13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1199

SEISMIC ISOLATION RETROFIT OF A PREFECTURAL GOVERNMENT OFFICE BUILDING

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SUMMARY

The main office building of the municipal government of Yamanashi prefecture, Japan, was retrofitted with a seismic isolation system as a measure against earthquake vibrations, which is the first of its kind as a prefectural government office. Strong ground motions from a seismic fault along the Itoigawa-Shizuoka tectonic line, which is one of the largest active faults in Japan and about 15 km distance to the building, were simulated as input ground motions. In addition to the regular seismic design based on the Japanese building code, a performance-based design was adopted to evaluate the retrofitting results. Consequently, it was estimated that the retrofitted building would secure its operability as a municipal government office, even after the largest earthquake. The building was isolated at the intermediate story between the ground floor and the basement. A new temporary supporting system based on post-tensioned units was developed in order to place the isolators efficiently and to secure an enough office space by eliminating the conventional supporting structure, which is inevitably left even after the retrofit work and thus limits the space available. Before the construction started, full-scale experiments were conducted to confirm safety and high performance of the system.

INTRODUCTION

The seismic isolation system of a building is one of the most effective and the most practicable countermeasures against earthquakes, because it drastically reduces its acceleration response during earthquakes. Therefore, when we planed to retrofit the 40 year-old municipal government office of Yamanashi prefecture, we adopted the seismic isolation system not only to improve its seismic safety and functionality, but also to keep its original design. In addition, it is the best method not to obscure its usage as an office building during the retrofitting works.

When we assessed a seismic retrofitting design, it was important to verify the seismic performance against appropriate earthquake ground motions. In our case, we simulated strong ground motions using a hybrid method (Hisada [1]) by considering a seismic fault on the Itoigawa-Shizuoka tectonic line, which exists in

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the vicinity of the site. Then, we evaluated the effectiveness of the retrofitting in accordance with a performance-based design; the building needed to ensure not only the safety but also the operability as the government office building even during and after the largest earthquake.

The seismic isolation retrofit was achieved by setting up seismic isolation devises on the lower ground floor. In order to maintain functions such as restranrant and office at the basement floor, we developed a new temporary supporting system based on a post tension unit, which was able to secure an enough space on the isolation floor after the retrofit (Masuzawa [2]). Before the construction started, full-scale experiments were conducted to confirm the safety and high performance of the system (Yamada and Masuzawa [3]). The seismic isolation retrofit construction was completed in September 2002, through the term of works of about 14 months. To evaluate the effects of the seismic isolation, microtremor measurements in the building were carried out before and after the retrofitting works (Toshinawa et al. [4]). After the retrofitting, Building Research Institute of Japan installed accelerometers in and around the building (Toshinawa et al. [5]).

In this project, in addition to the improvement in the seismic performance by the seismic isolation retrofit, the renovation of the entire building was carried out; building's interiors and equipments were completely renewed, and the external walls and the building frames were repaired. Consequently, it was estimated that about 50 % of the cost was reduced as compared with the cost of creating a new building (Masuzawa [6]).

DESCRIPTION OF BUILDING

The main office building of the Yamanashi Prefecture municipal government is shown in **Figure 1**. This building is designed by Dr. Tachu Naito, who is famous for the inventor of the seismic shear wall. The building is the center of prefecture’s administration including governor's secretariat office, which was build in 1963. It is a reinforced concrete building with eight stories on the ground, one basement and three stories in penthouse. The description of the building is shown in **Table 1**.

Table 1 Description of building

Address	1-6-1, Marunouchi, Kofu city, Yamanashi prefecture, JAPAN
Design Years	1961
Completion Years	1963
Building Area	1,174.20m^2
Architectural Area	10,035.45m^2
Eave Height	28.20m
The Highest Height	37.10m
Structural Kind	Reinforced concrete construction
Structural Type	Frame structure with shear wall
Foundation Type	Spread foundation

Figure 1 East side front panorama

PERFORMANCE-BASED DESIGN

Input earthquake ground motion

When we conduct a performance-based design, one of the most important key parameters is to use appropriate strong ground considering its seismic circumstance. The building site is located at about 15km distance to the Itoigawa-Shizuoka tectonic line, which is regarded as the highest possibility of earthquake occurrence around the area. Seismic fault model is shown in **Figure 2**. We used a hybrid method to simulate strong ground motions, which combined theoretical and empirical methods at lower and higher frequencies, respectively (Hisada [1]). We took into account the south section and a part of the middle section of the tectonic line as the fault model. Accordingly, the estimated fault length is about 70 km, and the corresponding JMA magnitude and moment magnitude are 7.9 and 7.2, respectively. The ground property model assumed nine layer model shown in **Table 2**. S-wave amplification over the bedrock (Layer No. 7) is provided in **Figure 3**. The earthquake ground motions were calculated by two asperity models. We adopted the results showing the largest response spectrum at the site as the input ground motion for the seismic design. **Figure 4** and **Figure 5** show the response spectra and time history waveforms, respectively.

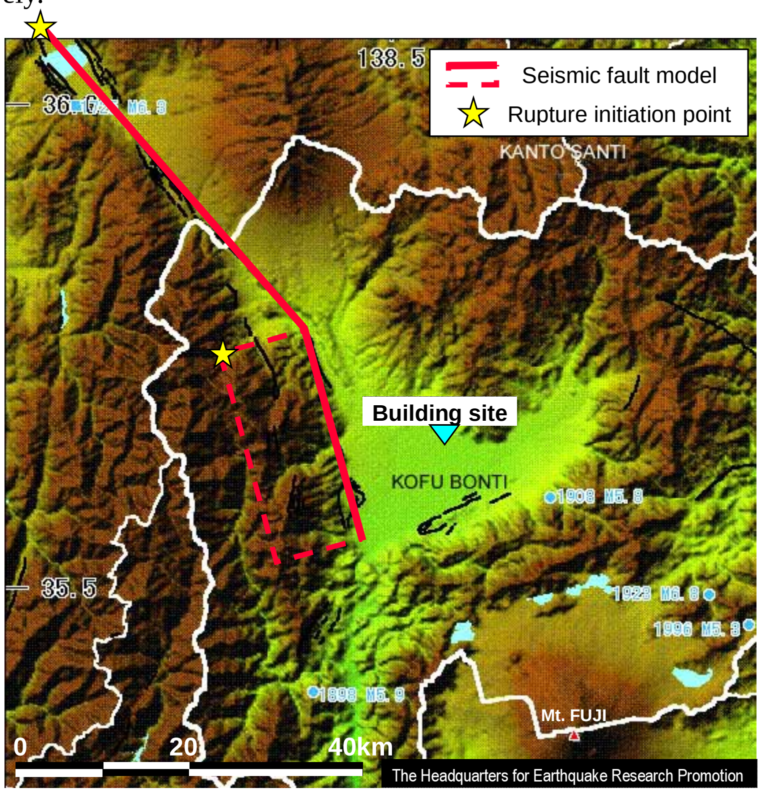

Figure 2 Itoigawa-Shizuoka tectonic line seismic fault model
Original figure: The Headquarters for Earthquake Research Promotion (www.jishin.go.jp/)

Table 2 Ground property model

Layer No.	ρ (g/cm^3)	Vp(m/s)	Vs(m/s)	Qp	Qs	H(m)
1	2.06	2395.0	1166.0	200.0	100.0	2.6
2	2.26	2938.0	1380.0	200.0	100.0	8.4
3	2.32	3535.0	1698.0	200.0	100.0	10.0
4	2.29	3333.0	1504.0	200.0	100.0	10.0
5	2.33	3693.0	1756.0	200.0	100.0	569.0
6	2.50	5500.0	3000.0	200.0	100.0	1400.0
7	**2.70**	**6000.0**	**3510.0**	**300.0**	**150.0**	**17500.0**
8	3.00	6800.0	3930.0	500.0	300.0	10000.0
9	3.20	8000.0	4620.0	1000.0	500.0	—

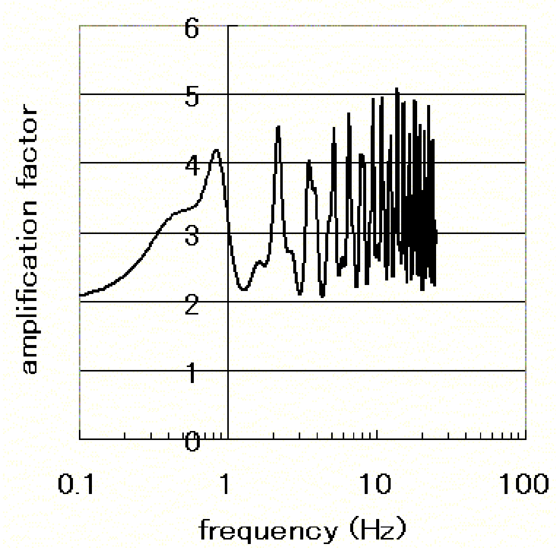

Figure 3 S-wave amplification

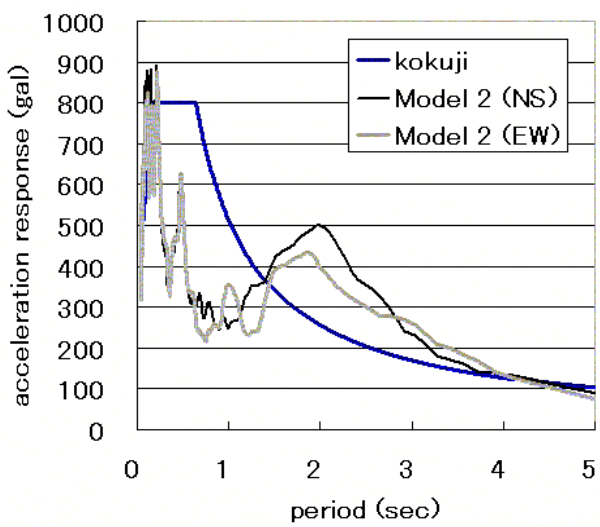

Figure 4 Response spectra

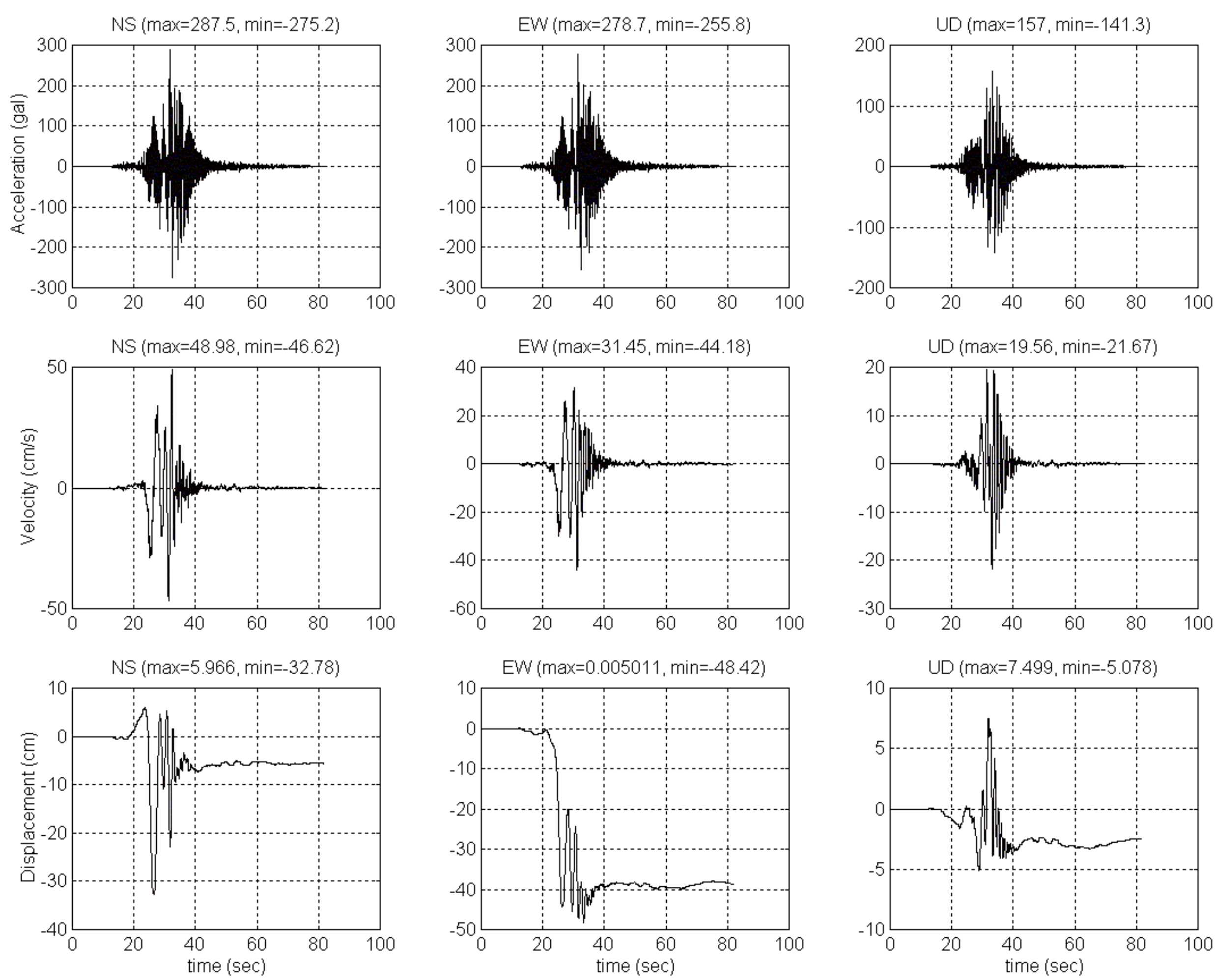

Figure 5 Time history waveforms of acceleration, velocity and displacement

Seismic design

We designed the retrofitting by following the current seismic building code to maintain the functions of the public office building by improving its seismic performance, such as by preventing falls or damages of important facilities like computers and furniture during the large earthquake. Using the load incremental method considering the inelasticity of the structure, we estimated the ductility of the building for the earthquake. As a result, we set the target value of the story deflection angle of the building to be 1/500 radian or less. We assumed the state of the structural member to be less than shear failure with the yield hinges in some boundary girders allowed. We also assumed that the criterion of the seismic isolation device deformation was within a safety deformation level (*i.e.*, shear strain 200 % = 33 cm in this case), and the under structure was less than allowable stresses. To prevent the damage of non-structural members and equipment, we assumed the horizontal response acceleration to be within 200 gal on the office floors (the 1-8th floor). We evaluated the seismic performance of the retrofitted building based on dynamic response analyses using both the input ground motions required by the building code and the simulated ground motions above-mentioned. The result satisfied all the target values. The analytical results are provided in **Table 3**.

Table 3 Results of dynamic response analysis

Level2 (Standard case)		Direction	Input earthquake ground motion					
			Building code (Kokuji1461) / The lower shows the phase characteristic model					Itoigawa-Shizuoka tectonic line
			Elcentro 1940 NS	Taft 1952 EW	Hachinohe 1968 NS	Kobe 1995 NS	Random	
Upper structure	Maximum story deflection angle	X	1/2174 5F	1/2229 7F	1/2692 7F	1/1683 5F	1/2011 7F	1/1509 5F
		Y	1/933 8F	1/1042 8F	1/1148 8F	1/845 8F	1/731 8F	1/1036 5F
	Maximum ductility factor	X	0.148 5F	0.156 8F	0.129 7F	0.191 5F	0.172 7F	0.213 5F
		Y	0.361 8F	0.323 8F	0.293 8F	0.398 8F	0.461 8F	0.323 4F
Seismic isolation device	Maximum horizontal deformation (cm)	X	21.4	18.3	19.4	29.6	14.9	31.0
		Y	21.6	18.3	19.3	30.5	15.2	30.8
	Maximum shear strain (%)	X	130	111	118	180	90	188
		Y	131	111	117	185	92	187
	Maximum base shear coefficient	X	0.110	0.100	0.104	0.132	0.090	0.136
		Y	0.110	0.100	0.103	0.135	0.091	0.136

SEISMIC ISOLATION RETROFIT SYSTEM

Outline of construction method

The building was isolated at the intermediate story between the ground floor and the basement, where it was used for electronic and air-conditioning facilities before the retrofitting. We planned that those facilities were renewed to the rooftop, and the lower floor was converted to a space for restaurant and office. Consequently, the lower floor, which became the seismic isolation layer, required keeping enough space. On the other hand, when the columns of the floor were cut to set up seismic isolation devices, it was necessary to support temporarily the upper building. Because it is very important to secure the safety of a building during the set up, it is common to build permanent supporting columns; they result to reduce the floor space. In order to carry out the seismic isolation retrofit works effectively and to secure enough space by eliminating unnecessary supporting structures after the retrofit, we developed a new temporary supporting system between upper and lower structures using post-tensioned units. The seismic isolation system consisted of forty lead rubber bearings, which were installed to columns, and did not require external dampers. **Figure 6** and **Figure 7** represent the basement floor plan and the cross section, respectively.

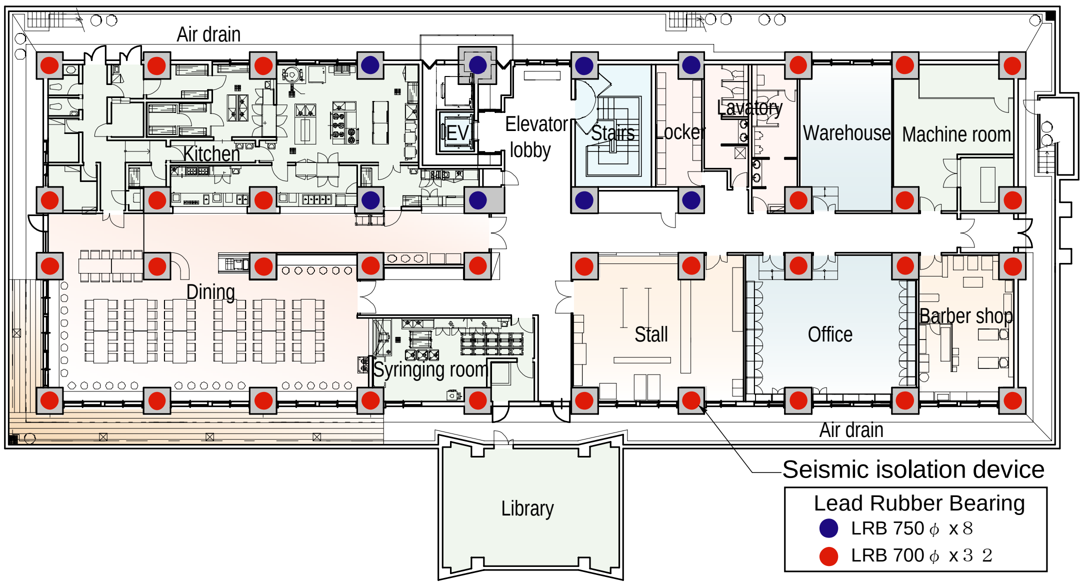

Figure 6 Basement floor plan

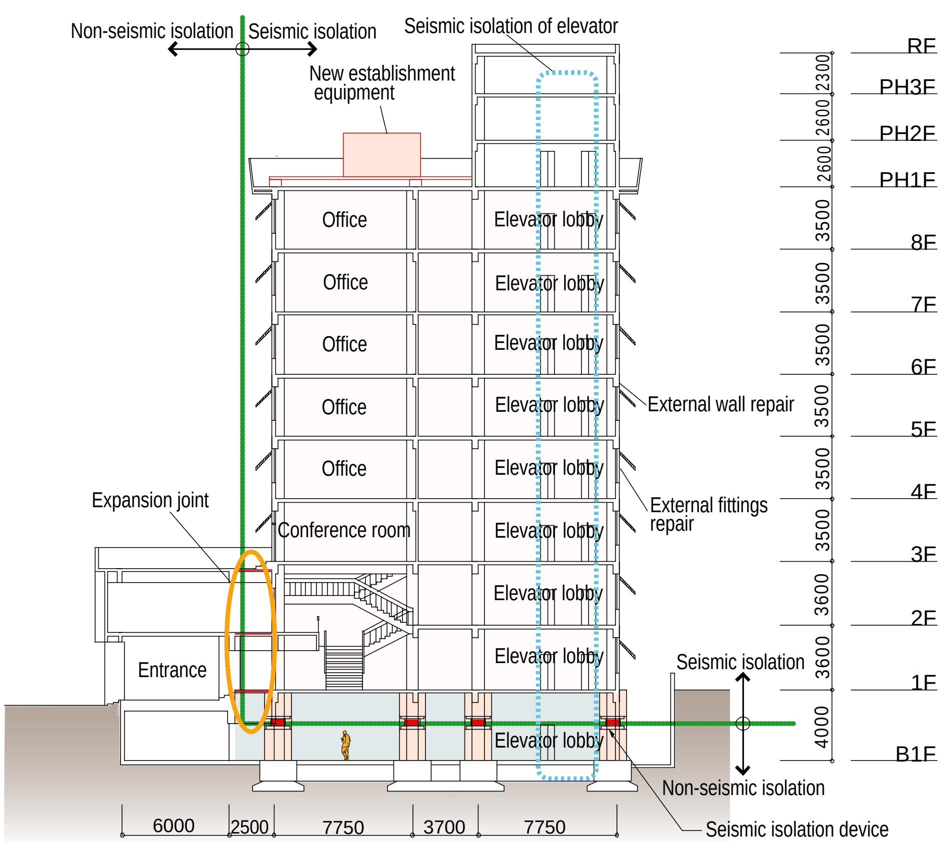

Figure 7 Cross section

Temporary supporting system

We developed the temporary supporting system based on a frictional force using the pre-stress of PC cables. The details of the temporary supporting system are shown in **Figure 8**. The main features of the system are shown below.

①Steel brackets are fixed on RC columns using pre-stress of the PC cables, and the axial forces of the columns are temporarily supported by the bracket using the frictional forces on the bond surfaces between the brackets and the column. Shear cotters are installed on the bond surfaces, and the non-shrink mortars are filled between the columns and the brackets.

②The PC cables, which penetrate the column, are aligned in one direction, and the steel brackets and hydraulic lifters are set in the orthogonal two directions.

③Using the PC strand cable and screw-type anchorage devices, we can expect not only large pre-stressing forces without setting loss, but also easy removal and reuse of the cables.

A big advantage of this construction method is to make both the supporting structure and the construction space compact.

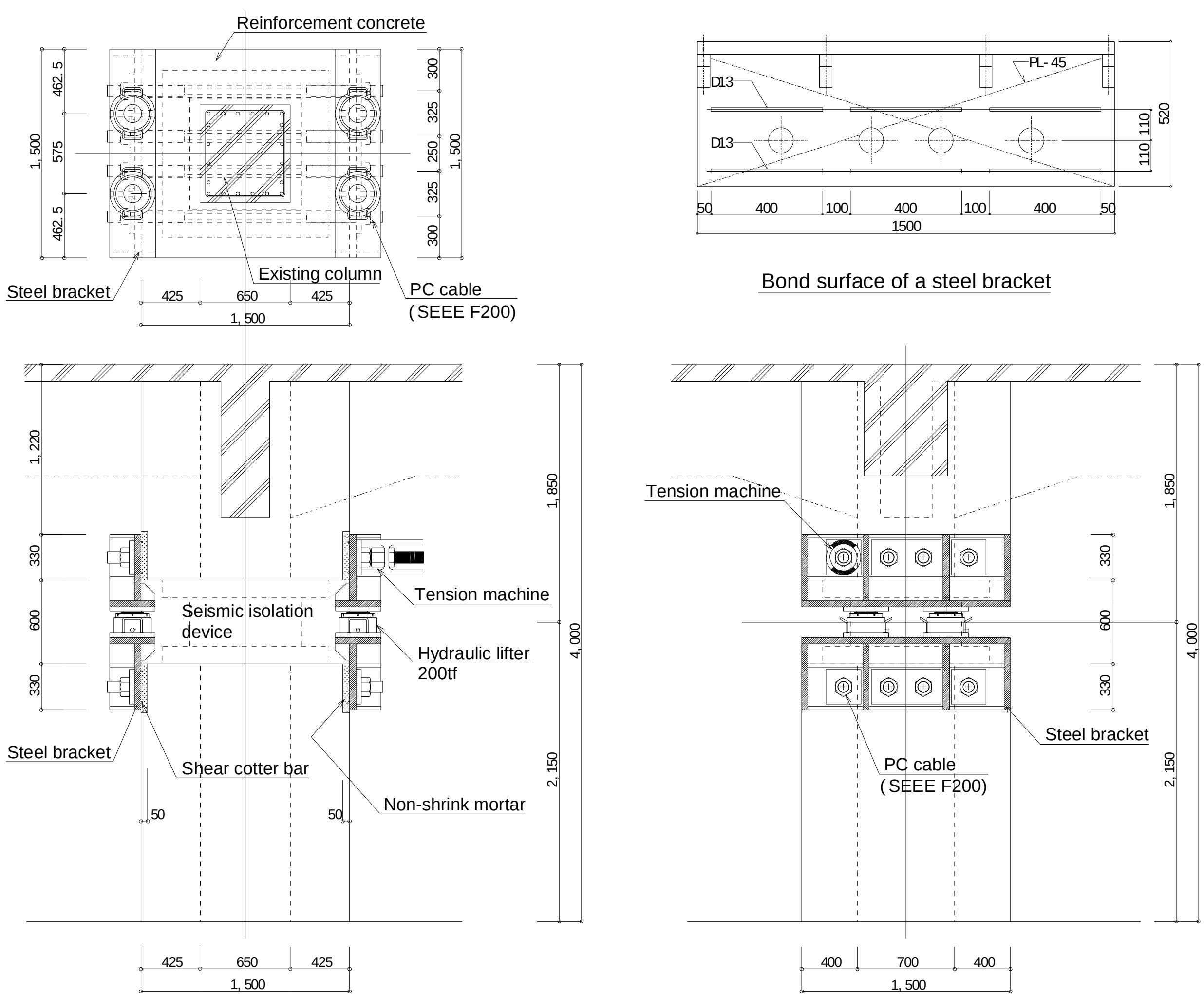

Figure 8 Temporary supporting system (four-cable type)

Full-scale experiment

Before the construction started, we conducted full-scale experiments to imitate the actual construction scheme and to confirm the safety and the support performance of the temporary supporting system. In addition, we investigated the coefficient of the friction and the axial force-deformation relation of the PC cables. Outline of the experiment and specimens are shown in **Figure 9** and **Table 4**, respectively. Since the capital and the pedestal of a column have a same mechanism, only the capital was modeled, and the actual-sized specimen with 1.5 square meters was produced. We made four specimens of two types according to the design axial forces of the columns: two of four-cable type, and two of six-cable type. In order to reproduce the actual construction scheme as much as possible, we made the columns same as the original, and then constructed the reinforcement columns around the columns. **Figure 10** shows the situations of the experiment.

Examples of the load-displacement relations are shown in **Figure 11** and **Figure 12**. In four-cable type, the vertical displacement was about 0.2 mm at 500 tf loading (equal to 150% of the design load), and in six-cable type, that was also about 0.2 mm at 750 tf loading (equal to 150% of the design load). These values are sufficiently small for the temporally construction. The sliding displacement on the bonded surface was not observed until it reached the maximum load, as clearly seen in the load-displacement curves. Also, we conducted load continuation examination for two weeks (a period for temporary support) using specimen No.4 (six-cable type), and confirmed that the sliding displacements on the bonded surface were negligible under the pressure of 120% of the design load. We also confirmed from the loading experiments that the coefficient of the friction was at least 1.86 and more, which agreed with the previous results (Kawamata [7]). Therefore, we concluded that this construction method was very safe.

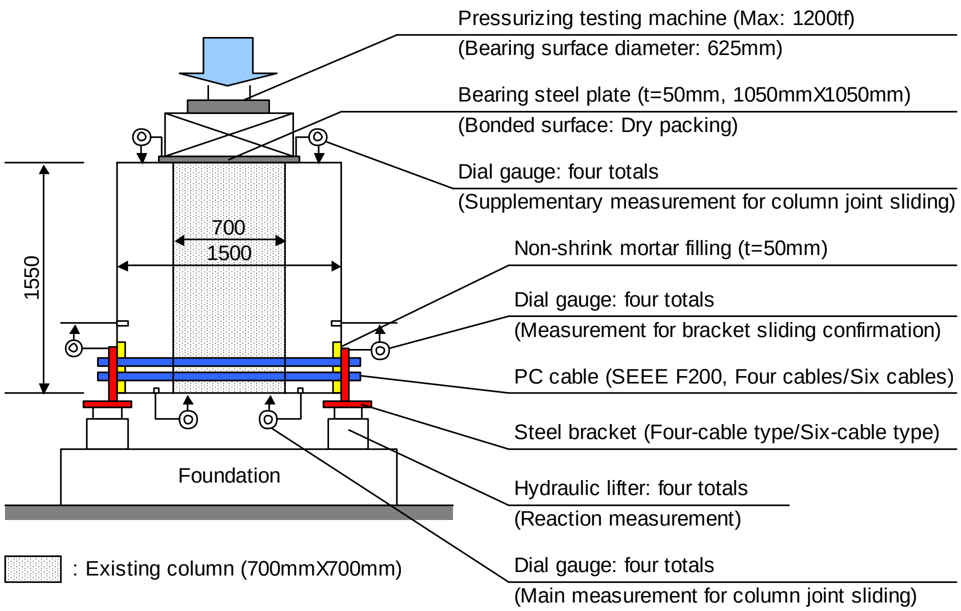

Figure 9 Outline of experiment

Table 4 Outline of specimens

Specimen	Concrete strength		PC cable strand		Design load of column (tf)
	Existing column (kgf/cm^2)	Reinforcement (kgf/cm^2)	Number of cables	Prestress (tf/cable)	
No.1	150	360	4	134	333.3
No.2			4		
No.3			6	134	504.8
No.4			6		

1) Tension of PC cable

2) Loading examination

Figure 10 Situations of experiment (four-cable type)

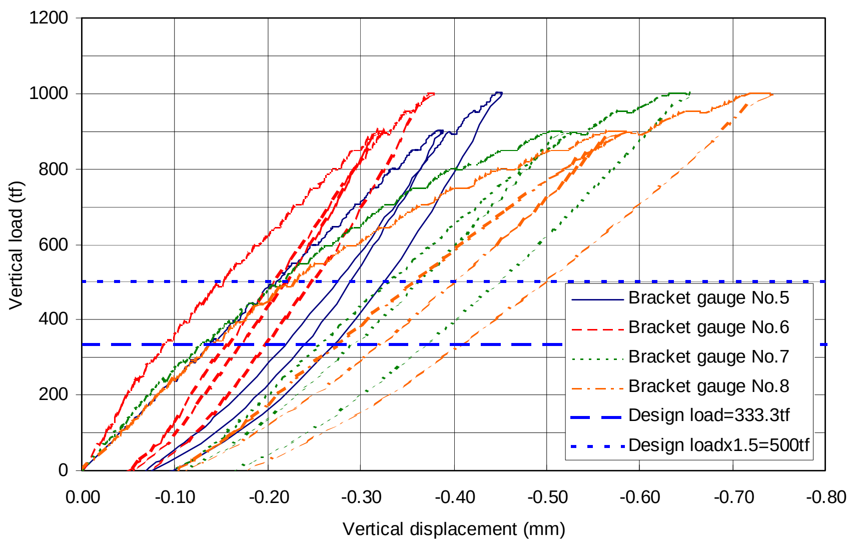

Figure 11 Experimental results (four-cable type)

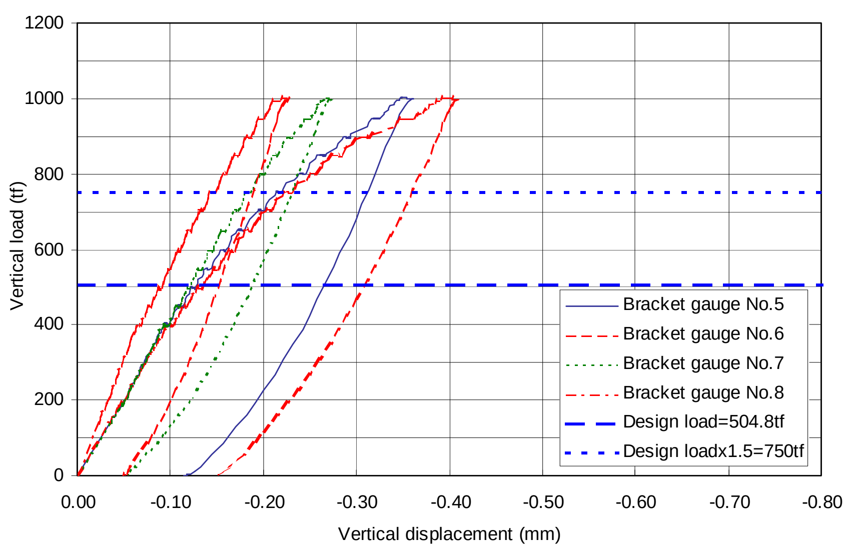

Figure 12 Experimental results (six-cable type)

Construction of Seismic Isolation Retrofit

A plan showing the arrangement of the seismic isolation devices together with the design reaction forces (the axial forces of the columns on the seismic isolation layer) is shown in **Figure 13**, and the construction process of the temporary supporting system is shown in **Figure 14**. We adopted the new supporting method at the capitals of the columns, and used a conventional supporting method on foundation girders at the pedestals of the columns. We used four 200 tf hydraulic lifters on one column as the preloading jacks, and scheduled four sets of the temporary supporting systems and the seismic isolation devices as one unit on the construction process. As shown in the construction flow in **Figure 13**, we rotated the temporary supporting construction system from the X1 axis to the X10 axis, one by one. To support the building during the retrofitting, we also constructed a temporary steel frame structure with braces, which was easy to build and to remove. We always kept the seismic shear coefficient of 0.2 during the construction using the temporary steel structure and other seismic resistant elements.

The reaction forces of the jacks on columns have been measured using pressure converters during construction. We also measured the displacements at 20 points on the capitals of columns using digital dial gauges (1/100mm): three points per column along preloaded lines, and two points in a line immediately after the installation of the seismic isolation devices. 3 mm of the relative displacement accuracy (1/2000 of span) has been allowed as the standard of the temporary supporting construction.

We first loaded up to 120% of the design reaction force in each column following the pressure step divided into eight, and confirmed reasonable reaction-displacement relations. Then, we reduced the pressure down to the designed reaction divided into two steps. Five-minute interval was taken after each step to fix the balance of the preloading columns. When we removed the jacks, we followed the reversed steps of preloading by measuring the vertical displacements in each step. The measurements of relative displacement on the capitals immediately after preloading are indicated in **Figure 15**. In all measured capitals, the measurement displacement values ware less than the permissible value (equal to 3mm).

Reaction pressures during the retrofitting construction and the results of the vertical displacements at a capital are shown in **Figure 16** and **Figure 17**, respectively. Since we set free the axial stress in the column immediately after cutting the column, the pressure of the jack and vertical displacement fluctuated. We speculate that the fluctuations not related to the cutting were probably caused by the change of the live load (increase during working hours) and the load shift by stiffening the grout in the anchorage zone after setting the seismic isolation devices. The maximum vertical displacement during the temporary supporting construction period were also satisfied the permissible value. **Figure 18** shows comparison simulations and observations about axial force of columns. Observations indicate measurement values immediately after cutting of the column. The average ratio of the design pressures with respect to the measurement pressure was 109 % using 36 columns. The values of the ratios were within a range of 93 % - 142 %, which indicated the accuracy of the design. **Figure 19** shows the actual situations of the seismic isolation retrofit works and the completion.

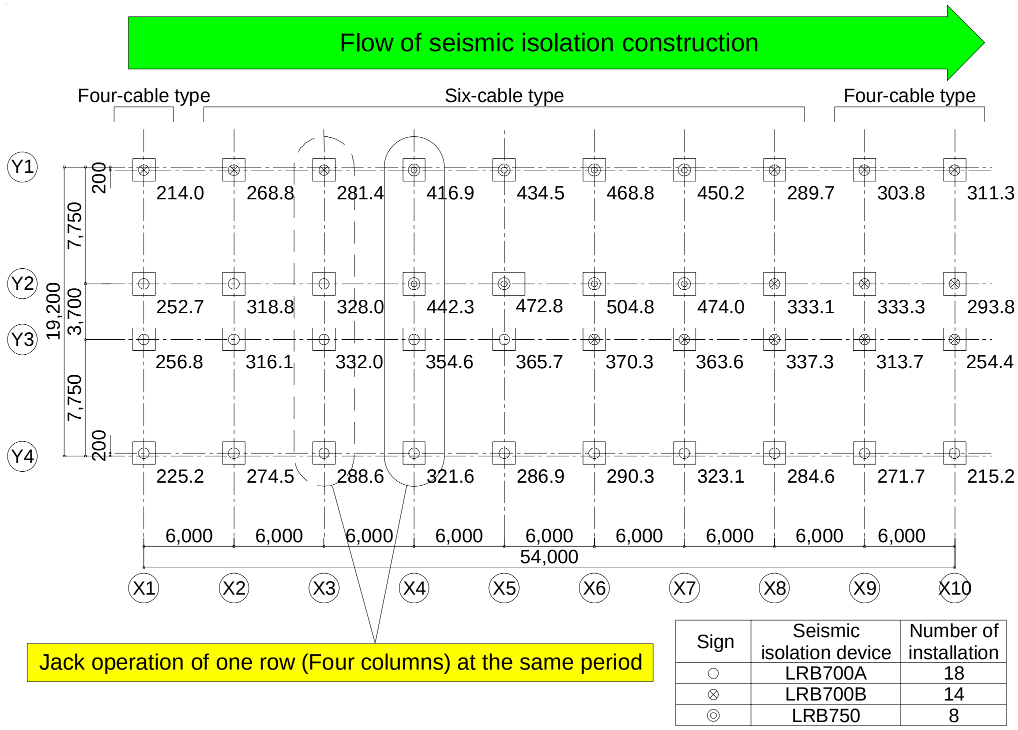

Figure 13 Layout of seismic isolation devices with design reaction force of column (tf)

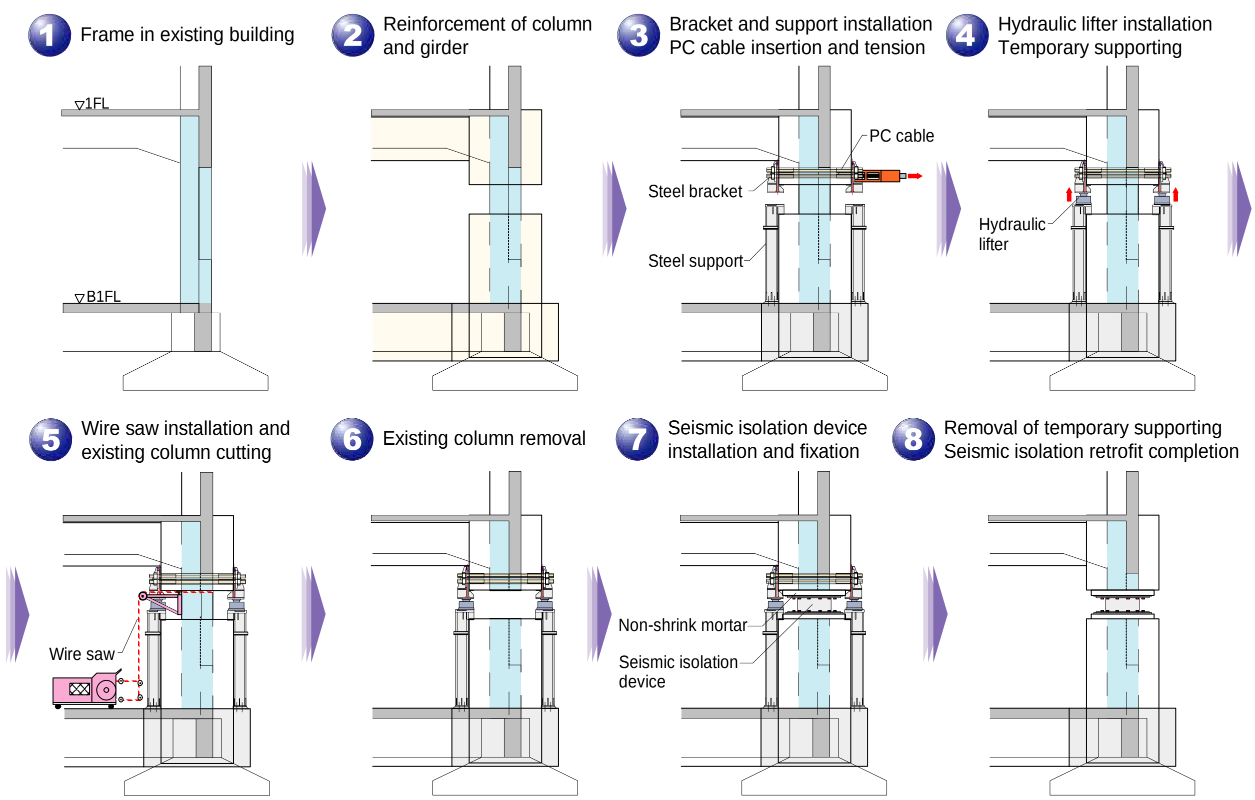

Figure 14 Seismic isolation retrofit construction process

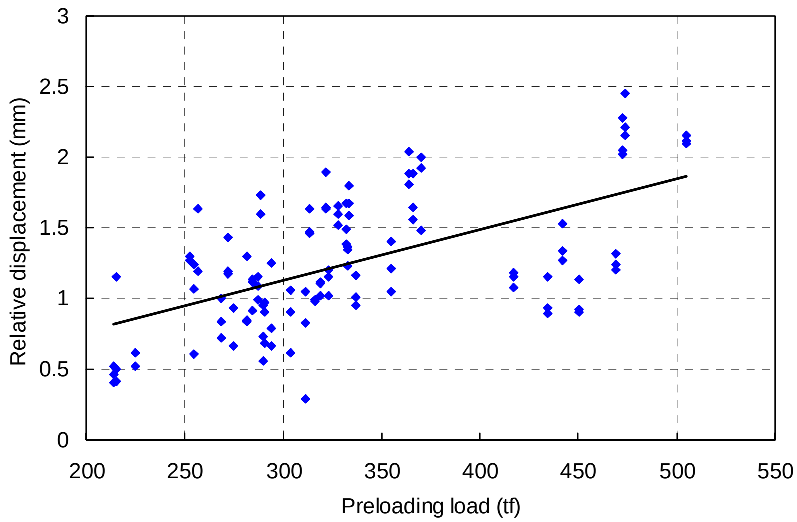

Figure 15 Preloading load (equal to design reaction) - relative displacement relations

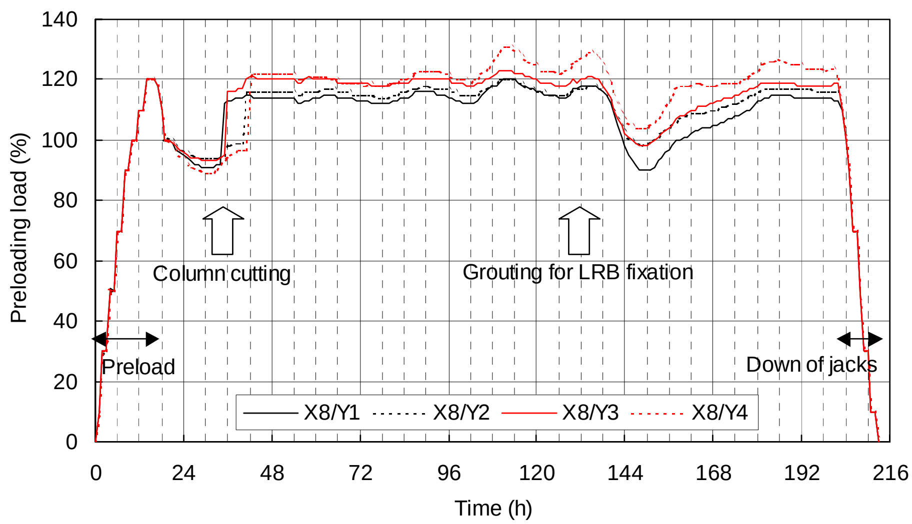

Figure 16 Time history of preloading load

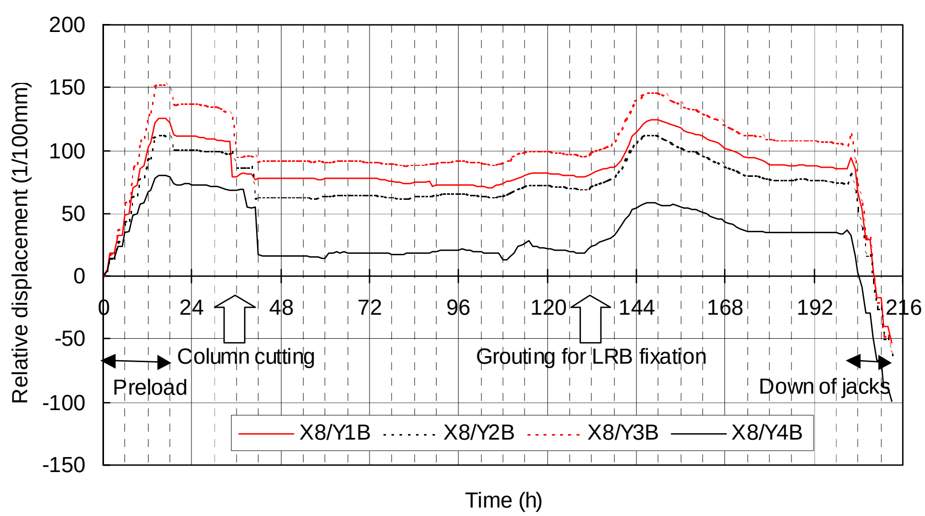

Figure 17 Time history of relative displacement

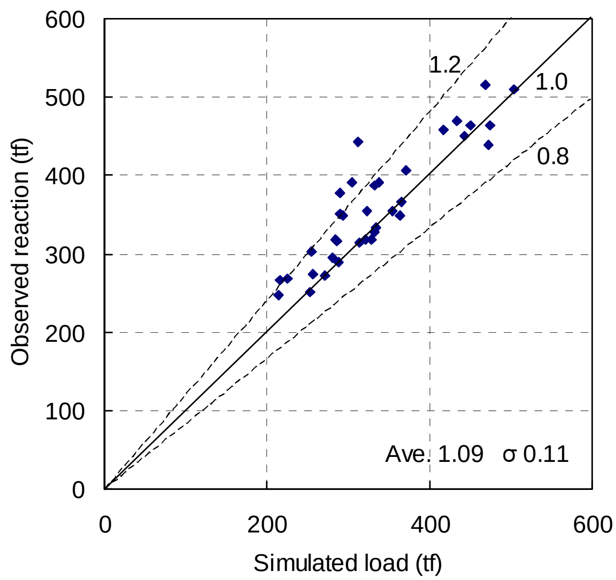

Figure 18 Relation between simulated load and observed reaction

1) Temporary supporting system

2) Seismic isolation retrofit completion

3) Dining after retrofit

4) Seismic isolation device and fireproof panels

Figure 19 Situations of the seismic isolation retrofit and the completion

CONCLUSION

We adopted a seismic isolation retrofit system to the main office building of the Yamanashi prefecture municipal government. We computed the input ground motion from the Itoigawa-Shizuoka tectonic line, which is one of the largest-scale active faults in Japan and is located at about 15 km to the site, and evaluated the retrofitting based on a performance-based design. We concluded that the retrofitted building would ensure not only safety but also the operability as the government office even during and after the large earthquake. The building was isolated at the intermediate story between the ground floor and the basement. In order to construct the seismic isolation retrofit effectively and to secure enough space by eliminating unnecessary supporting structures after the retrofit, we developed a new temporary supporting system using post-tensioned units. Before the construction started, we conducted full-scale experiments to confirm the safety and high performance of the supporting system. From the measurements during construction period, the simulated results agree well with the observations, also the vertical displacement of the capitals satisfied design criteria.

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ACKNOWLEDGMENT

We thank all the members of General Affairs Department of Yamaguchi Municipal Government for their cooperation and understanding through the design and construction periods. Also, we would like to express our gratitude to all the members of the construction work including Yokogawa construction Co., Ltd., which was in charge of the experiment and the construction management.