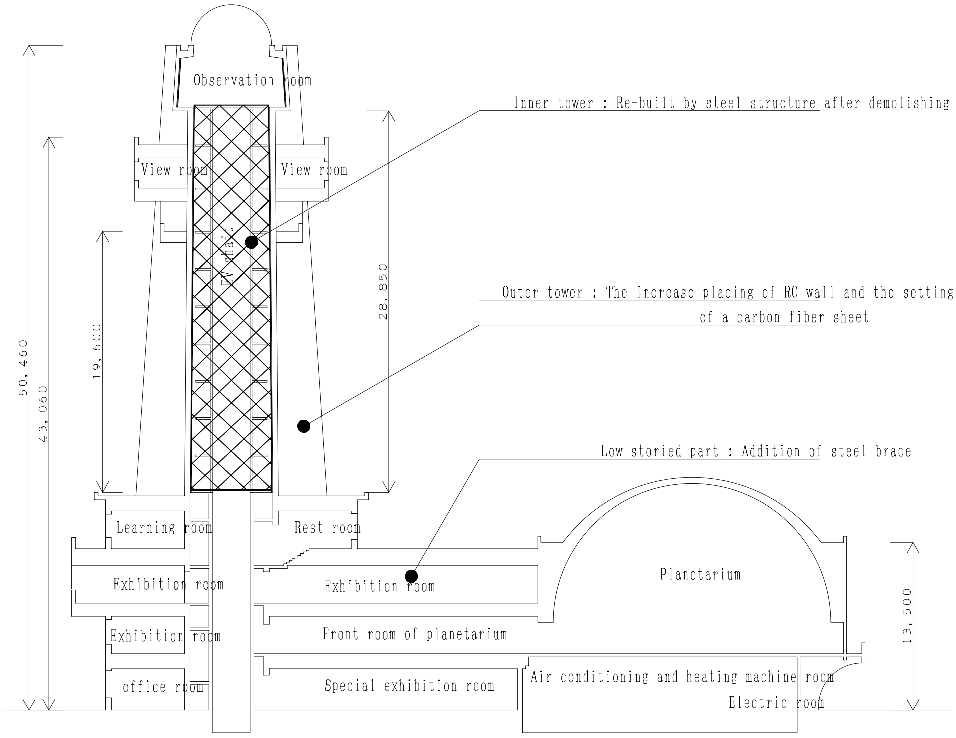


Fig.6 Outline of the restoration work

Low Stories Section:

As a reinforcement method for the 1st to 4th stories of the lower raised section, addition or new installation of reinforced concrete wall, new installation of steel framed brace were examined for the purpose of inereased load – carrying capacity. Since addition of reinforced concrete wall creates weight increase and it is difficult to secure sufficient open space such as exhibition rooms even after reinforcement, the retrofittng method of new installation of the steel framed brace was considered the mostly suitable option.

OUTLINE OF RESTORING WORKS

Inner Tower Section:

The part of the 6th and higher stories of the inner tower had been demolished and was newly constructed in the steel structure. In addition, it is connected with the outer tower by H-shape steel (H – 125 x 125 x 6.5 x 9) in 8 points per story for avoiding collision with the outer tower in the case of a large earthquake (Fig.7).

Outer Tower Section:

For reinforcement of the outer tower, addition of a reinforced concrete wall on the peripheral surface was adopted. The thickness of the new concrete wall was 12 – 15 cm, and D16 reinforcing bars were structured as single arranged bars in 100 mm pitch both horizontally and vertically (Fig.8). In addition, carbon fiber sheets were fixed onto this surface. For those crossover sites with buttress, carbon fiber sheets were fixed in conducting hammer-setting of the bonded anchors onto the steel plates.

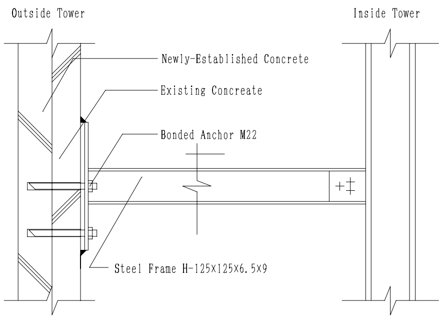


Fig 7 Detail of the connection between the inner tower and the outer tower

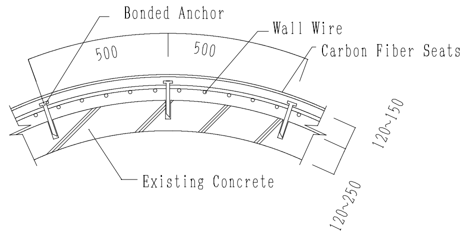


Fig 8 Strengthening detail at the outer tower

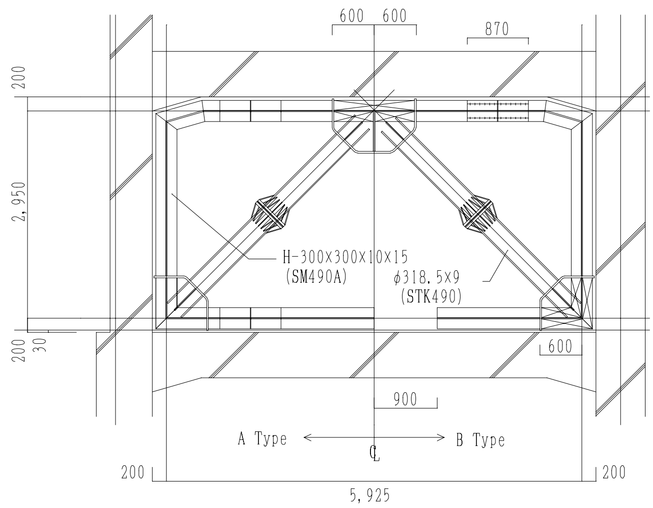


Fig 9 Detail of the steel brace with frame

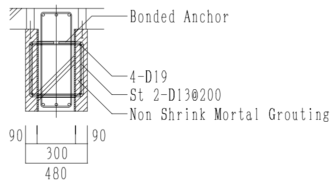


Fig 10 Strengthening detail of the beam

Low Stories Section:

The lower stories section was mainly strengthened by adding steel framed braces (steel pipe: φ318.5 x 9). This steel framed braces were added at 4 spans per story for the 1st, 3rd and 4th stories, and 3 spans for the 2nd story (Fig.9). In addition, reinforcement by additional concrete placing for existing beam was conducted for every part where braces were added since sizes of cross sections of existing beams were small (Fig.10). Other than these, addition of H-shape steel framed brace was conducted for 1st story (1 span) and additional concrete placing to an existing RC wall (1part) at the 2nd story was conducted.

Furthermore, all cracks occurred in the concrete structure of the outer tower and lower stories sections were repaired by filling with epoxide resin.

SEISMIC PERFORMANCE AFTER STRENGTHENING

Outer Tower Section:

Is value after reinforcement is 2.41 – 3.47 for buttress direction and 1.56 – 2.25 for buttress 45-degree, which represents an increase of approximately 1.79 – 4.02 times as large as it was before reinforcement (Fig.4 and 5). Moreover, the outer tower's seismicity is sufficiently secured since the Is value after reinforcement is 1.19 – 2.65 times as large as the Earthquake Index $I \times I_E$ (1.31).

Low Stories Section:

Is value after reinforcement is 0.832 – 1.124 for ridge direction (X) and 0.852 – 1.026 for span direction Y which is approximately 1.11 – 2.64 times as large as it was before reinforcement (Fig.3). Moreover, earthquake resistance of the lower stories section is sufficiently secured since Is value after reinforcement is 1.05 – 1.42 as large as the Structural Seismic Index I_{SO} (= 0.792).

CONCLUSION

1. Mainly, the damage of this building is most severe at its elevated part. Especially, the damage level of the 13th and 14th stories of inner tower was rated as "heavy damage." Additionally, damage for the lower stories section was caused by soil pressure from one side and its damage level was relatively minor.

2. The seismic capacity of the inner tower was the least at the 13th to 15th stories, and the retained flexural capacity of this tower was 25 - 35% of its required resistance against this earthquake. Concerning the outer tower, the 13th floor had the least seismic capacity for both buttress and buttress 45-degreee. In the lower stories section, the 4th story had the least seismic capacity for both directions X and Y.

3. Concerning the tower sections, there was some correlation between the damage level and the seismic performance of each part recognised.

4. In the occasion of the restoration of this structure, the 6th and higher stories of the inner tower were totally removed and constructed with steel framed construction because of the severity of damage of this part. The outer tower was restored with additional concrete placing of reinforced concrete wall and reinforcement by carbon fiber sheet wrapping. Furthermore, the inner and outer towers were connected with each other with H- shape steel towards avoiding collision.

5. Seismic performance of every section of the structure after the reinforcement is secured as such: outer tower: 1.2 times as much as $I \times I_E$ values or more; low stories section: 1.05 times as much as I_{SO} values or more. Thus, it is considered that this restored structure has an seismic performance as high as any structure constructed in compliance with the current seismic provisions.

REFERENCE

1. "Overall Earthquake Resistant Design Code for Government Facilities," Building and Repairing Association, Incorporated company

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