Restoration of Architectural Stone Heritage Damaged by 2011 Great East Japan Earthquake

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Abstract. The Great East Japan Earthquake of 2011 caused serious damage to a number of architectural heritages. Fukushima Photo Museum (Former Laboratory of Electrics Ministry of Communications), constructed in 1922 and designated as the local governmental important cultural property, was damaged. This heritage structure was categorized into an unreinforced stone structure with local tuff stones. The strong ground motion caused cracks in the stone walls, tilting of the pediment, and falling of the stucco ceiling in the rooms. The multi-disciplinary expert committee for the restoration project was established. In order to discuss the seismic reinforcement of this damaged stone building, structural survey was performed at the first stage of the restoration project. The construction material was the local tuff stone characterized by its rather low specific gravity of 1.2. While the joints were made of cement with insufficient bonding strength, the friction coefficient of the joints was evaluated to be as high as 1.5. Finally, the committee proposed the seismic reinforcement by employing prestressing technique utilizing vertical high-strength steel-bars. The girders on the stone walls were placed by utilizing laminated wood panel of the local products. 3-D finite element model was made to detarmine the prestressing force induced. Furthermore, seisimc safety of the structure was ensured by the static structural caluclation acccording to the Japanese Seismic Evaluation Standard of exsisting RCC building. The microtremore measurement was performed before and after the structural restoration. After completion, the long-term monitoring of stress of the prestressig force induced at the steel-bars has been conducted to check the relaxation of the induced stress to the steel-bars.

Keywords: Earthquake damage · Seismic reinforcement · Stone heritage · Prestressing technique · Monitoring

1 Introduction

The Great East Japan Earthquake of 2011 caused serious damage to a number of architectural heritages. Except for the catastrophic damage caused by the devastating Tsunami, it should be emphasized that the predominant short period of the strong ground motions

gave severe impact to masonry heritage structures. In Fukushima City, Fukushima City Museum of Photography (Fig. 1), constructed in 1922 and designated as a local governmental cultural property, was damaged and had been closed for public. Because of the economic reason affected by the severe accident of the Nuclear Power Plant, Fukushima City, the owner of this architectural heritage, considered dismantlement and removal of this damaged architectural heritage just after the earthquake. However, the volunteers and the experts who understood its historical and cultural values appealed the local government not to be demolished and to be restored for opening again. One of the authors wrote the letter to the local Government to express it would be able to be restored from a structural engineering point of view and should not be removed as a cultural property. The Government finally decided that this architectural heritage would be restored, having established the multi-disciplinary expert committee for restoration. The present case study introduces, as the 10 years story of restoration project from the crisis of demolition to restoration and opening for public, the seismic reinforcement employed by prestressed technique utilizing high-strength steel bars. This architectural heritage was categorised into non-reinforcement structure from a structural engineering point of view. In consideration of the light tuff stone and rather high friction coefficient of the joints, the committee proposed the seismic reinforcement by employing prestressing technique utilizing vertical high-strength steel bars. Furthermore, the girders for structural confinement on the stone walls and the diagonal struts for enduring horizontal stiffness were placed by utilizing laminated wood panel of the local products. Before and after the restration, microtremore meassuments were performed to investigate the fundamental dynamic characteristics. After the structural restoration was completed, the long-term monitoring of the induced tension stress of the high-strength steel bar installed in the stone walls has been conducted for maintemance, as the wooden materials that would be affected by creep phenomenon was utilized for the girder on the stone walls.

Fig. 1. Appearance of building in 1930's

2 Overview of Earthquake and Damage of 2011 Great East Japan Earthquake

2.1 Outlines of Earthquake

On 11th March, 2011, a devastating earthquake of the moment magnitude 9.0(M_w) struck with an epicenter off the Pacific Ocean of Japan from the Tohoku to the Kanto region [1]. This 2011 Tohoku Area Pacific Offshore Earthquake was the largest earthquake to have hit Japan since records began. The huge fault caused the main shock was a low angle reverse fault and was estimated to be approximately 450 km long, approximately 200 km wide with the maximum strike slip of 20 to 30 m [1]. Furthermore, dividing the main rapture area was dividing into three, it was estimated that the depth of the epicenter was 24 km, and the duration of rapture was approximately 3 min [1]. There have been 500 aftershocks exceeding magnitude 5.0 [2]. The seafloor movement of the mainshock generated the devastating huge tsunami, and caused a large number of human loses and missing. Since 1996, the Japan Meteorological Agency has conducted the mechanical measurement of seismic intensity and announced the instrumental seismic intensity. Based on this, regions where the seismic intensity of the mainshock was X or exceeded X under the modified Mercalli Intensity Scale (6 or more under the JMA intensity Scale) were widely distributed from Iwate Prefecture to the Pacific coast of northern Ibaragi Prefecture. According to the seismic records collected by KNET strong motion seismogram network system provided by the National Research Institute for Earth Science and Disaster Prevention, there were approximately 20 stations with peak acceleration exceeding 1.0G. However, those records showed that the short period components was large, and it was reported that the predominant frequency was higher than 5 Hz [2]. In Fukushima City, the strong ground motions at the level of 0.3G, corresponding to seismic intensity 6 (JMA Scale), were recorded by KNET system.

2.2 Damage to Cultural Properties

The human damage caused by the 2011 Great East Japan Earthquake as of September 16th, 2011, was approximately 16,000 dead, 4000 missing and 5,500 injured [2]. According to the Preliminary Reconnaissance Report of the 2011 Tohoku-Chiho Taiheiyo-oki Earthquake published by the Architectural Institute of Japan, approximately 105,000 houses were completely destroyed and 107,000 were partially destroyed. Most of this human and residential damage were caused by the Tsunami. As for the cultural properties, more than 700 damage cases relating to nationally-designated or registered cultural properties were reported as of August 3rd. It was believed that a total number of cultural properties damaged by the earthquake exceeded 1,000. The earthquake damage was caused the tangible cultural properties, intangible ones, fork ones, monuments, cultural landscapes, and groups of traditional buildings. In particular, the earthquake ground motions with short predominant period affected the masonry heritage structures more than wooden buildings.

3   Description of Fukushima City Museum of Photography

3.1   Building Description

Fukushima City Museum of Photography is a 2 stored stone building with eave height of 9.2 m, having been a city-designated cultural property. The stone buildings was constructed in 1922, as Laboratory of Electrics Ministry of Communications. The building has a plan of 25.5 m and 12.7 m in longitudinal and width direction, respectively (See Fig. 2). The stones used for the construction was called Kunimi-stone, a type of volcanic tuff stone. The typical dimension of a stone block is about 55 cm in height, 85 cm in length, 35 cm in depth. The mechanical properties of this tuff stone are introduced in the next section. A cementitious materials were used for the joints of about 5mm thickness. It was not certain whether some kind of connector was used for the joinery, but at least, metal was not used for strengthening. The framework of roof truss was made of wood. The foundation was of reinforced concrete with bricks.

3.2   Earthquake Damage and Process for Restoration

Major damage found after the earthquake included the tilting and the cracks in the decorative pediment above the entrance (See Fig. 2(a) (b)) To support the pediment, the temporary scaffolds were erected as an emergency countermeasure, shown in Fig. 2(a). In addition, visible cracks and the slight movement were found near the southwest corner (See Fig. 2(c)). Inside of the building, there were cracks at the corner of the openings, Some plastering of interior walls fell as well, shown in Fig. 2(d). For safety reason, the museum had been closed since the earthquake occurred. In April 2011, an investigation was conducted by the Architectural Institute of Japan. In November of the same year, Cultural Properties Doctor Dispatch also surveyed the building. In September 2014, Fukushima City, the owner of the building, decided to restore the building to use as a regional cultural asset.

4   Survey for Structural Restoration

4.1   Material Properties Tests

For seismic diagnosis and seismic retrofit studies, mechanical tests of the building materials were performed. This heritage structure composed mainly of local volcanic tuff stone "Kunimi-stone" that was used to construct the structural walls. This stone was characterized by rather low specific gravity of 1.2 with compressive strength of 4.1 (N/mm^2), Young'th modulus of 1.4 (N/mm^2), and tensile strength of 0.37(N/mm^2). In the base and the foundation, another rigid stone and concrete were used, of which compressive strength of 14.9(N/mm^2) and 13.8 (N/mm^2), respectively. In addition, bricks were utilized at some parts of the building. It should be emphasized that seismic performance was significantly affected by the joints between stones of the walls. In the present study, laboratory shear test was performed by using the specimens sampled by the core boring from the walls. As the shear strength was affected by the normal stress caused by the dead weight, Fig. 3 shows relation of strength and normal stress. Portland
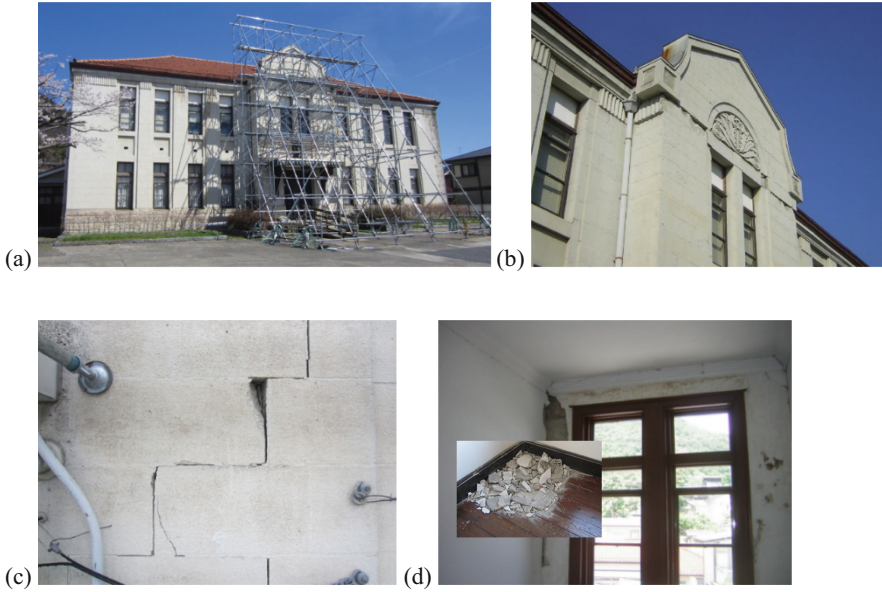
Fig. 2. (a) Appearance after earthquake (b) Tilting of pediment (c) Cracks at SW corner (d) Falling of plaster finishing

cement was already used widely in Japan during this period of the construction, it was believed that the cement was used for the joint materials. However, the bonding effect of the joints was too small to evaluate the strength, as shown in Fig. 3. On the other hand, the friction coefficient was evaluated to be approximately 1.5, because the surface of the stone that touched the joints was rather rough.
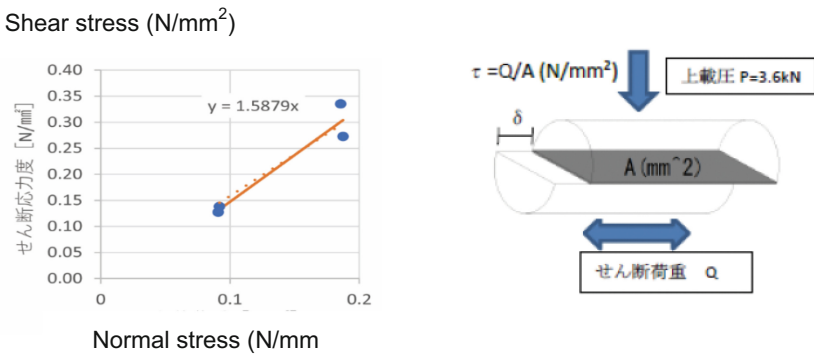
Shear stress (N/mm²)

Fig. 3. Shear tests of joint

4.2 Microtremore and Earthquake Monitoring

Before the restoration operation of this damaged architectural heritage started, microtremore measurement was performed by utilizing 12 velocity sensors. The natural frequency was evaluated to be 4.6 Hz in EW direction. On the other hand, in NS direction, it was evaluated to be 3.5 Hz and 4.5 Hz in the east wall and the west wall, respectively. It was considered that this difference was caused by the structural condition that the east side wall had much more openings than the west side. These fundamental dynamic characteristics were compared with those measured after the seismic reinforcement, shown in Sect. 6. Earthquake monitoring utilizing 3 MEMS accelerometers (3 components sensors) was also performed for 6 months during May 16, 2017 and December 8. During this period, a total of 16 earthquake records were obtained. Those earthquake records were compared with the dynamic structural analysis utilizing simplified SR model (Fig. 4).
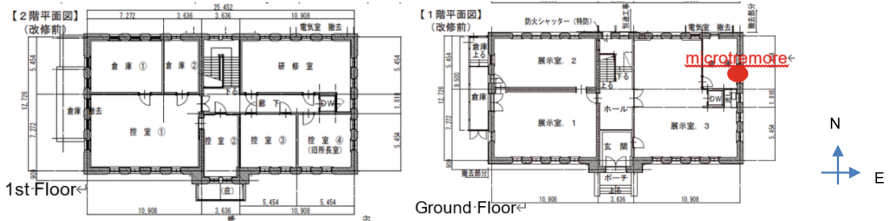

Fig. 4. Plan (upper: 1st Floor, lower: Ground) floor)

5 Seismic Reinforcement

5.1 Seismic Loads for Designing Reinforcement

After the structural restoration was completed, a major aftershock that occurred on Feb. 13, 2021 with magnitude of 7.3(M_j) struck this building. Table 1 shows the outline of the mainshock and this aftershock, related to the present study. Magnitude, peak acceleration and intensity scale (Japan Meteorological Agency Scale) recorded in Fukushima City are compared in this table. In addition, the simulated ground motions compatible with the strongest earthquake provided by Japan Building Code were also utilized in the present study. As the spectrum was given at the engineering bedrock, surface soil response should be taken into account. The amplification due to soil response, corresponding to Type 2, was accounted. Figure 5 shows the acceleration response spectra of those ground motions with damping factor of 5%. It can be noticed that the predominant period of the recorded motions were around 0.1–0.3 s, being very short one, in addition, there finds another peaks around 1.0 s. As masonry structures are, in general, short period structure, the earthquake ground motions generated by 2011 East Japan Great Earthquake might affect structurally masonry buildings. Furthermore, it can be recognized that the intensity of the recorded ground motions was lower than that of the strongest earthquake by Japan Building Code. The intensity and frequency characteristics of one of the most severe aftershock of Feb. 13, 2021 was similar to the ground motions of the mainshock.

For structural designing of reinforcement, both dynamic 3-D FEM analysis and static calculation method defined by Japan Standard for Seismic Evaluation of existing reinforced concrete buildings[3]. The present building was of brick masonry, however, we applied mutatis mutandis of the standard for existing reinforced concrete structure. To conduct dynamic FEM analysis, the following input ground motions were adopted as, 1) Simulated ground motion at the site when 2011 Great East Japan Earthquake (mainshock) occurred. In order to simulate the ground motions, the strong motion records in Fukushima City was utilized, shown in Table 1. Non-linear soil response of the surface layer was taken into account by employing equivalent linearization method where strain-dependent characteristics of soil stiffness and damping factor was considered. 2) According to Japan Building Code, the ground motion that conformed to the response spectra of strongest earthquake motion at the engineering bedrock was synthesized. Non-linear soil response of surface layer was also taken into account. On the other hand, for the latter static calculation used for seismic diagnosis, seismic index of structure *Is*, representing the seismic performance, was introduced for in-plane direction. Here, seismic safety evaluation for out-of-plane direction was performed by the dynamic 3-D FEM analysis, as the prestressing technique was introduced in the present case study.

Table 1. Outline of earthquakes

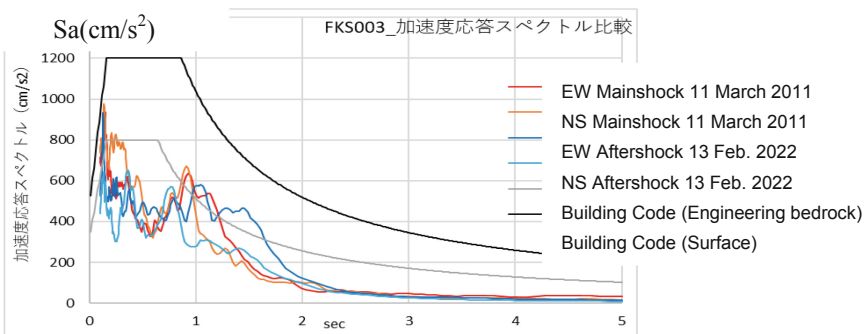| Occurence date	Main shock	After shock
	March 11, 2011	Feb. 13, 2021
Magnitude	9.0(Mw)	7.3(Mj)
Intensity in Fukushima	5.3	5.2
PGA (EW)	0.30G	0.27G
PGA (NS)	0.33G	0.21G
PGA (UD)	0.15G	0.20G

Fig. 5. Response spectra of input ground motions (h = 5%)

5.2 Reinforcement Concept and Decision Making

In order to ensure the seismic safety of this damaged unreinforced stone heritage architecture of stone, the following issues should be solved from an earthquake point of view as (Fig. 6),

1) Stone walls should be reinforced. The strength of joint mortar was insufficient against seismic loads. In particular, reinforcement to ensure safety in out-of-plane direction should be done.
2) In order to connect between the walls, the girder (kerb) should be installed.
3) Horizontal stiffness at the 1st floor and the roof should be endured to transmit the seismic loads to the baring walls.
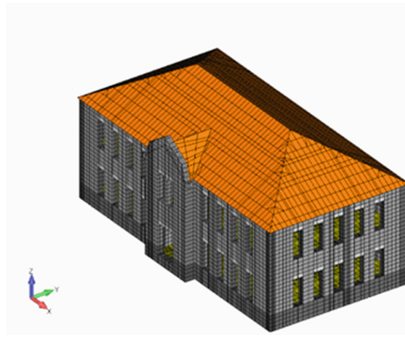4) The pediment should be reinforced so that it would not fall.

Fig. 6. 3-D finite element model

As shown in Fig. 1, the friction coefficient of the joints was as large as 1.5, while the bonding strength was negligibly small. Furthermore, the specific gravity was as low as 1.5. These mechanical condition at the joint suggested that prestressed technique was the most appropriate method to reinforce the stone walls. The expert committee for restoration finally decided to employ the prestress technique for seismic reinforcement. The tensile loading to be induced would be determined by dynamic linear FEM analysis. Figure 7 shows the 3-D analysis model in the present study. The stone walls were modeled by 3-D solid elements. Note that FEM analysis was employed to evaluate the tensile force induced for the prestress technique. In the present study, the elements used were solid elements, which were divided into 6 elements in the cross-sectional direction so that variation in cross-sectional stress could be evaluated more accurately.

The boundary condition of the FE model at the base was fixed. The eigenvalue analysis of unreinforced structural model resulted in the natural frequencies were 3.8 Hz and 4.7 Hz in NS direction and EW direction, respectively.

First, the dynamic response analysis was performed for the case of the ground vibrations due to the Great East Japan Earthquake of 2011. Figure 7 shows the peak normal stress (tensile stress) of the south wall. It can be found in this figure that higher tensile stress was generated at the base stone as high as 600 kN/m², which is lower than the
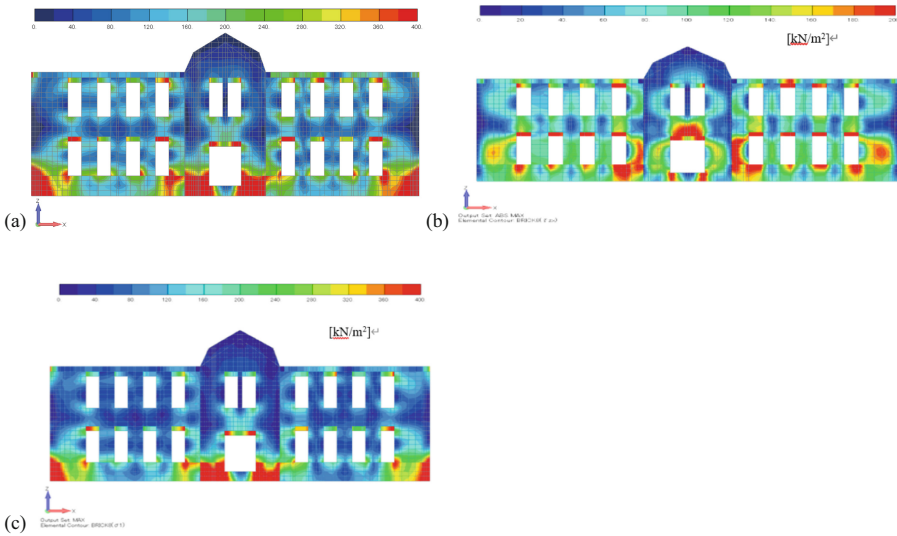
Fig. 7. (a) Peak normal stress induced by simulated ground motions of Great East Japan Earthquake at the site (before reinforcement) (b) Peak shear stress induced by simulated ground motions of Great East Japan Earthquake at the site (before reinforcement) (c) Peak normal stress induced by simulated ground motions of Great East Japan Earthquake at the site (after reinforcement)

tensile strength of the base stone, 1,400 kN/m^2 evaluated from the mechanical material tests (See Sect. 4.1). Furthermore, the tensile stress was generated around the openings, in particular, rather high tensile strength was shown at the lintels. However, as the mean stress was less than 300 kN/m^2, it would not be anticipated that the structure would collapse by the failure of the wall stone. The lintels had enough strength because they were of reinforced concrete. Figure 7 shows the in-plane peak shear stress of the south wall before the reinforcement. The analysis indicated that the shear stress would exceed locally the shear strength, however, it would be caused just in a small area. The stone wall was damaged at the small area near the SE corner (See Fig. 2) by that devastating earthquake, but was not destroyed. The analysis was consistent with the actual damage.

5.3　Implementation of Reinforcement Utilizing Prestressing Technique

A total of 46 high tensile steel bars (φ23 mm in diameter) were placed into the holes after the core boring (φ50 mm). The specification showed that yielding and ultimate strength of the steel bar was 397 kN, 449 kN, respectively. Those bars were modeled by using spring elements, shown in Fig. 8. Described in Fig. 8, the beam elements were utilized to show not only vertical high strength steel-bar but also the steel braces to ensure the horizontal stiffness at the 1st floor, and the wooden horizontal angle brace at the corner to improve the horizontal stiffness at the girder (kerb) level on the walls. Figure 9 shows the elevation in which the high strength steel bars were inserted and tensile force was applied. Un-bond space existed between the hole and the steel bars. At the lower and the upper ends of the bars were fixed with steel plates (See Fig. 8).
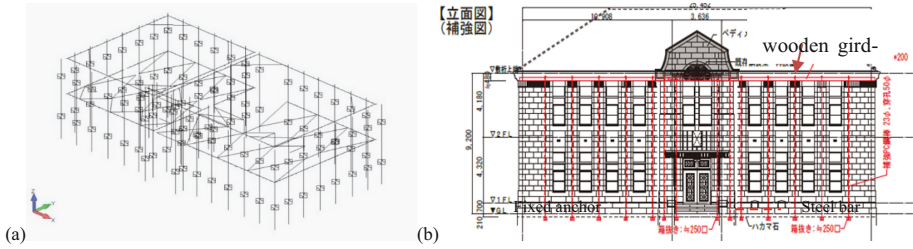
Fig. 8. (a) 3-D Arrangement of reinforce elements (b) Reinforcement of front wall
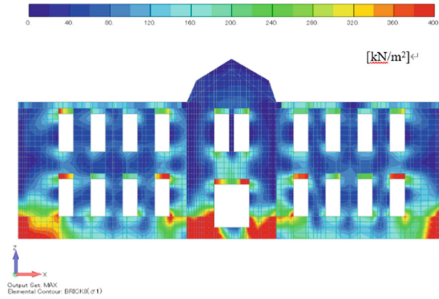

Fig. 9. Peak normal stress after reinforcement induced by strongest earthquake given by Japan Building Code

Fig. 10. Section where peak normal stress shown in Fig. 11

Prestressing force was applied with a jack (See Fig. 7) The force was determined from the seismic response analysis utilizing FEM. In the response analysis, the strongest earthquake ground motions defined by Japan Building Code was used as the input motions. The acceleration response spectrum at the engineering bedrock and the ground surface are shown in Fig. 2. The ground motions compatible to those spectra was simulated. In Japan, even cultural heritage, it is normally applied the seismic safety that is required by the National Building Code. For designing seismic reinforcement employing prestressing technique, the ground motion level was the same one as the National Building Code.

As analysis results, we determined that the prestressing tensile force should be 50kN for each steel bar. After reinforcement with applying prescribed high tensile force to the steel bars, the dynamic response analysis was conducted by utilizing FEM. Figure 9 shows the peak normal stress induced by the strongest ground motion given by the Japan Building Code. The analysis showed that the peak stress approximately 600 kN/m^2 was generated at the base stone, which was enough smaller than the tensile strength 1340 kN/m^2. On the other hand, the peak tensile stress generated in the lintel of the opening was larger than the tensile strength of the local tuff stone of 370 kN/m^2, however, the lintel was of RCC, therefore, it would be judged to be safe.

In order to verify the effectiveness of the prestressing force, the normal stress induced at the section of the walls was described. Figure 10 shows the section level where the peak stress is compared between before and after the reinforcement. The peak normal stress generated by the strongest earthquake ground motion by Japan Building Code is presented in Fig. 11 comparing before and after the reinforcement. It can be recognized in this figure that the reinforcement employing prestressing method is effective in reducing the tensile stress generated during the most severe earthquake. The peak tensile stress induced during the strongest ground motion was evaluated to be 155 N/mm^2, which was enough smaller than the tensile strength of the high strength steel bar of 1080 kN/mm^2.

Output Set: STEP9001
Elemental Contour BRICK8(σ1)

Before reinforcement

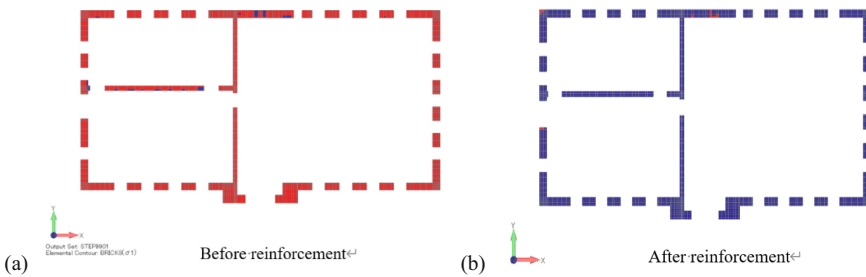(a)   (b)   After reinforcement

Fig. 11. Normal stress generated at 99.19 s for the case of the strongest ground motion by Japan Building Code (a) induced tensile stress exceeded tensile strength of joint (b) induced tensile stress was lower than tensile strength of joint

5.4 Strengthening for Structural Connection

Another important reinforcement was confinement of stone walls, i.e. connection at top (girder) of the walls. In this restoration project, wooden material was used for reinforcement. It was reported that reinforcement using RCC structure might cause the severe damage to the historical stone heritages by the recent devastating earthquake in Italy. Endo. Y [3] indicated the difference in rigidity between the stone masonry and the concrete might cause the collapse of the historical stone church in L'Aquila when 2009 earthquake occurred. In the present restoration project, therefore, wooden material was used in place of reinforced concrete members for strengthening. Laminated woods of local products (Section 500 mm × 200 mm) were utilized here (See Fig. 12) to connect horizontally at the top of the stone walls. Furthermore, the horizontal connection at the

level of 1st floor was endured by steel beam. On the other hand, the horizontal stiffness at the level of the tops of the walls were endured by angle brace of wood. Steel members were utilized to endure the horizontal stiffness of the present stone structure.

5.5 Calculation of Ultimate Lateral Strength

As for the ultimate lateral strength of the structure, the Seismic Index of *Is*, the index for seismic diagnosis of existing reinforced concrete structure in Japan [4], was modified for masonry structures and employed [5] (See Appendix). Table 2 shows the seismic index of Is, which compares before and after the reinforcement. For the diagnosis of existing reinforced concrete buildings, the safety limit of *Is* = 0.6 is recommended for the criteria for judgement. *Is* = 0.6 was evaluated from experience of the earthquake damage and collapse of the reinforced concrete buildings due to past earthquakes in Japan. This table indicates the effectiveness of the reinforcement employed in this practical project.

Table 2. Ultimate lateral capacity represented by seismic index of structure *Is*

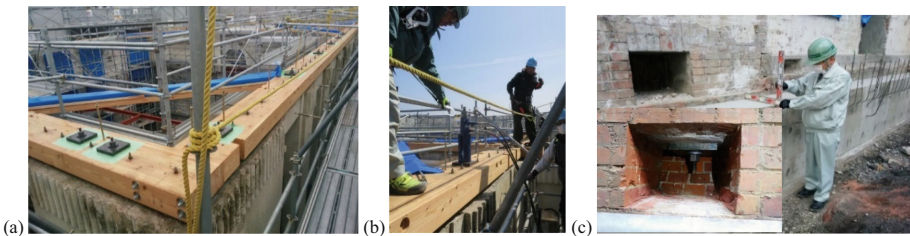|	Before Reinforcement		After reinforcement	
	Ground floor	1st Floor	Ground floor	1st Floor
Longitudinal (EW)	0.32	0.31	2.63	1.06
Transverse (NS)	0.36	0.35	1.88	0.86

Fig. 12. (a) Placement of girder of laminated wood (b) Prestressing work (c) Reinforcement of foundation and anchor of prestressing bar

5.6 Another Reinforcement

As shown in Fig. 2, the pediment just above the entrance was damaged and tilted. It was needed to restore and strengthen it. Reinforcement was done by employing steel frame (See Fig. 13).

Fig. 13. Reinforcement of pediment by steel frame

6 Microtremore Measurement to Verify Reinforcement Implementation

In the present case study, microtremore measurement was done to verify the effectiveness of the seismic reinforcement. Figure 14 compares the transfer function (amplification) direction in EW (out-of plane) direction from the base to the top of the east wall, pointed in Fig. 2, before and after the reinforcement. Shown in Fig. 14, the reinforcement rose the natural frequency and reduced the peak amplitude. This figure demonstrated that the reinforcement increased the rigidity of the structure and would reduce the seismic response. The same effectiveness in dynamic response was found in the other measuring points, a total of 9 points.
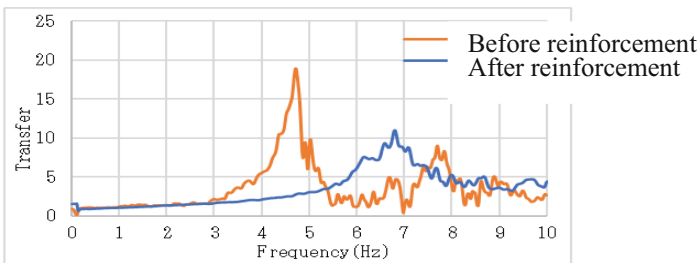

Fig. 14. Transfer function before and after reinforcement

7 Monitoring of Prestressing Force

As described in Sect. 5.4, wooden materials were introduced for the connection of the structure as the girders placed at the top of the stone walls. There is a creep phenomenon in the properties of wood. Also, there is a possibility of such tuff stone. Therefore, the long term monitoring of prestressing force has been conducted to investigate the effect of relaxation of the prestressing force due to creep phenomenon. Shown in Fig. 15, a total

of 4 strain gages to measure the variation of the prestressing force were installed at 2 steel bars. Since material's strain is affected by temperature, the temperature at the holes where the steel bars were inserted has been also monitored as well. The monitoring started at the same time when the prestressing force was induced on April 2, 2020. Figure 15 describes the temperature and the tensile force calculated from the measured strain, respectively. Figure 15 shows that, although the prestressing force was reduced just after the loading and affected by the temperature, it varied with in allowable criteria shown in the prestressed concrete structure standard in Japan. The monitoring has been ongoing to endure the prestressing force.

8 Concluding Remarks

Prestressing method was introduced to the heritage stone structure that was damaged by the 2011 Great East Japan Earthquake, and it was reinforced using wooden girders to endure the structural connection. Microtremor measurement and monitoring of prestressing force were performed to check the effect of reinforcement. This architectural heritage was opened for public in April 2011, 10 year's anniversary. As commemoration of the recovery from the earthquake, the use of local people had begun. No structural damage was caused by the major aftershock of Feb. 13, 2021, described in Table 1, which would demonstrate that the seismic reinforcement introduced in this case study was appropriate.
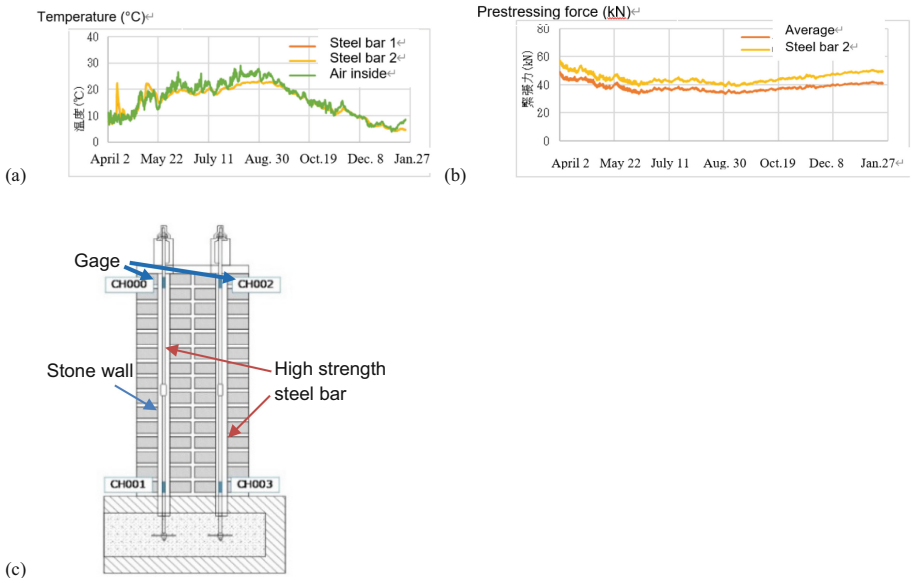

Fig. 15. (a) Strain gage installation (b) Temperature (c) Prestressing force

Appendix

Seismic Index of Structure Is.

On the basis of the Japanese Seismic Evaluation Standard of existing RCC building was modified for brick masonry for each story Is is calculated by the following equation (Hokkaido Building Engineering Association [5]).

$$Is = \frac{Q_u \cdot F \cdot T \cdot S_D}{\sum (W_i \cdot A_i \cdot Z \cdot R_t)}$$

Here,

Q_u: Ultimate lateral load carrying capacity
F: Ductility index (F = 0.6 recommended for brick masonry)
T: Time index (T express effects of structural defects)
S_D: Irregularity index (S_d = 1.0 for regular shape)
W_i: Wight of each story
A_i: value representing a vertical distribution of seismic shear coefficient
Z: Value representing regional seismic activity (Z = 1.0 for Fukushima)
R_t: Value representing vibration characteristics evaluated from structural natural period and soil conditions

The denominator $\Sigma W_i \, A_i \, Z \, R_t$ is lateral shear resistance required by Japan Building Code for designing new buildings.

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