Kumamoto Castle Keeps

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1. Introduction

Kumamoto Castle Keeps were damaged in the 2016 Kumamoto Earthquake and are currently undergoing restoration and seismic reinforcement work. The original wooden Castle Keeps were constructed in 1607; however, they were lost in a fire during the Satsuma Rebellion of 1877. The current Castle Keeps were reconstructed accurately reproducing their original appearance. They were completed in 1960 as part of the commemorative project marking the 70th anniversary of Kumamoto City's municipal establishment.

Before construction begins in 1959   Upon completion in 1960

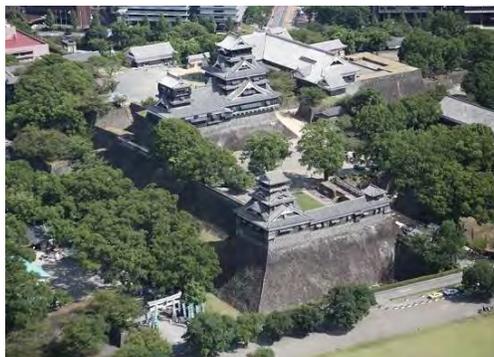Fig.1-1   Panoramic photo of the Castle Keeps

Fig.1-2   Birds-eye view photo of Kumamoto Castle

2. Building Overview

"Kumamoto Castle" refers to the entire castle grounds, with the Castle Keeps located at the highest point within the grounds.

The Castle Keeps consist of the Main Keep (a large tower) and the Small Keep (a small tower), both designated as "museum" use under the Building Standards Law.

The area below ground level, including the stone walls, is designated by the Japanese government as a Special Historic Site under the title "Kumamoto Castle Ruins".

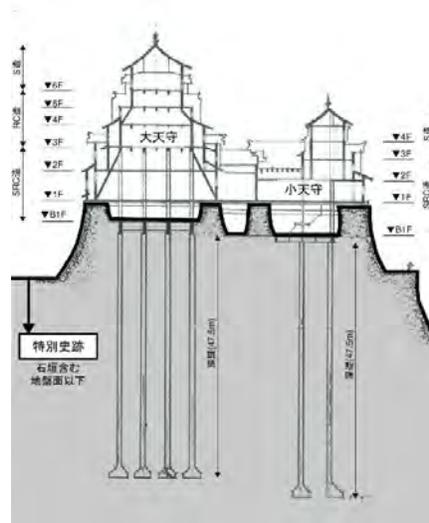Fig. 2-1 shows a cross-sectional view of the Castle Keeps.

Fig.2-1　　Cross-section of the castle keeps

Location:　Honmaru, Chuo-ku, Kumamoto City, Kumamoto Prefecture

Owner: Kumamoto City　　　　　Building use: Museum

Maximum height: 31.2 m

Total floor area: 2,925.28 m2　　Building area: 1,105.36 m2

Number of floors: 6 floors above ground, 1 floor below ground

Structural type: Mixed structure (RC+SRC+S)

Foundation Type: Deep Foundation

Existing Architectural Design: Architectural Art Research Institute

Existing Structural Design: Hattori Makoto Kozo Keikaku Engineering Inc.

　　　　　　　　　(Now: Kozo Keikaku Engineering Inc.)

Existing Construction, Restoration Work, and Seismic Retrofit Work: Obayashi Corporation

Retrofit Design: Obayashi Corporation First-Class Architectural Office

3. Summary of damage caused by the 2016 Kumamoto Earthquake

The epicenter (seismic intensity 7) of the Kumamoto Earthquake was approximately 10 km away from Kumamoto Castle, which is located in the center of Kumamoto City (seismic intensity 6+). The earthquake caused extensive collapse of the buildings and stone walls throughout the castle grounds. Castle Keeps, which were restored in 1960 with a structural design that carefully considered the risks of earthquakes and foundation instability, sustained considerable damage, including the collapse of roof tiles and partial stone wall failure. However, the main structure remained in a condition that can be sufficiently repaired.　Furthermore, the pile integrity test (IT test) documented in the "Castle Keeps Damage Investigation Report" [1], indicated the high integrity of the piles, therefore, the deep foundations supporting Castle Keeps were deemed suitable for reuse.

Before the disaster　　　　　　　　　　　　After the disaster

Fig.3-1 Photos of the site before and after the 2016 Kumamoto earthquake

falling roof tiles　　　　　　　　　　　　collapse of the stone wall

Fig.3-4 Photos of damage to the castle keeps

4. Reinforcement Overview

4-1 Reinforcement Policy

As the area beneath the deep foundation has been designated a Special Historic Site, the installation of new piles or the reinforcement of the existing deep foundation has not been permitted. Therefore, seismic reinforcement was implemented for the superstructure, utilizing seismic dampers to absorb earthquake energy and reduce the seismic forces transmitted to deep foundations. The following seismic dampers were adopted for this project:

①A crossed configuration of Brake Dampers and Oil Dampers on a single structural frame (Cross Damper®)

② Hysteretic-type seismic dampers (Brake damper)

③ Viscous-type seismic dampers (Oil damper)

④ Viscoelastic-type seismic dampers (VE damper)

The reinforcement measure was evaluated using two verification methods.

First, based on seismic diagnosis standards [2], only Brake Damper (item ②) was considered effective in contributing to load-bearing capacity, and the amount of reinforcement required to satisfy Is > 0.6 was determined.

Second, all damper types were considered in the dynamic analysis verification, and it was confirmed that the building would not collapse under seismic input equivalent to 1.25 times the Level 2 ground motion.

4-2 Configuration of the Brake Damper

The Brake Damper is a seismic control device designed to reduce building response during earthquakes by converting building vibrational energy into frictional heat.

It comprises a pair of friction plates (brake material) and stainless-steel plates, which are clamped together within high-strength bolt joints.

This system has been successfully implemented in multiple real-world building projects. [3]

The configuration of the Brake Damper is illustrated in Fig.4-1.
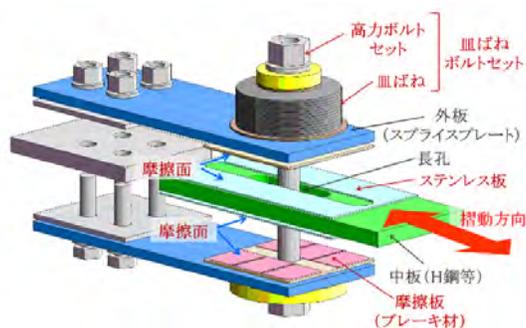

Fig.4-1　The Brake Damper Configuration

The seismic reinforcement method using Brake Dampers[4] has been certified under the Construction Technology Review and Certification system.

It is capable of achieving a ductility index (F value) equivalent to that of conventional steel frame bracing reinforcement. For this project, a ductility index of F = 2.0 was adopted.

5. Seismic Reinforcement Plan

5-1. Overview of Seismic Reinforcement

Owing to the building's distinctive geometry, a variety of seismic reinforcement components have been strategically implemented throughout the structure.

This paper primarily reports on the details of seismic control reinforcement. Explanations regarding strength-oriented reinforcement and other supplementary measures applied

are omitted. Fig.5-1 illustrates the post-reinforcement framing diagram, and Fig.5-2 illustrates the structural perspective view.
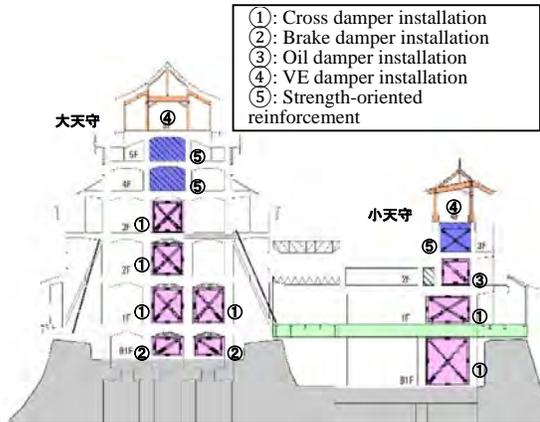
Fig.5-1 Structural reinforcement plan

(entire building)

Fig.5-2 Structural perspective view after reinforcement

(entire building)

5-2 Seismic Dampers with Compact Installation Design

To ensure alignment between the architectural plan and the seismic reinforcement plan, the placement of seismic dampers was carefully optimized.

On the lower floors, Cross Damper® [5], which combines Brake Dampers and Oil Dampers in a crossed configuration on a single structural frame, was employed to reduce the number of reinforced frames and enhance circulation within the exhibition space.

Fig. 5-3 presents an overview of the Cross Damper, Fig. 5-4 illustrates its plan layout, and Fig. 5-5 shows the structural perspective.
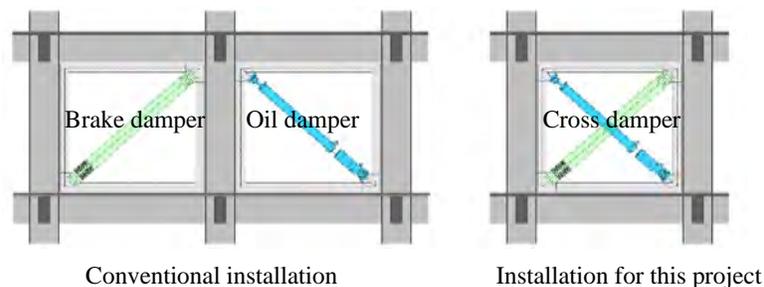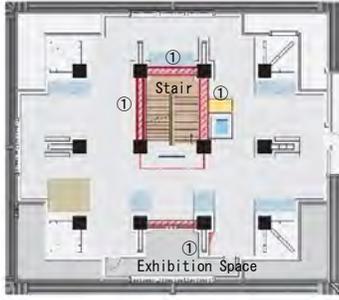

Conventional installation

Installation for this project

Fig.5-3 Overview of cross dampers

① : Position of Cross damper   Cross dampers are installed

                                inside the partition wall of stairwell

Fig.5-4 Floor plan   Fig.5-5 Cross damper structure perspective

(2nd floor of the main keep)   (2nd floor of the main keep)

On the top floor, VE dampers were installed in a strut configuration within the interior finish of the dropped wall, contributing to the creation of an open and unobstructed observation floor by keeping the dampers out of sight. VE dampers absorb seismic energy through the shear deformation of high-damping rubber layers, which are tightly bonded between steel plates and have a thickness of 3 mm. Fig. 5-6 presents the structural perspective, while Fig. 5-7 illustrates the configuration of the VE damper.
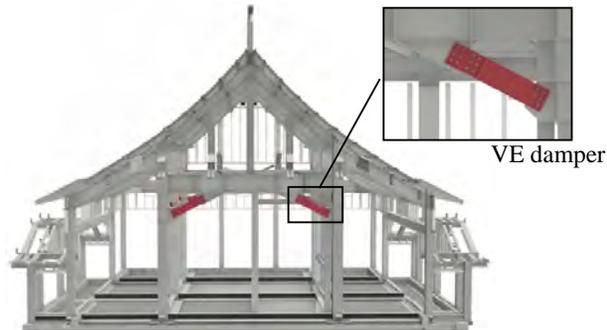

VE damper

Fig.5-6   Structural perspective (top floor)
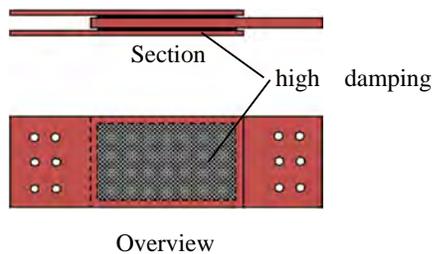

Section

high   damping

Overview

Fig.5-7   VE damper configuration

6 Overview of Dynamic Analysis

6-1 Analysis Model of Upper Structure

An elasto-plastic time history response analysis was performed on the superstructure using a three-dimensional frame element model (Fig.6-1) in which the main structural members were modeled as line elements. Regarding the modeling of each structural member, the ends of columns and beams were modeled using the MS model; the frictional components of Brake Dampers were modeled using a fully elasto-plastic bilinear model; oil dampers were modeled using a Maxwell model that accounts for axial stiffness; and VE dampers were modeled using a Voigt model, in accordance with the method described in Reference [6].

Internal viscous damping proportional to instantaneous stiffness was assigned to each structural member. For existing RC and SRC members, a damping coefficient of 5% was applied, considering the enhanced damping effects expected due to factors such as cracking. In contrast, a damping coefficient of 2% was assigned to newly constructed steel-framed members, including the top floor.
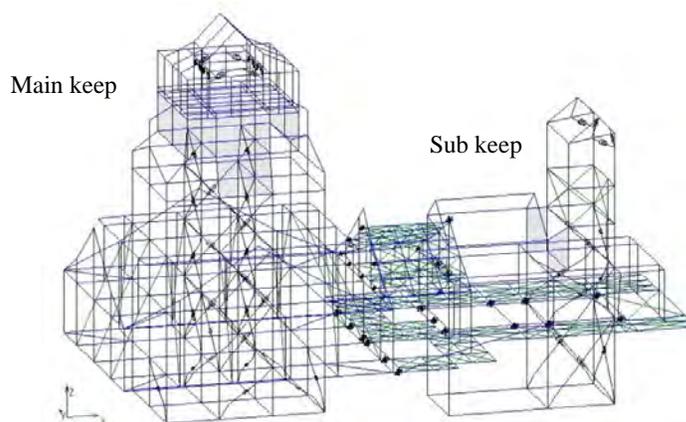

Fig.6-1   Analytical model of superstructure

6-2 Adopted Ground Motion

The adopted ground motion consists of the notification waves and standard observation waves specified in Ministry of Construction Notification No. 1461 of 2000, as well as the Kumamoto earthquake NS component recorded at the Kumamoto Local Meteorological Observatory, which was used as a reference wave for safety margin verification.

In the pseudo-velocity response spectrum (Fig.6-2), the predominant period is observed around 0.7 seconds. This corresponds to the fundamental natural period of the structure prior to reinforcement.

Due to the increase in overall structural stiffness resulting from seismic reinforcement, the fundamental natural period is expected to shorten to approximately 0.3–0.4 seconds, which is anticipated to reduce the input energy.
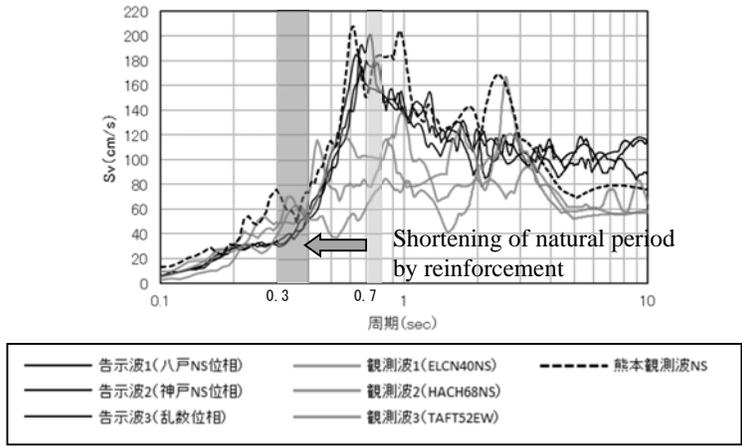
Fig.6-2 Pseudo-velocity response spectrum of adopted seismic motion (h=0.05)

6-3 Response Analysis Results

It was confirmed that the maximum inter-story drift angle during Level 2 seismic motion (Fig.6-3) was less than 1/100, which satisfied the design criteria.

The maximum response acceleration results (Fig.6-4) indicate that a whipping phenomenon can be observed on the top floor of the steel structure.

This is attributed to the change in structural type and is consistent with the observed damage caused by falling roof tiles.

In Section 8, "Performance Verification Test for Roof Tile Fall Prevention Measures," the response acceleration waveform of the top floor (Fig.6-5) was utilized.
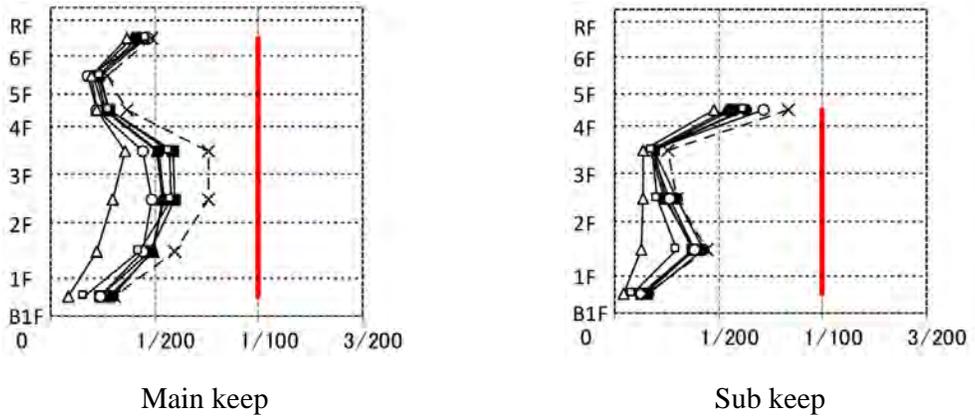

Main keep   Sub keep

Fig.6-3   Maximum inter-story drift angle
(Level 2 earthquake in the Y direction)

Main keep   Sub keep
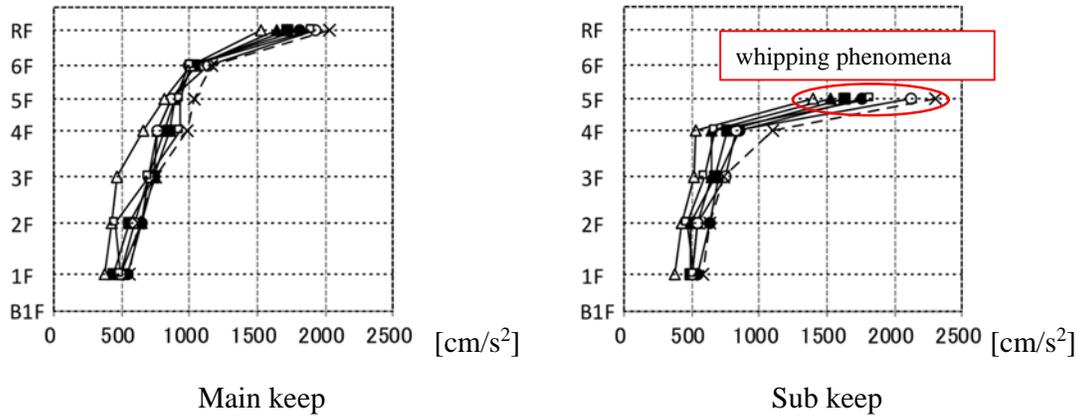
Fig.6-4   maximum response acceleration
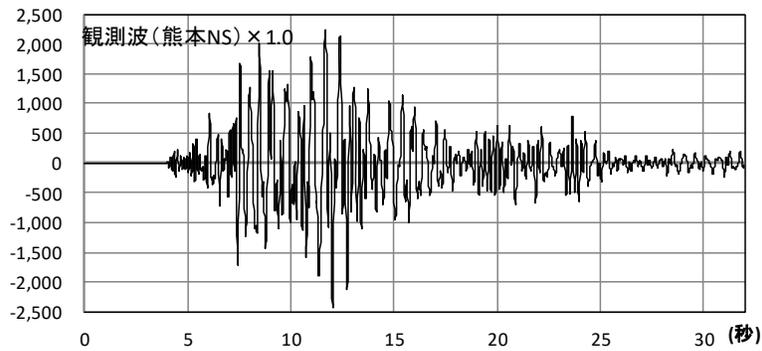(Level 2 earthquake in the Y direction)

Fig.6-5   time history of response acceleration on the top floor of the small keep
(Level 2 earthquake in the Y direction)

6-4 Earthquake Energy Absorption

The effectiveness of the seismic control elements was evaluated by calculating the amount of energy absorbed by each element relative to the input energy during a Level 2 earthquake.

It was confirmed that the seismic dampers absorbed approximately 50 to 60% of the input energy, significantly reducing the amount of energy absorbed by the main structure. (Fig.6-6)
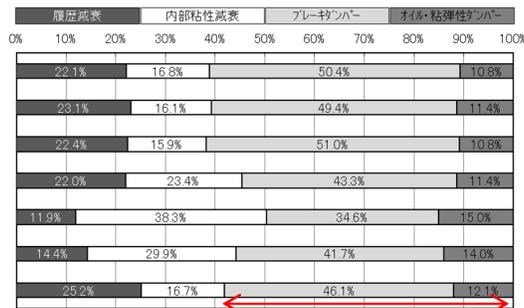
50-60% absorbed by seismic dampers

Fig.6-6 Earthquake energy absorption distribution ratio
by damping element (Y direction)

6-5 Analysis of the Foundation Structure

The deep foundation of this building contains a small amount of reinforcing bars and has a length-to-diameter ratio exceeding 10. Therefore, following the pile design methodology, it was modeled using line elements combined with ground springs (Fig.6-7), and an elasto-plastic pushover analysis was performed. The maximum seismic story shear force of the superstructure and the fluctuating axial forces of the columns were applied as incrementally increasing loads at the top of the deep foundation.

As the foundation is classified as a deep foundation, only shear and axial forces were considered in the analysis, and the absence of brittle failure was confirmed.
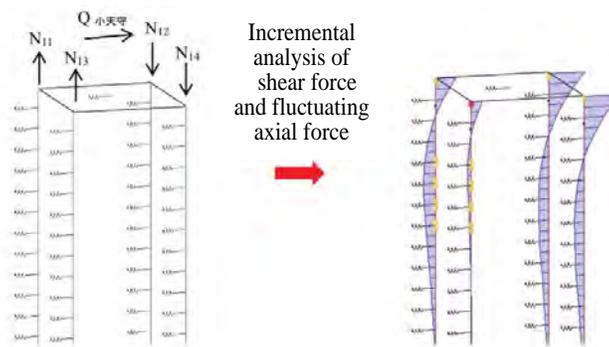

Fig.6-7 Analytical model of the foundation structure (small keep)

A three-dimensional coupled seismic response analysis (superstructure-deep foundation-ground model), which considered the ground conditions resulting from the unique embankment shape of castle structure characteristic, was also conducted. It was confirmed that the dynamic maximum response shear force of the deep foundation—caused by inertial forces from the superstructure and kinematic soil deformation —was approximately half of that obtained from the static pushover analysis.　(Fig.6-8)
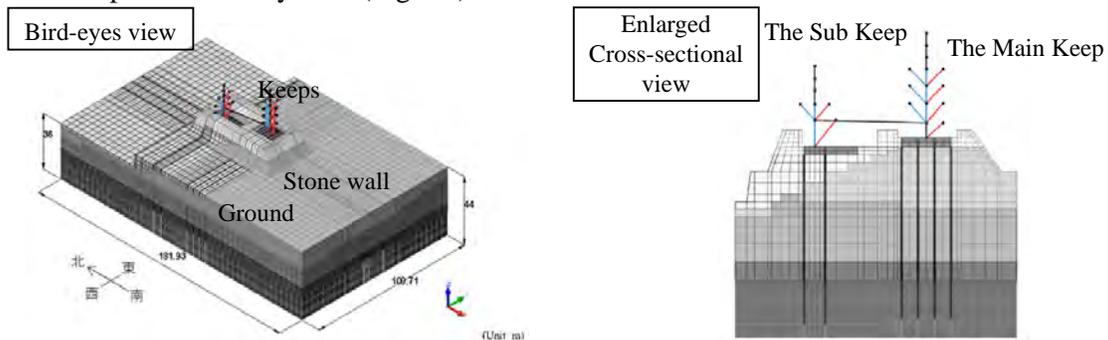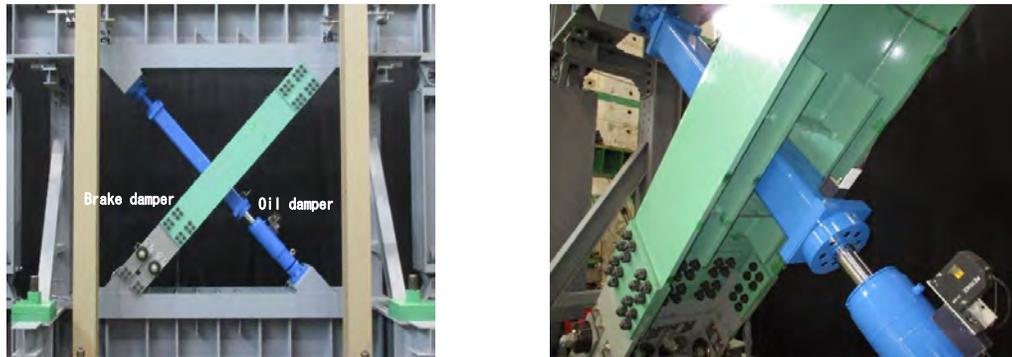

Fig.6-8　　Three-dimensional coupled seismic response analysis model

7. Cross damper performance verification test

In order to verify the structural performance of the Cross Damper®, full-scale dynamic loading tests were conducted using a loading frame as shown in Fig. 7-1.[7]

the penetration part of the damper

Fig.7-1 Cross damper test specimen (full view)

Fig. 7-2 illustrates the load–displacement (axial deformation) relationship of each seismic damper under sinusoidal wave loading conditions (T = 2.0 sec, δ = ±40 mm).

The solid line represents the calculated maximum damping force, while the dashed lines indicate the range of loads within ±10% of that value.

The Brake Damper exhibits perfectly elasto-plastic hysteresis behavior, forming a rectangular hysteresis loop that efficiently absorbs seismic energy.

Due to its velocity-dependent characteristics, the Oil Damper generates maximum damping force within a narrow range of axial deformation near the origin.
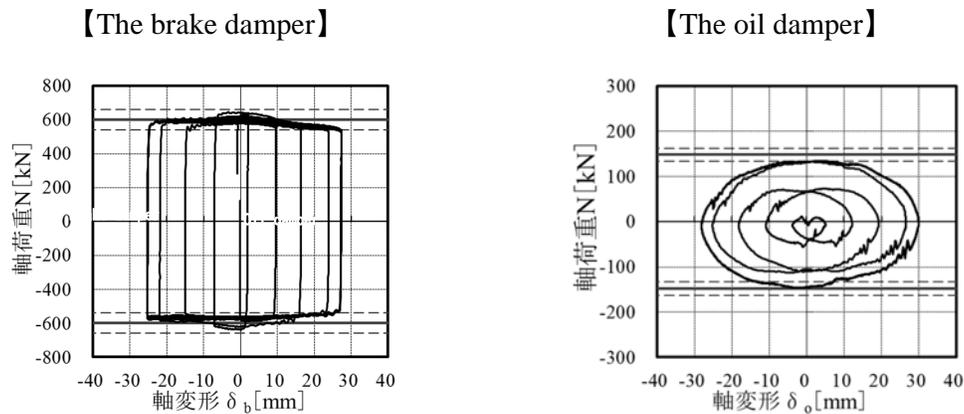
【The brake damper】　　　　　　　　【The oil damper】

Fig.7-2　　Load-displacement (axial deformation) relationship of each seismic damper

Fig. 7-3 presents the horizontal load–displacement relationship of the Cross Damper, based on the combined results described above.
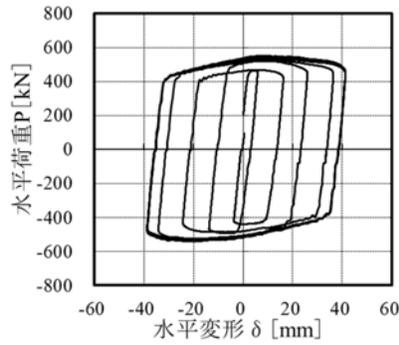
Fig.7-3   Load-displacement (horizontal) relationship of cross dampers

Fig. 7-4 illustrates the variation in energy absorption ratios of each seismic damper with respect to loading amplitude.

The Oil Damper begins to absorb energy even at very small amplitudes (±3 mm).

For amplitudes of ±10 mm or greater, the Brake Damper predominantly contributes to energy absorption. However, as the amplitude increases, the energy absorption ratio of the Oil Damper also rises, resulting in an overall increase in the total energy absorption of the Cross Damper.
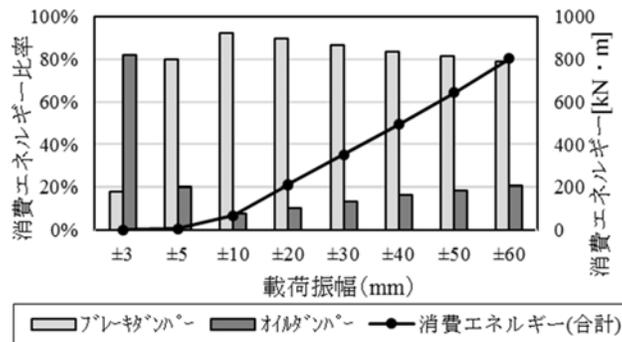

Fig.7-4   Changes in the energy absorption ratio of each seismic damper

8. Performance Verification Test of Roof Tile Fall Prevention Measures

During the 2016 Kumamoto earthquake, roof tiles fell across a wide area. However, it was fortunate that the earthquake occurred during a time when no visitors were present.

For the restoration of the tiled roof, a dry construction method was adopted, in which each tile was individually fixed to the underlying wooden battens as a measure to prevent tile fall (Fig. 8-1).

Prior to on-site installation, a full-scale test specimen was constructed on a three-dimensional shaking table capable of simultaneous excitation in three directions, and a performance verification test of the tile fall prevention measures was conducted. The response acceleration waveform of the top floor, shown in Fig. 8-2, was also used for the excitation input.

Ultimately, the input was increased to a horizontal acceleration of 3G, which is the maximum capacity of the shaking table, and no tiles fell, confirming the effectiveness of the measures.
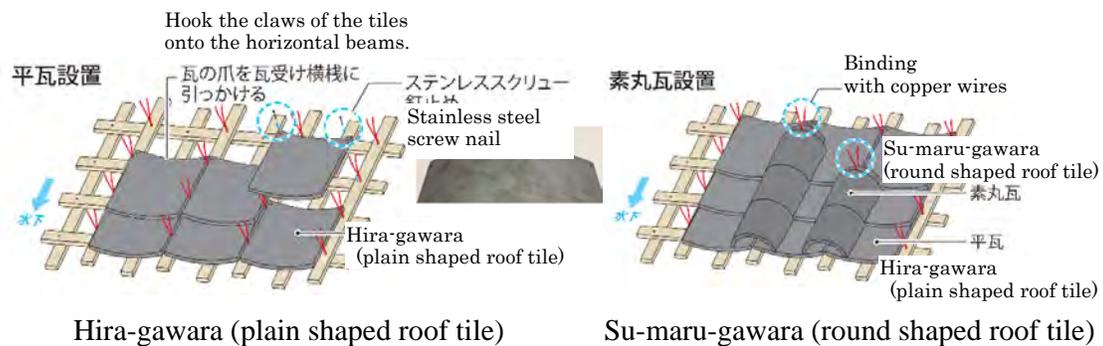
Hira-gawara (plain shaped roof tile)   Su-maru-gawara (round shaped roof tile)

Fig.8-1: Overview of measures to prevent roof tiles from falling

Fig.8-2 Test specimen for measures to prevent roof tiles from falling (full view)

Fig.8-3 Test specimen for measures to prevent roof tiles from falling (roof tile installation section)

9. Conclusion

As a symbol of reconstruction, restoration of the Castle Keeps is progressing with the aim of completion by the end of fiscal 2021.

Since October 2019, a special public viewing has allowed visitors to observe the restoration progress of the Castle Keeps firsthand, attracting large numbers of people.

We express our sincere gratitude to the following parties who made this project possible despite the stringent constraints imposed on buildings within the Special Historic Site:

the Kumamoto Castle Comprehensive Office Architecture Team, acting as the contracting authority, coordinated with the Agency for Cultural Affairs and other relevant organizations; to Professor Emeritus Akira Wada of Tokyo Institute of Technology, who provided structural engineering guidance as a member of the Kumamoto Castle Site Preservation and Utilization Committee; and to the members of Kozo Keikaku Engineering Inc., who were responsible for the structural design during the Showa-era restoration and post- disaster surveys, and who provided invaluable documentations.

We would also like to take this opportunity to extend our heartfelt appreciation to all those who contributed to this project.

Fig.9-1 Panoramic view of the castle keeps (February 2020)

【References】
1)『熊本地震に伴う熊本城天守閣地震被害調査業務委託 天守閣被害調査報告書』(株式会社構造計画研究所：2016)
2)『2017 年改定版 既存鉄筋コンクリート造建築物の耐震診断基準 改修設計指針 適用の手引き』(日本建築防災協会：2017)
3)『多様な要求性能を実現する「ブレーキダンパー®」』(大林組技術研究所報 No.76 鈴井康正ら:2012)
4)『高力摩擦接合滑り用耐震補強工法「ブレーキダンパーを用いた耐震補強工法」』(株式会社大林組/BCJ審査証明 238：2017)
5)『省スペースで耐震性能と制震効果を発揮する「クロスダンパー®」』(大林組技術研究所報 No.82 鈴井康正ら:2018)
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7)『高力ボルト摩擦接合滑りダンパーとオイルダンパーを交差させたブレース型ダンパーの構造性能その 1〜その 2』(日本建築学会大会学術講演梗概集/堂地利弘ら:2018)