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SCALED SHAKING TABLE TESTS SIMULATING THE DAMAGE OF THE SCHOOL GYMNASIUM IN THE 2016 KUMAMOTO EARTHQUAKE

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ABSTRACT

The transverse out-of-plane response of the RC moment frame on the arena floor damaged a school gymnasium that was expected to function as a natural disaster shelter. It was composed of heavy RC substructure and a light steel roof, which partially collapsed, resulting in the facility's permanent destruction in the 2016 Kumamoto earthquake. This study describes scaled shaking table tests that simulate out-of-plane response-induced damage to the RC moment frame of the school gymnasium during the 2016 Kumamoto earthquake and verifies the damage mechanism proposed by the previous numerical simulation and the efficiency of a response control by the friction damper support. Furthermore, the study validates two methods for evaluating the response of the RC moment frame and the overall buckling strength of the roof truss member with semi-rigid joints.

Keywords: *Composite Structure, Gridshell, Shaking Table Test, Seismic Response, Fracture, Roof support, RC substructure*

1. INTRODUCTION

Spatial structures normally used for music or sports are often expected to function as shelters after natural disasters (e.g., a typhoon, deluge, volcanic eruption, heavy snow, and earthquakes). Particularly in Japan, where the entire country is in a high-seismic-hazard zone, school gymnasiums are designated as disaster evacuation facilities, and thus their seismic performance is still being actively debated (e.g. Ministry of education, culture, sports, science and technology, Japan [1]). Transverse out-of-plane response-induced damage by the RC moment frames in medium school gymnasiums composed of heavy RC substructures and light steel roofs, as shown in Fig. 1, has become a popular topic since the 2011 Tohoku earthquake. According to Architectural Institute of Japan (AIJ) [2], the transverse out-of-plane response of RC moment frames in a large earthquake causes subsequent damages such as flexural yielding of the RC columns, tension yielding of the anchor bolts fixing the roof supports, cracking of the base mortar and RC beams caused by the forced displacement of the roof supports on the RC moment frames, and falling of the roof ceiling materials. Furthermore, during the 2016 Kumamoto earthquake (AIJ [3]), serious damage occurred, with the fractured truss members of double-layered metal gridshells falling on the arena floor, thus preventing gymnasiums from being used as disaster shelters indefinitely.

Following the 2011 Tohoku earthquake and the 2016 Kumamoto earthquake, Japanese researchers are actively promoting the understanding of seismic behavior and developing simple seismic performance evaluation and retrofit methods. Yamada, Shimada, and Tomatsu et al. [4] performed static loading tests on the pinned supports to analyze the damage mechanism. Shimada and Yamada et al. [5] proposed an efficient reinforcement method. Furthermore, Ito, Wada and Yamashita et al. [6], [7] carried out the static loading tests of the roller supports to establish the shear capacity formula. Watanabe and Yamashita et al. [8] proposed the backbone curve of the roof supports and performed the nonlinear response history analysis of a gymnasium damaged by the 2011 Tohoku earthquake. Narita and Takeuchi et al. [9] proposed the seismic capacity evaluation method of the gymnasium with pinned supports. Narita, Terazawa, and Inaba et al. [10], [11], [12], [13], [14] proposed a response control method using friction damper supports and verified its efficiency through dynamic loading tests. Terazawa, Inanaga and Takeuchi et al. [15] and [16] performed the nonlinear response history analysis of the gymnasium damaged during the 2016 Kumamoto earthquake to analyze the damage mechanism of the collapsed roof. Terazawa and Kishizawa et al. [17] proposed an overall buckling strength evaluation method for the roof truss members with semi-rigid joints. Terazawa and Nishikawa et al. [18] proposed a simple out-of-plane response evaluation method for RC moment frames. While some of the aforementioned findings and proposals are included in design guidelines (Ref. [19], [20] and [21]), the actual out-of-plane response of the cantilevered RC and the damage mechanism of the metal gridshell roof have not been experimentally observed.

This study reports scaled shaking table tests simulating the out-of-plane-response-induced damage by the RC moment frame of the school gymnasium in the 2016 Kumamoto earthquake. It also verifies the damage mechanism suggested by the numerical simulation [15], the efficiency of the response control by the friction damper support [10] and the evaluation method [18] of RC moment frames, and the overall buckling strength [17] of the roof truss member with semi-rigid joints. Furthermore, it aims to provide international researchers with an overview of the research field on the seismic performance of school gymnasiums, which has previously been reported in Japanese as a domestic topic.

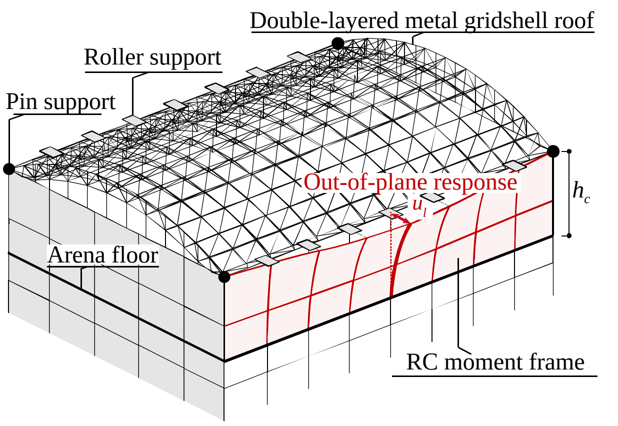


***Figure 1**: Schematic image of the out-of-plane response of RC moment frame*

2. SUMMARY OF THE DAMAGE AND PREVIOUS PROPOSALS FOR RC GYMNASIUM WITH METAL GRIDSHELL

The damage to a school gymnasium caused by the 2016 Kumamoto earthquake and the previous proposal validated in the shaking table tests are briefly summarized in this section.

2.1. Observed damage in the 2016 Kumamoto earthquake

Fig. 2 and Fig. 3 show the drawings and actual damage to a school gymnasium caused by the 2016 Kumamoto earthquake, respectively. According to the current Japanese building code, the target building (referred to as Gym. A hereafter) is a high school gymnasium with a plan size of 34.4 × 45.8 m. Gym. A composed of a RC substructure and a double layered cylindrical space truss structure. The third-floor line (3FL) is the arena floor, and the RC moment frame on the arena floor is connected by roller supports to the roof. As shown in Fig. 3(f), the crack by flexural yielding is observed in the RC column base. In Figs. 3(b) and 3(d), the anchor bolts in the roller bearings were contacted to the end of the slotted hole. Furthermore, the base mortar and RC beams were cracked by the transferred force. The damage to the roof members was concentrated around the roof supports. As shown in Figs. 3(a) and 3(b), the upper chords, overall and local buckling, and post-buckling ductile fracture were observed. As shown in Fig. 3(e), the overall buckling and flexural deformation were observed in the diagonals of the truss. As shown in Figs. 3(a), 3(c) and 3(g), the bolts connecting the lower chords to the nodes were fractured and the lower chords fell on the arena floor.

However, Gym. A was closed after the 2016 Kumamoto earthquake because of the caving roof. Terazawa and Inanaga et al. [15] also provided more insight on this subject.

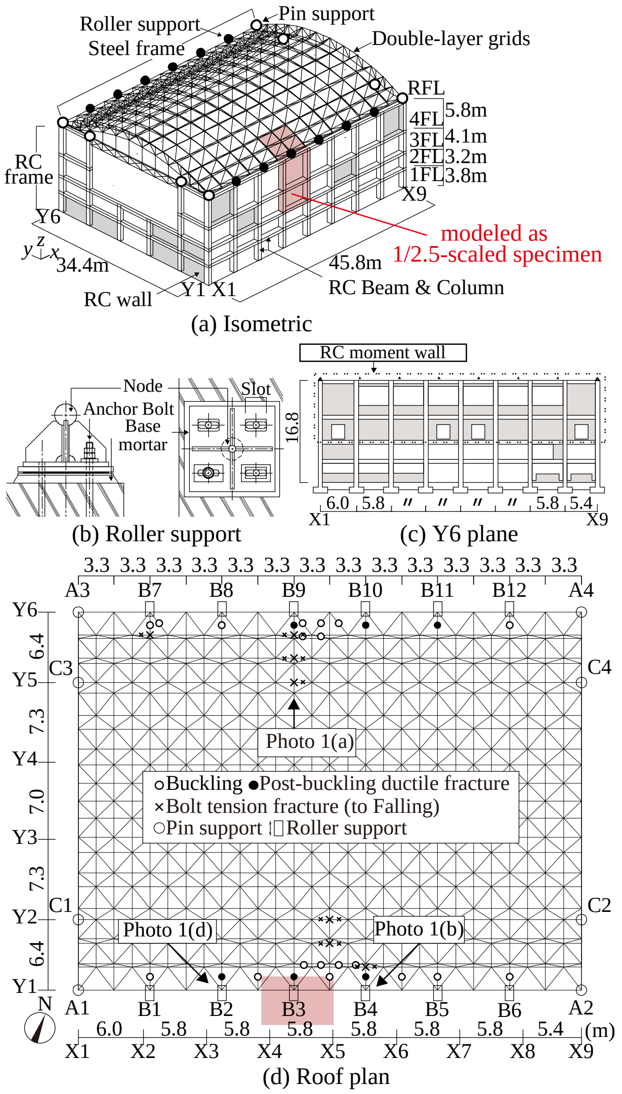


Figure 2*: Schematic drawings of Gym. A* [15]

behavior. The roof supports were modeled as simple shear spring elements with a phenomenological model simulating the contact between the anchor bolt and the end of the slotted hole. The NS- and EW-directional observed earthquake records close to Gym. A was adjusted based on the geographical location and was assigned as the input ground motions. Following the 2016 Kumamoto earthquake, the foreshock on 4/14/2016 and the main shock on 4/16/2016 were continuously input to the numerical model with a free vibration period in between. The Newmark β numerical integration method was assigned in the nonlinear response history analysis.

2.2. Damage mechanism suggested by numerical simulation

A numerical simulation (nonlinear response history analysis, NLRHA) was performed to investigate the damage mechanism of the roof in Gym. A. The numerical model is shown in Fig.4. The model was constructed following the actual structural drawing and design documents. The RC columns and beams were modeled as fiber elements with a bilinear degradation hysteresis model. The roof members were modeled by truss elements with a phenomenological model simulating the buckling behavior. The roof supports were modeled as simple shear spring elements with a phenomenological model simulating the contact between the anchor bolt and the end of the slotted hole.

The response histories of the roof bearings and roof members are shown in Fig. 5. First, the anchor bolts fixing the roof supports are contacted to the end of the slotted hole by the transverse out-of-plane responses of the RC moment frames. Subsequently, the upper chord connected to the roof supports experiences the overall buckling by the transferred force from the anchor bolt, subjecting the diagonals of the truss to the overall buckling. The tension force of the lower chord in the traverse direction significantly increases and reaches the ultimate force of the connection bolt following the loss of the load-bearing capacity in the arch direction. Although the damage of the diagonals of the truss connected to the roof supports was not observed in the numerical simulation, a large vertical displacement occurred at the nodes following the overall buckling of the upper chords, which suggested the collision of the diagonals of the truss to the RC beams. The transverse out-of-plane response of the RC moment frames triggered the complete damage of Gym. A. according to the numerical results. Terazawa and Inanaga et al. [15] provide more details on this subject.

2.3. A buckling strength evaluation method for roof truss members with semi-rigid joints

A buckling strength evaluation method for roof truss members with semi-rigid joints was proposed for the accurate seismic performance evaluation of the school gymnasia using numerical simulation. The schematic illustration is shown in Fig. 6. The rotational stiffness K_r of a semi-rigid joint is evaluated by Eq. (1), according to the member experiments and theoretical section analysis of the connection shown in Fig. 6(d).

$$K_r = \frac{E}{L_{BC}} \begin{pmatrix} 0.702r_1^4 + 0.264r_1^3 r_2 - 0.156r_1^2 r_2^2 \\ +0.057r_1\, r_2^3 - 0.0051r_2^4 \end{pmatrix}$$

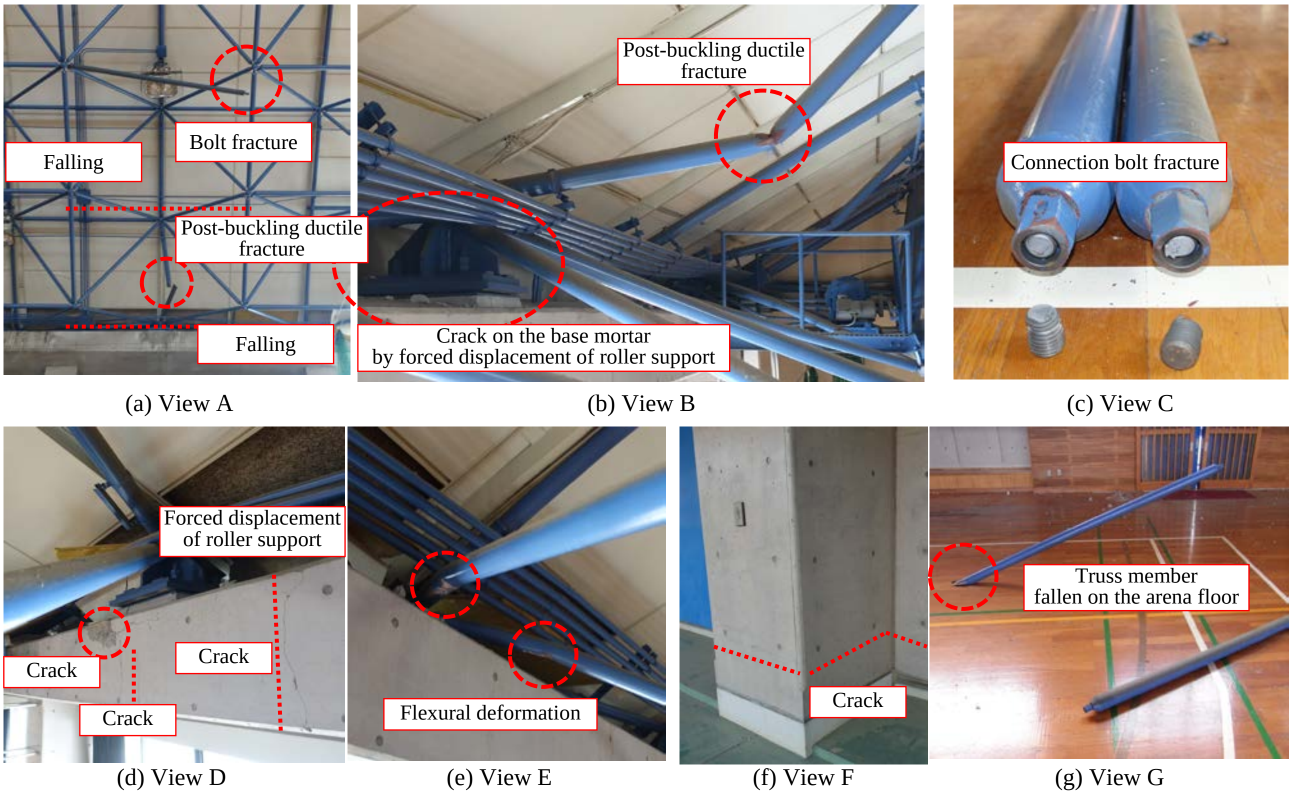


***Figure 3**: Damage of Gym. A in the 2016 Kumamoto earthquake* [3], [15]

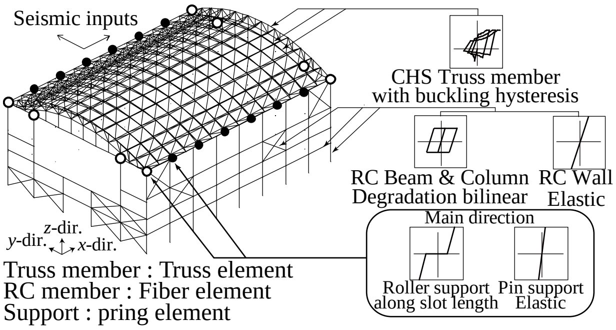


***Figure 4**: Numerical model of Gym. A* [15]

Where r_1 is the radius of the bolt, r_2 is the radius of the coupler, E is the young's modulus of the steel material, and L_{BC} is the effective connection length determined by the connection size.

The effective length factor K of a roof truss member with semi-rigid joints is evaluated by Eq. (2) from the virtual work principle as shown in Figs. 6(e) and 6(f). The corresponding overall buckling strength is calculated following a design standard (e.g. AIJ [21]).

$$K = \begin{cases} (1\text{-}2\alpha)\sqrt{\dfrac{k_r^2+14k_r+64}{4k_r^2+40k_r+64}} & (\textit{semi-rigid - semi-rigid}) \\ (1\text{-}\alpha)\sqrt{\dfrac{17.6k_r^2+120k_r+408}{34k_r^2+187k_r+408}} & (\textit{semi-rigid - pin}) \end{cases} \tag{2}$$

Where k_r is the normalized rotational stiffness (K_r/EI) to the flexural stiffness of the truss member, and α is the length ratio of a node.

Terazawa and Kishizawa et al. [17] also provide more details on this subject.

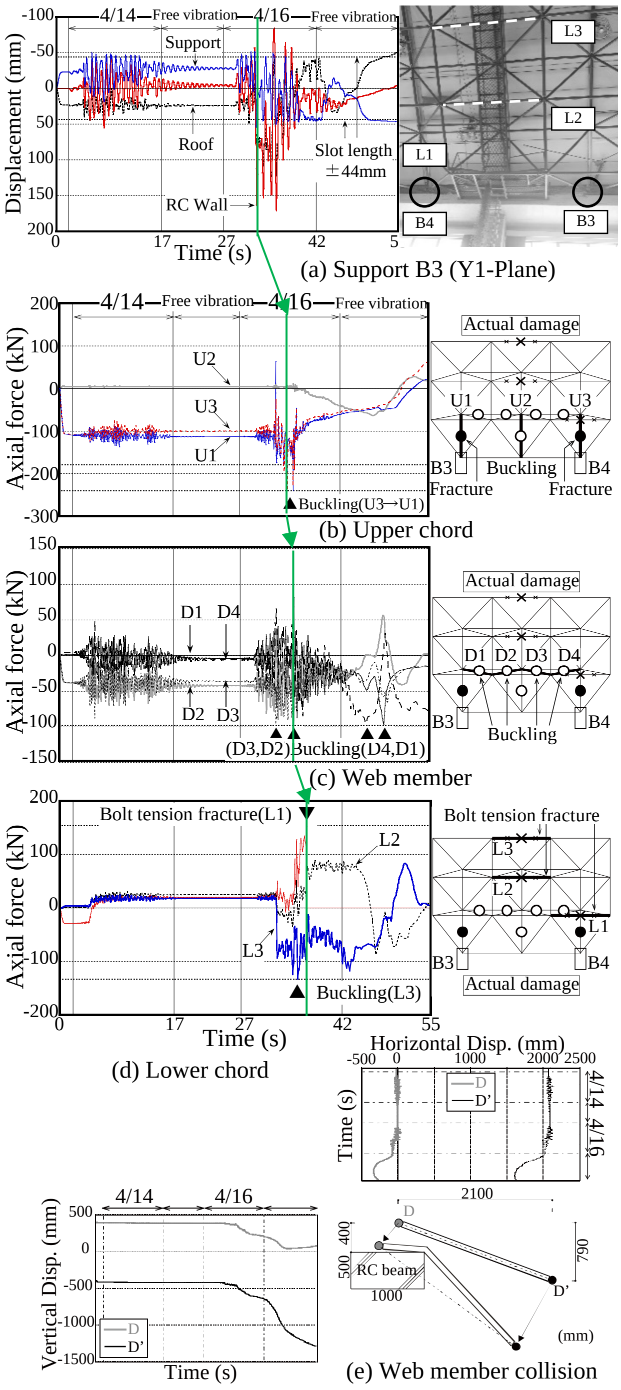


***Figure 5**: Response history results of Gym. A* [15]

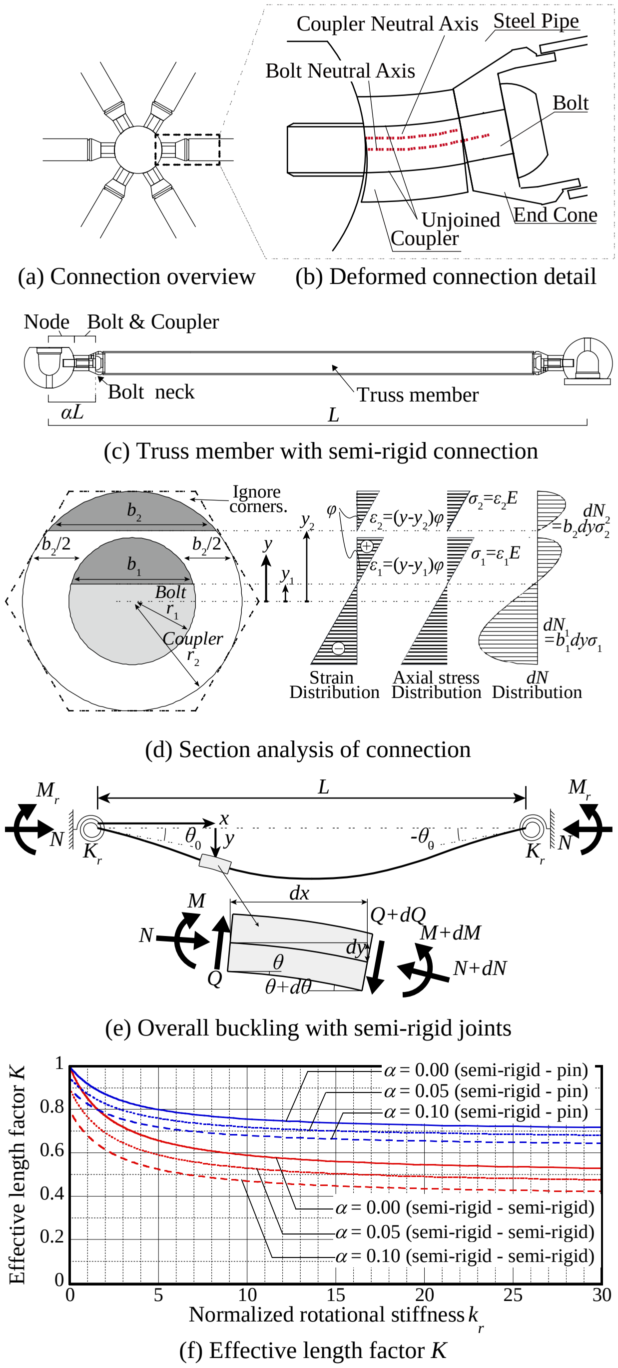
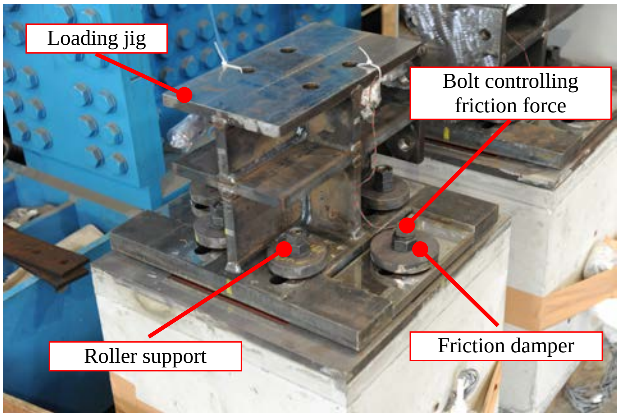


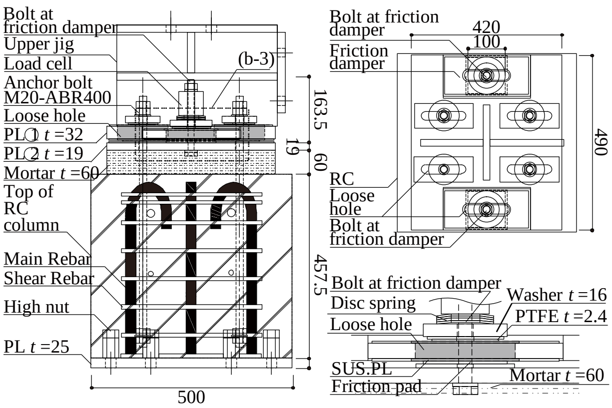
***Figure 6**: Schematic illustration of the overall buckling strength evaluation* [17]

2.4. A response control method by friction damper supports

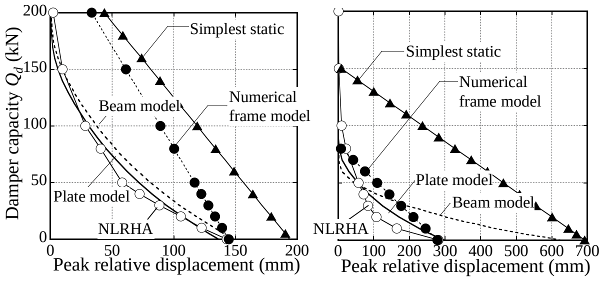
A conventional strength-based seismic design requires the strengthening of every member to remain elastic against the transverse out-of-plane response of the RC moment frames, and thus, this approach is uneconomical. Narita and Terazawa et al. [10], [11] proposed a response control method of the RC moment frames by friction damper supports and verified the efficiency through dynamic loading tests and numerical simulation. The schematic image of the friction damper and the corresponding numerical simulation result are shown in Fig. 7. As shown in Figs. 7(a) and 7(b), the friction dampers are placed next to the slotted holes working as the conventional roller support. The vertical force controlling the friction force is thus independent of the dead load of the roof. As shown in the NLRHA results of Figs. 7(c1) and 7(c2), the relative displacement u_l of the roof supports can be significantly reduced by the proposed friction damper supports.



(a) Previous specimen overview



(b) Previous specimen drawings



(c1) $q \leq 0.409$ (Gym. A) (c2) $q > 0.409$

***Figure 7**: Previously proposed friction damper support* [10], [11], [13], [14], [18]

While a school gymnasium is important as a shelter in Japan, is legally classified as a middle-to low-rise building for seismic design, with equivalent force procedure assigned, and dynamic analysis is rarely used for the design practice. Therefore, a simple transverse out-of-plane response evaluation method of RC moment frames was proposed. The schematic illustration is shown in Fig. 8. A RC moment frame is modeled as a single equivalent beam or plate, and the relative displacement of the roof support without friction dampers is evaluated using Eq. (3) to Eq. (5), which follow the vibration theory of distributed parameter systems

$$u_l = 1.2 \times \begin{cases} \dfrac{2.066 S_A}{\omega_w^2} & (q > 0.409,\ \textit{plate model}) \\ \dfrac{1.566 S_A}{\omega_c^2} & (q \geq 0.409,\ \textit{beam model}) \end{cases} \quad (3)$$

$$\omega_w = 312 q \sqrt{\frac{D_x L}{m_w h_c^3}} \quad (4)$$

$$\omega_c = 111 \sqrt{\frac{E I_c}{m_c h_c^3}} \quad (5)$$

Where S_A is the design spectrum of accretion, D_x and D_y are the flexural stiffness of the equivalent plate, I_c is the moment inertia of the equivalent beam, q is the influence factor by the beam to column determined by Fig. 8(c), E is the young's modulus of concrete, L is the span of the RC moment frame, m_w and m_c are the mass of the equivalent plate and beam, respectively.

Moreover, the damper capacity Q_d per roof support satisfying the response ratio R_d to reduce the relative displacement of the roof support less than the length of the slotted hole is evaluated by Eq. (6) and Eq. (7) based on the theory of the equivalent linearized single degree of freedom system. An R_d corresponding to a specific Q_d can be also computed by the back calculation.

$$Q_d = \left(\frac{n}{n_d}\right) K_{eq} u_l (-0.279 R_d^3 + 0.653 R_d^2 - 0.725 R_d + 0.351) \quad (6)$$

$$n = \begin{cases} 1 \ (q > 0.409,\ \textit{plate model}) \\ n_c \ (q \leq 0.409,\ \textit{beam model}) \end{cases} \tag{7}$$

Where n_d is the expected number of friction damper supports, K_{eq} is the equivalent stiffness of the plate or beam model, n_c is the number of RC columns.

As shown in Figs. 7(c1) and 7(c2), while the appropriate model (the beam or plate model) has to be selected according to the influence coefficient q, the proposed method accurately evaluates the results of NLRHAs. More detail is explained by Terazawa and Nishikawa et al. [18].

3. OUTLINE OF EXPERIMENTS

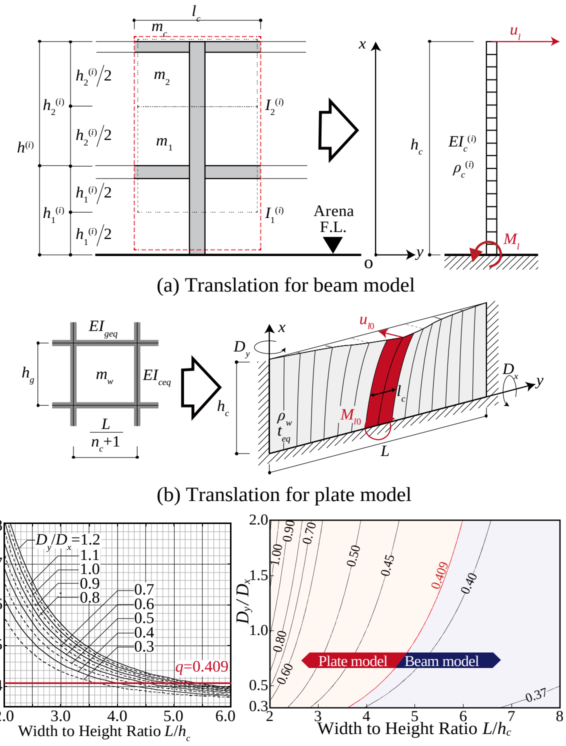
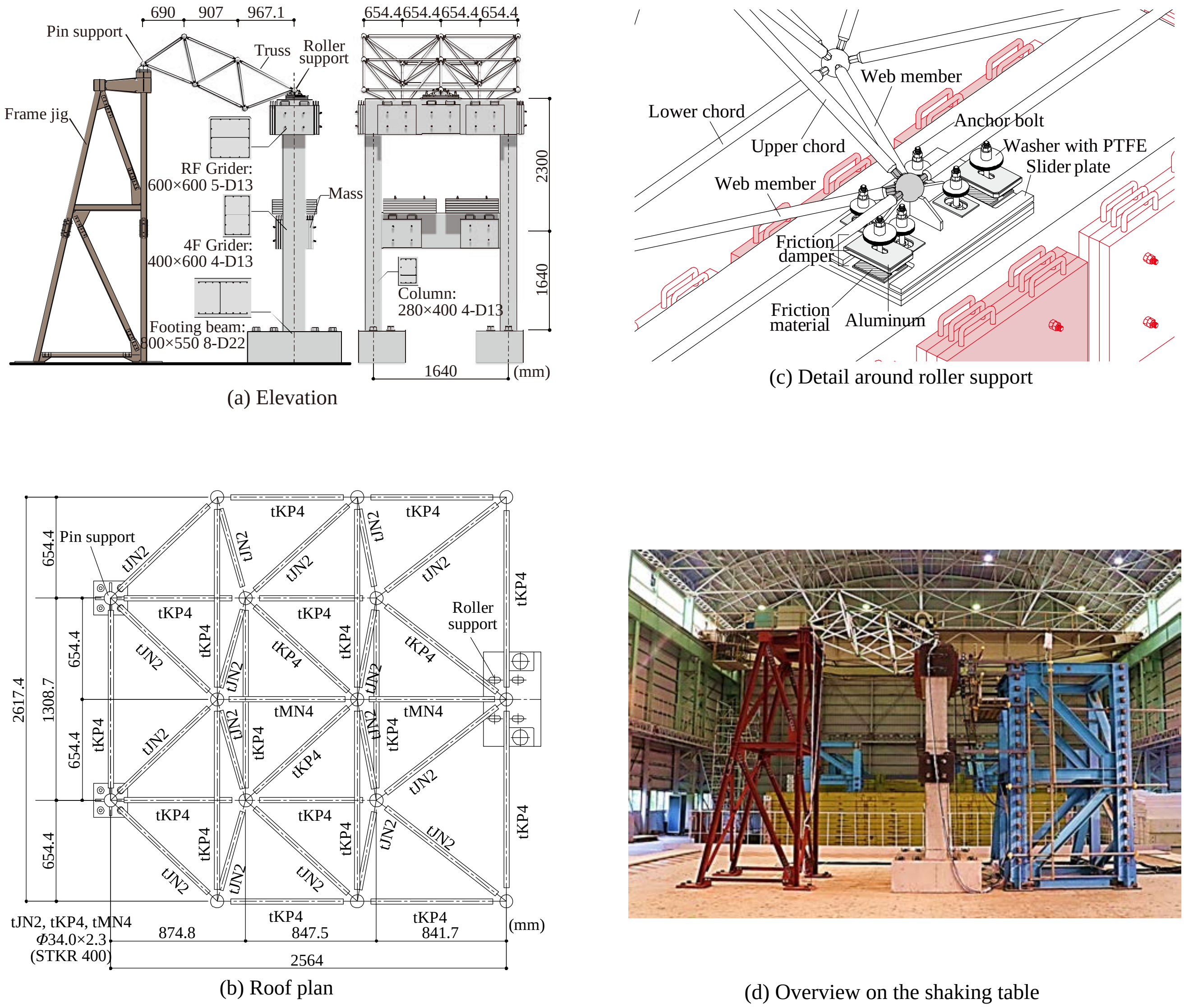


***Figure 8**: Schematic illustration of the out-of-plane response evaluation method* [18]

The schematic of the specimen and the testing overview is shown in Fig. 9. As shown in Fig. 2(a), the specimen is a 1/2.5-scale partial model of Gym. A. Furthermore, Gym. A is composed of the RC moment frame on the arena floor, a double-layered metal gridshell, and roof roller support. Additionally, the shaking table is assumed to be the arena floor. Furthermore, the specimen is attached to the 6.21-ton of inertia mass and frame jig that supports the roof structure. The section size of the RC column, RC beam in 4F, RC beam in RF, and RC footing beam is 280 × 400 mm, 400 × 600 mm, 600 × 600 mm, and 800 × 550 mm, respectively. The size of the main rebars is ϕ13 mm for the RC column and beam or ϕ22 mm for the RC footing beam. The size and intervals of the stirrups are ϕ10 mm @ 110 mm for every RC member. The footing beams are fixed by the anchor bolts on the shaking table. The section size of the roof truss members (tJN2, tKP4, and tMN4) is ϕ34.0 × 2.3 mm (a circular hollow section), and the steel material is STKR 400. The steel ball with the size of ϕ85.0 mm is used as the joint. Furthermore, the size of the connection bolt is M12. The effective length of the slotted hole working as the roller support is ± 18 mm. The roller support is fixed to the RC beam by the M16 anchor bolts with double-nut fastening. Inaba and Terazawa et al. [13], used the same material, which is assigned to the friction damper. The size of inertia mass was selected to make up for the mass of the scaled RC sections. The frame jig was designed to simulate the boundary condition shown in Fig. 2(a).

The response acceleration on the shaking table and the RFL of the RC moment frame are primarily measured in the experiments, along with the displacement of the RC moment frame, the relative displacement and reaction force of the roller support, the axial deformation and force of the upper chord connected to the roller support, and the axial strain and force of the other roof truss members connected to the roller support. The measurement detail around the roller support is shown in Fig. 10. The laser displacement sensors are assigned to measure every displacement response. The axial force of the roof member is computed by the section area, young's modulus, and the measured axial strain. The schematic illustration of the load measurement is shown in Fig. 11. The total reaction force of the pinned supports on the frame jig is equal to that of the roller support, and it is computed as the total axial force of the roof truss members connected to the pinned supports. The axial force of the upper chord connected to the roller support is computed as the difference between the reaction force of the roller support and the total axial force of the other roof truss members connected to the roller support. This measuring plan is adopted based on the preliminary experiment (Fujiwara et al. [23]) and numerical simulation. The time history and spectra of input wave is shown in Fig. 12. The scaled floor response acceleration on the arena of Gym. A was used as the input wave. The floor response acceleration was produced by the numerical simulation with the 2016 Kumamoto earthquake spectrally matched following the Japanese building code. The response control test with the friction damper support was first carried out in the experiment schedule, and the collapse test simulating the damage of the 2016 Kumamoto earthquake was subsequently carried out. Furthermore, the specimen remained elastic in the response control test. However, the waves with various amplitudes were input to the specimen without the friction damper one after another, interspersed with free vibration periods to observe the damage mechanism. The peak ground acceleration with 100%-amplitude was about 600 cm/s^2. The experiment was performed on the shaking table of NIED at Tsukuba city (Japan) in 2020.



(a) Elevation

(c) Detail around roller support

(b) Roof plan

(d) Overview on the shaking table

***Figure 9**: 1/2.5-scaled specimen for the shaking table tests*

***Figure 10**: Measurement detail around the roller support*

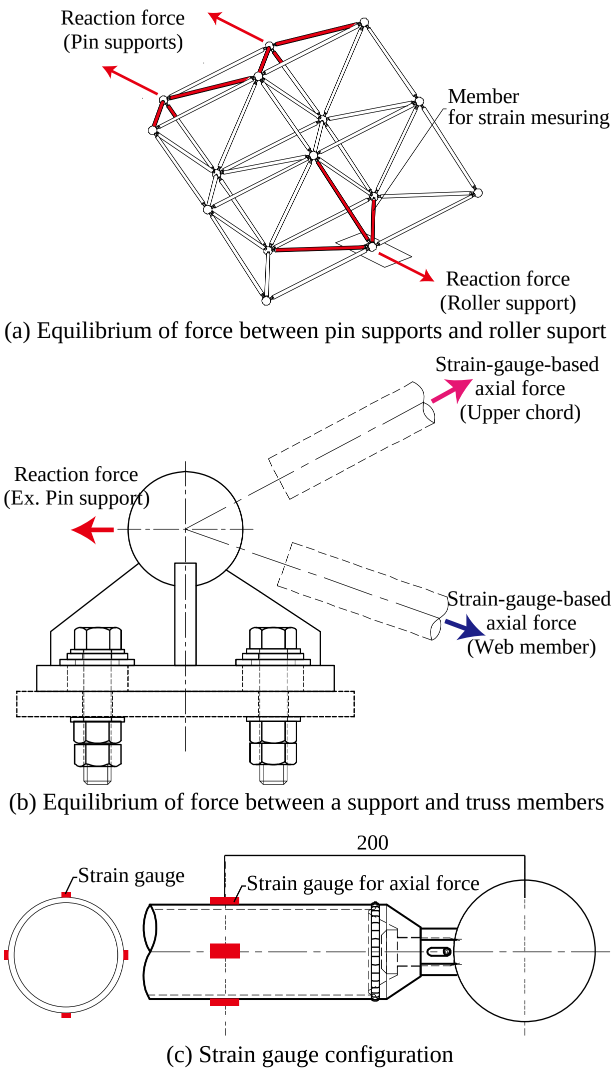
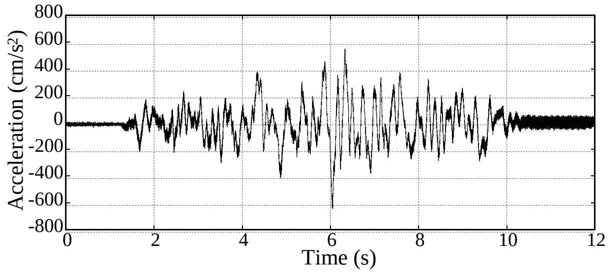
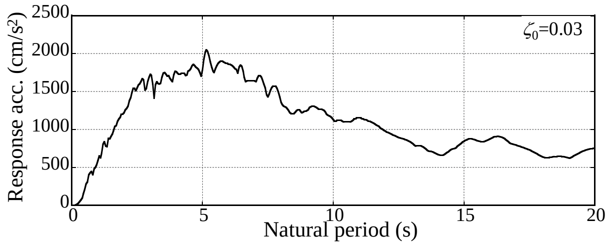


***Figure 11**: Schematic illustration of load measurement*



(a) Time history of input acceleration on the shake table



(b) Acceleration response spectra

***Figure 12**: Input wave (100% amplitude)*

4. RESPONSE CONTROL TESTING RESULTS USING FRICTION DAMPER ROOF SUPPORT

The reaction force–relative displacement relationships of the roller support with and without the friction damper are shown in Fig. 13(a). The transverse out-of-plane response of the RC moment frame was significantly mitigated by the friction damper, and the relative displacement of the roller support is reduced less than the length of the slotted hole (± 18 mm). The origin of the slotted hole was moved by the base mortal cracking in the input with 100% amplitude for the response with the roller support without friction dampers, as shown in Fig. 13(a3).

The friction force of the damper accidentally increased over the designed value during the seismic response. The converted force from the axial force of the bolts controlling friction force and the friction coefficient of 0.85 obtained from the friction material test accurately matched the bare reaction force, as shown in Figs. 13(b1) and 13(b2). This indicates that the bolt controlling friction force at the damper rotated during the seismic response, as shown in Fig. 13(d). Furthermore, the axial force of the bolt increased as shown in Fig. 13(b3). These results indicate that the friction damper's detail could be improved. Nevertheless, the experimental results demonstrate the effectiveness of the response control method proposed by Narita and Terazawa et al. [10], which uses friction dampers.

Fig. 14 shows a comparison of the relative displacement of the roller support between the simple evaluation method described in Section 2.4 and the experimental results. Table 1 contains the evaluation specifications. The RC moment frame is modeled as the single beam for the experiment and the second line of Eq. (3) was used. In the comparison shown in Fig. 14, the friction force generated by the damper and roller support was considered, which was included in the reaction force. No. 1 to 3 represents the results with the friction damper, and the No. 4 to 5 are the results without the friction damper. The intrinsic damping of the bare specimen was identified by the half-power method and the results of white noise input. The proposed method by Terazawa and Nishikawa et al. [18] is shown in Fig. 14, and evaluated the relative displacement of the roller support within the error of about 10 mm, and is thus accurate as a simple method.

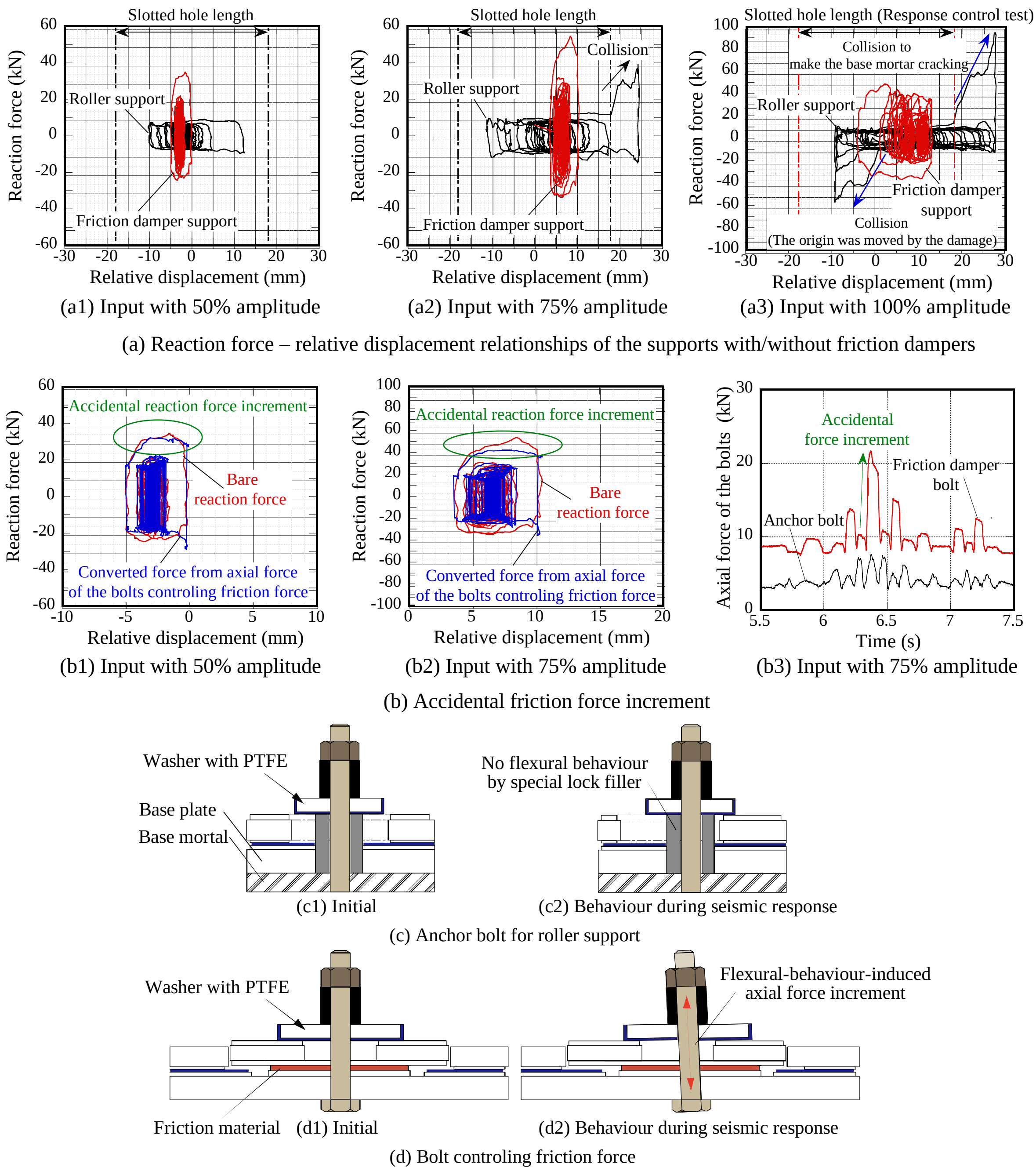


***Figure 13**: Response control testing results*

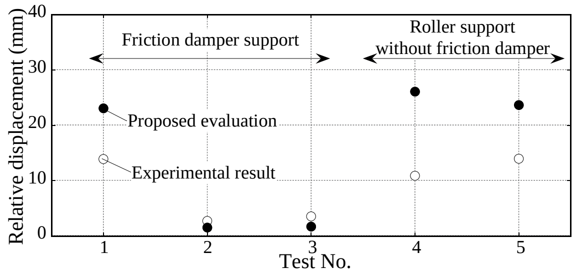


***Figure 14**: Comparison of the relative displacement of the supports*

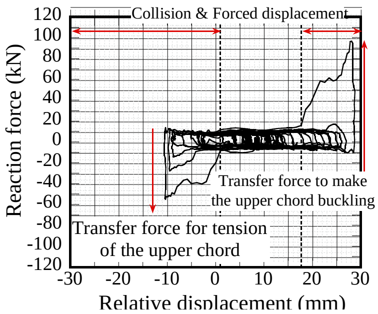
***Table 1**: Summary for the evaluation*

(a) Summary of the experimental results

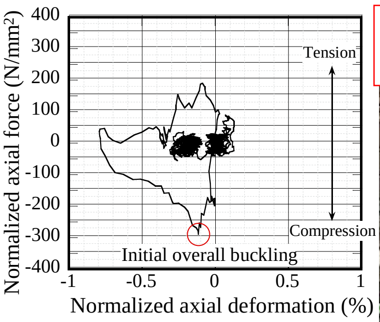
Test No.	Damping ratio ξ_0(%)	Slip load Q_d (kN)	Acc.Spectrum value S_A (m/s^2)	Relative displacement u_l (mm)
1 (100%)	2.16	32.26	16.56	13.90
2 (50%)	2.88	32.88	8.77	2.70
3 (75%)	2.28	47.70	12.87	3.53
4 (50%)	2.41	8.79	9.40	10.90
5 (50%)	2.98	8.97	8.68	13.97

(b) Member specification for the evaluation

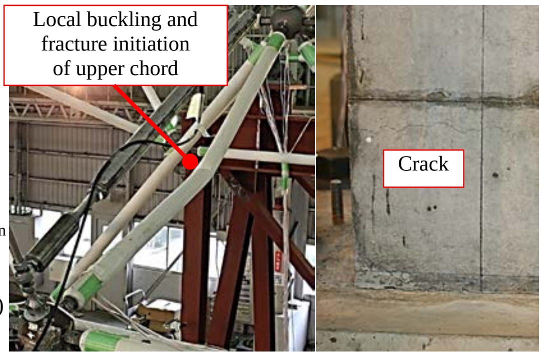
E (N/mm^2)	I_c^{center} (mm^4)	m_c (kg)	m_1 (kg)	m_2 (kg)
24895	4.12E+08	6120	2678	3217



(a1) Roller support

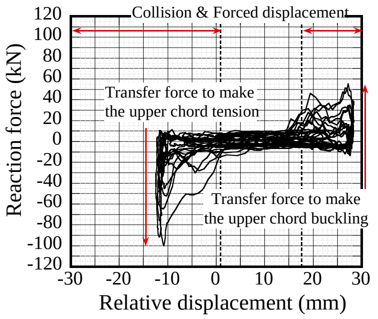


(a2) Upper chord

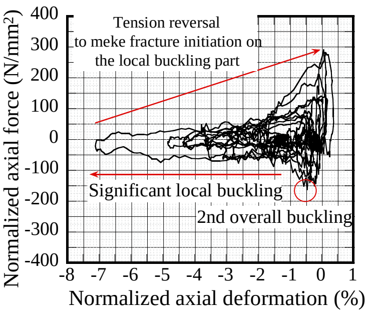


(b3) Local buckling (b4) Flexural yielding of RC column

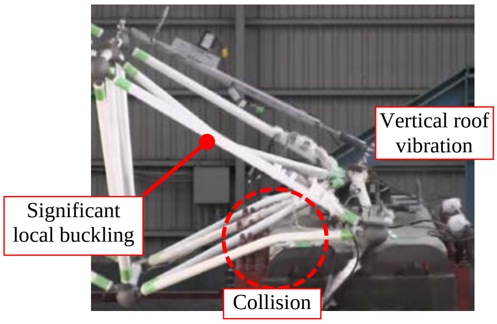
(a) The 1st input with 100% amplitude



(b1) Roller support

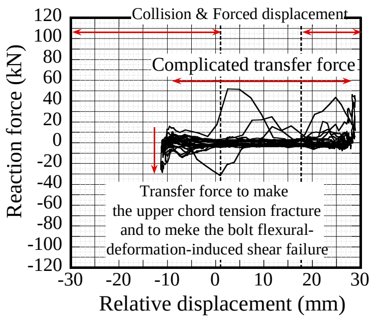


(b2) Upper chord

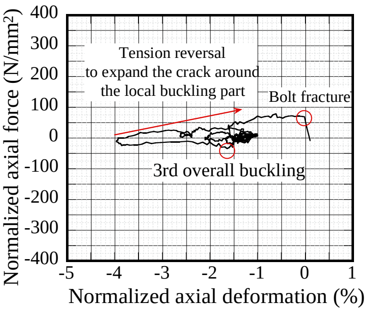


(b5) Collision of diagonals of truss
(a screen shot from video)

(b) The 2nd input with 150% amplitude



(c1) Roller support



(c2) Upper chord

(c3) Bolt fracture (c4) Crack propagation

(c) The 3rd input with 150% amplitude

***Figure 15**: Collapse testing results*

5. COLLAPSE TESTING RESULTS SIMULATING THE DAMAGE

The reaction force – relative displacement relationships of the roller support, normalized axial force – deformation relationships of the upper chord, and the damage situations are shown in Fig. 15. The anchor bolts of the roller support were contacted at the end of the slotted hole by the out-op-plane response of the RC moment frame in the first input with 100% amplitude, as shown in Figs. 15(a1) to 15(a2), and the roller support then experienced forced displacement. The transverse out-of-plane response-induced inertia force of the RC moment frame was transferred to the roof through the roller support, resulting in the overall buckling of the upper chord. Flexural yielding occurred in the RC column base in the second input with 150% amplitude, as shown in Figs. 15(b1) to 15(b5). The upper chord's normalized axial deformation reached around 7% before experiencing local buckling and fracture initiation. Furthermore, the entire roof structure began to vibrate vertically after the upper chord's load-bearing capacity was lost, and the diagonals of the truss connected to the roller support collided with the RC beam numerous times. As shown in Figs. 15(c1) to 15(c4), the connection bolt experienced flexural deformation-induced shear failure in the second input with 150% amplitude, while the crack was expanding around the local buckling part of the upper chord before the gross section fracture occurred. The fourth input was canceled for safety reasons. The collapse test indicates the transverse out-of-plane response of the RC moment frame is the main cause of the subsequent damage to the school gymnasium and validates the damage mechanism proposed by the numerical simulation [15].

The evaluated rotational stiffness of the connection of the upper chord is 7.96 kNm/rad., and the corresponding effective length factor is 0.71. The evaluated buckling strength considering the yielding stiffness of 409 MPa obtained by the material test was approximately 72 kN accordingly. The experimental buckling strength was 66.7 kN. The method proposed by Terazawa and Kishizawa et al. [17] evaluates the experimental result within the error of 6 kN. Compared with the 49 kN of buckling strength with the pin-pin connection used in the usual design standard, the evaluation value was close to the experimental result and is suitable for use in the seismic performance evaluation by nonlinear analysis.

6. CONCLUSION

According to the shaking table testing, the following conclusions were obtained:

1. The transverse out-of-plane response to the RC moment frame on the arena floor caused damage to the school gymnasium with an RC substructure and steel roof. The roof supports were first forced to move by the transverse out-of-plane response, and the upper chords connected to the roof support buckled as a result of the transferred force. The local buckling of the upper chord caused the loss of load-bearing capacity of the roof, and the diagonal of the truss collided with the RC beam following the caving of the roof. Post-buckling ductile or brittle fractures occurred in the upper chords and connection bolts, respectively.
2. The friction damper roof supports effectively mitigated the transverse out-of-plane response of the RC moment frame. The relative displacement of the roof support in the shaking table tests was reduced from 28 mm to 12 mm (less than the length of the slotted hole), with other members remaining elastic. Furthermore, the transverse out-of-plane response evaluation method of the RC moment frame, which considered the response reduction effect by the friction damper support, evaluated the experimental results with an error of approximately 10 mm and was sufficiently accurate as a simple method.
3. The experiment buckling strength (66.7 kN) was evaluated with an error of approximately 6 kN using the buckling strength evaluation method for roof truss members with semi-rigid joints. Considering that the buckling strength error with the pin-pin connection is approximately 30 kN, the proposed method is suitable for use in nonlinear seismic performance evaluation.

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