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Seismic retrofit of existing SRC frame using rocking walls and steel dampers

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Abstract: A retrofit of an existing 11-story steel reinforced concrete frame that features the innovative use of post-tensioned rocking walls and shear steel dampers is presented. The main components of the retrofitting plan and important design considerations are described. The retrofitting system is expected to effectively control the deformation pattern of the existing structure and significantly reduce damage to the existing structure during major earthquakes.

Keywords: building structure, seismic retrofit, rocking wall, steel damper

1 Introduction

Major earthquakes have indicated that the seismic retrofit of existing buildings is necessary because the buildings may fail to satisfy the latest seismic design provisions. In this paper, a novel seismic retrofit plan with rocking walls and steel dampers for a multistory steel reinforced concrete frame is proposed. The following two aspects form the primary focus: 1) The possibility of weak story failure of the existing SRC frame should be eliminated. The difficulty suppressing the unintended weak story failure in frame structures is evident from building damages in historic and recent earthquakes, despite various implementations of the widely accepted “strong column-weak beam” concept in the seismic design of frame structures ([1][2][3]). Instead of a strength hierarchy between beams and columns, the effect of continuous columns on reducing the story drift concentration has been extensively examined for steel frames ([4][5][6]). These attempts may lead to an effective solution for suppressing the weak story mechanism in frames. 2) Damage to the existing frame should be minimized. The SRC frame presented herein was designed and constructed during the late 1970s before the major revision of the seismic code in Japan in 1981, which was mainly a consequence of the 1978 M7.1 Miyagiken-oki earthquake [7]. As suggested by the damage observed in the M7.3 Kobe earthquake 1995, SRC frames designed and constructed in old days usually lack deformability to accommodate damages [8].

Given the above concerns, a rocking wall system is developed to enhance the seismic performance of the existing SRC frame. Rocking walls are global vertical components that are strong and stiff and have sufficient rotating capacity at the bottom. They are responsible for controlling the deformation pattern along the height of the structure to reduce the story drift concentration. Rocking walls need to be firmly connected to the rest of the structure to ensure that the lateral forces can be transmitted (Fig. 1). Energy dissipating devices can be arranged between the rocking wall and the rest of the structure by taking advantage of the concentrated vertical deformation that forms when the structure deforms laterally. It's expected that most of the energy dissipations as well as damages will be concentrated in the energy dissipating devices to minimize damage to the rest of the structure.

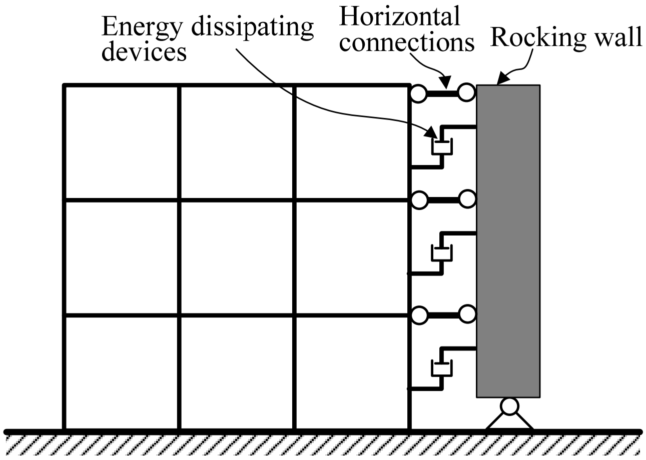


Figure 1 Basic components in a rocking wall system

The advantages of rocking wall systems have been explored by Kurama et al ([9][10][11]), they pressed several precast concrete wall panels together with post-tension tendons to form a rocking wall. Marriot et al [12] introduced steel dampers at the bottom of the rocking wall to increase the energy dissipation capacity. The first applications of a rocking wall system were in a newly built 4-story office building and in the rehabilitation of an existing 6-story RC moment-resisting frame in California, USA [13][14].

The rocking wall system to be introduced in this paper differs from the previous studies in the following aspects: 1) the rocking interface between rocking walls and their foundations is replaced by explicit pin bearings to avoid unfavorable impact at both corners of the wall when it rocks. 2) by placing steel dampers on both sides of the rocking wall, energy dissipation is distributed along the height of the rocking wall, rather than being concentrated at the bottom, which permits more energy dissipation devices to be used in the structural system to greatly increase the energy dissipation capacity. (3) the post-tensioning of the rocking walls is only responsible for increasing the crack strength of the rocking walls, rather than providing any self-centering capacity to the system. On the one hand, it is thought that the strength and stiffness of the rocking wall is much more important than its self-centering capacity, and on the other hand, anchoring the post-tension tendon on the wall instead of in the foundations considerably reduces the cost of strengthening the foundations.

2 Seismic Retrofit of G3 Building in Tokyo Tech

The G3 Building is an 11-story steel reinforced concrete frame structure on the Suzukakedai campus of the Tokyo Institute of Technology in Japan (Fig. 2(a)). As mentioned above, it was designed and constructed before the major revision of the seismic code of Japan and it has already been occupied for more 30 years. As concluded by a recent seismic inspection, there is an urgent need to strengthen the structure, especially in its longitudinal direction.

2.1 Retrofit plan

Figure 2(b) shows the north view of the retrofitted G3 Building. During the retrofit, the building remained occupied because most of the construction was done from outside the building. Figure 3 shows the structural plan of the G3 Building before and after retrofitting. There are several other multi-story concrete buildings on the same campus with similar configurations to that of the G3 Building. A common feather is that there are several slots along the perimeter of the building. This feather makes it easier to implement the rocking wall system. For the G3 Building, 6 pieces of post-tensioned concrete walls with pin bearings at the bottom were installed in the existing slots and firmly connected to the existing frame at each floor level by horizontal trusses. Shear steel dampers were installed in the gaps between the rocking walls and adjacent existing SRC columns as well as between the rocking walls and the added transverse walls at both ends. Main components of the rocking wall system, i.e. the post-tensioned concrete walls, the steel dampers, and the bottom pin bearings, are visible from outside the building; thus people can see them and appreciate the engineering solution.

The seismic behavior of the retrofitted G3 building is different from that of a shearwall-frame structure and a moment-resisting frame. No recommendations for the seismic design of such a structural system are available yet. Nevertheless, several basic criteria regarding the expected seismic performance of the retrofitted structures are met. First, the post-tensioned rocking walls should remain elastic, even if the structure is subjected to major earthquakes, such as the Level II earthquake in the design practice in Japan. In other words, the rocking walls should not yield or crack, which may significantly impair their stiffness. Second, the story drift ratio of the structure should remain below 1/200 during a major earthquake. This requirement is very strict compared with current seismic codes for reinforced concrete structures. However, it is believed necessary in the current case considering the fact that the existing SRC frame is built before 1981, and its deformability might be rather poor. Lastly, steel dampers at different levels of the structure should be proportioned such that the energy dissipation is as evenly distributed along the height of the building as possible.

Bearing in mind these concepts, nonlinear time history analysis are carried out to determine the earthquake action on each part of the structure and to evaluate the seismic performance. In the following, these key components are described in detail.

(a) Existing (b) Retrofitted

Figure 2 The north views of G3 Building before and after the retrofit

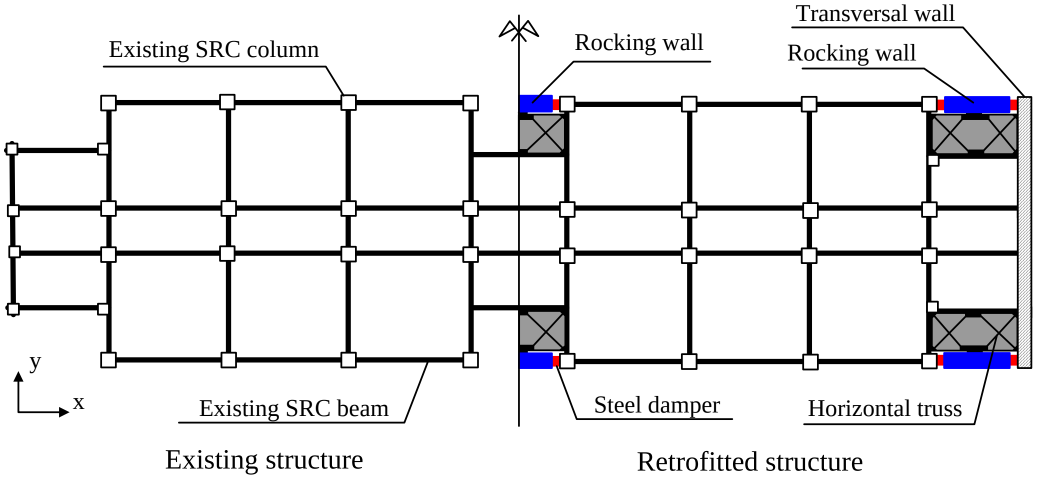


Figure 3 Structural plan of the existing and retrofitted building

2.2 Post tensioned concrete walls

Because the rocking walls are responsible for controlling the deformation pattern of the structure, it is expected that their stiffness and strength can be retained even under a major earthquake. All 6 pieces of post-tensioned concrete walls have identical cross-sections with a width of 4300mm and a depth of 600mm, as shown in Figure 4. The total cross section area of the rocking walls at each story is about 50% to 61% of that of the existing SRC columns from the bottom to the top story. Concrete with a nominal compressive strength of 36MPa is used. Each rocking wall is pre-stressed by 6 units of post-tensioned tendons to increase its cracking strength. Each tendon unit comprises 30 strands of 12.7mm. The initial pre-stress for each rocking wall is 22500kN, and the corresponding control stress is about 68% of its nominal tensile strength. The resultant effective pre-stress is over 18000kN for each rocking wall.

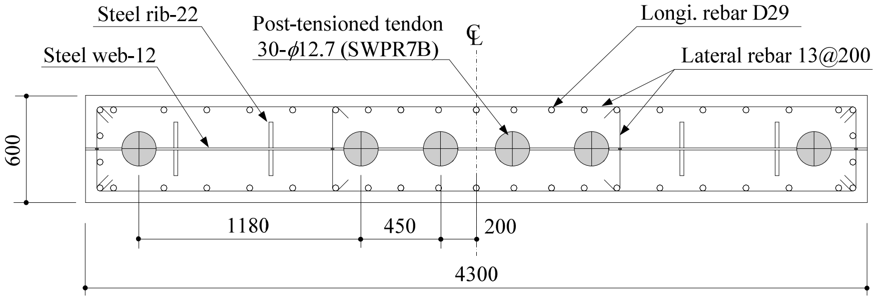


Figure 4 Cross section of rocking walls

2.3 Connections for rocking walls

Rocking walls are connected to the foundation and the existing structure. Cast iron pin bearings are installed at the bottom of the rocking walls. Details and a photo of the completed bearing are shown in Figure 5. It was designed to resist large shear force while permitting the wall to rotate freely around its base. The cast iron bearing consists of two separated tooth-shaped pieces (the lower and the upper piece), which interlock with several teeth and a separated stopper in the middle to prevent displacement in the out-of-plane direction of the wall (Fig.6). The teeth in the lower piece are 20mm longer than those in the upper ones to create a small gap, and their tips are filleted to allow for rotations of the upper piece. Cast iron NCN490 with nominal yield strength of no less than 325MPa was used for the bearings.

It should also be noted that the rocking walls have little effect on the fundamental period and the maximum base shear force of the existing structure. As a result, the foundation work for the rocking walls is not excessive, and the shear demand for the pin bearing is not very large.

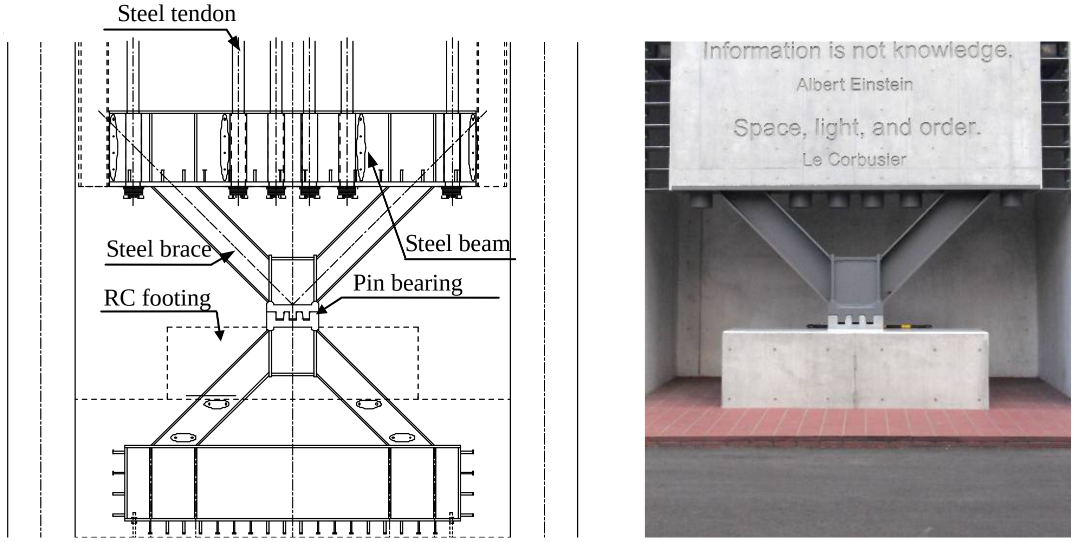
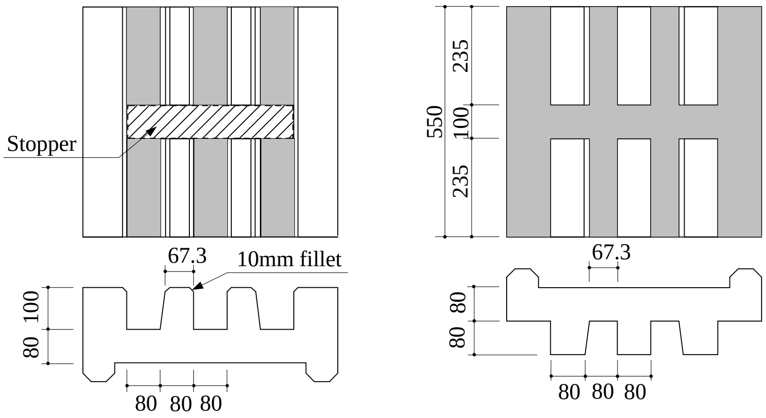


Figure 5 Pin bearing at the bottom of the rocking walls



Lower piece　Upper piece

Figure 6 Details of the cast-iron bearing

Rocking walls are connected to the existing structures by the horizontal trusses at each floor level in the slots of the existing structure behind the rocking walls. It can be seen in Figure 7 that the horizontal trusses are firmly connected to the existing structures by anchor bolts. Steel shear keys are used to connect the horizontal truss and the rocking wall to permit the rocking walls to rotate while transmitting the lateral force.

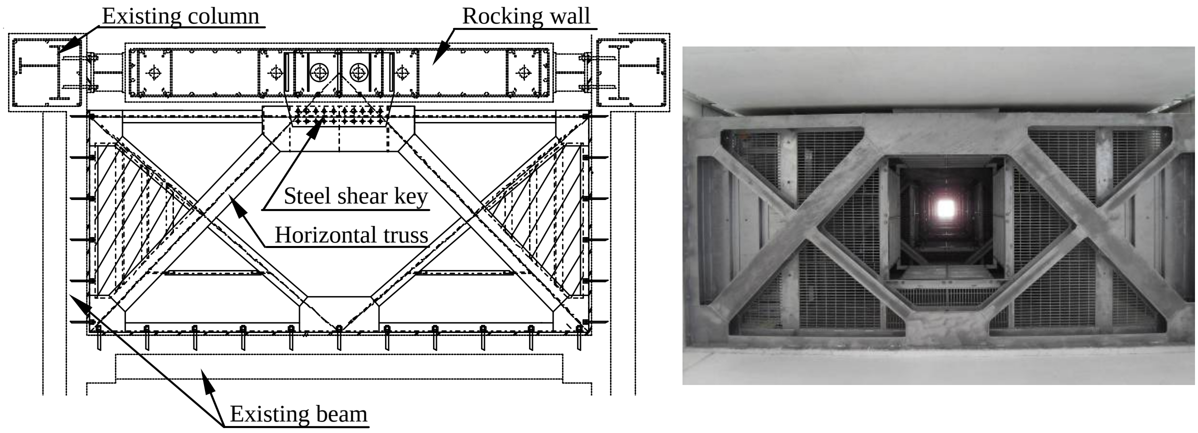


Figure 7 Horizontal truss

2.4 Shear steel damper

Shear steel dampers are installed on both sides of the rocking walls. Low yield steel SLY225 with a nominal yield strength σ_y of 225MPa was used for the 6mm steel web of the damper, which functions as the energy dissipater and is constrained by transverse ribs with a spacing of 250 mm. The web height H was 312 mm for all the dampers, and the length L varied from 750 mm to 1500 mm. Figure 8 shows details and a photo of a completed steel damper with a web length of 1500 mm. The cyclic loading test of the steel dampers shows that the nominal strength of the damper can be satisfactorily retained up to 9% shear strain, which is about 58 times the yield shear strain of the damper. The nominal strength of the damper is calculated by multiplying the steel nominal shear strength (taken as $\sigma_y/\sqrt{3}$) and the cross section area of the web [15]. The deformation of 750 mm steel damper at the end of the test as well as its hysteresis loop is shown in Figure 9. Most of the earthquake input energy is expected to be dissipated by these dampers.

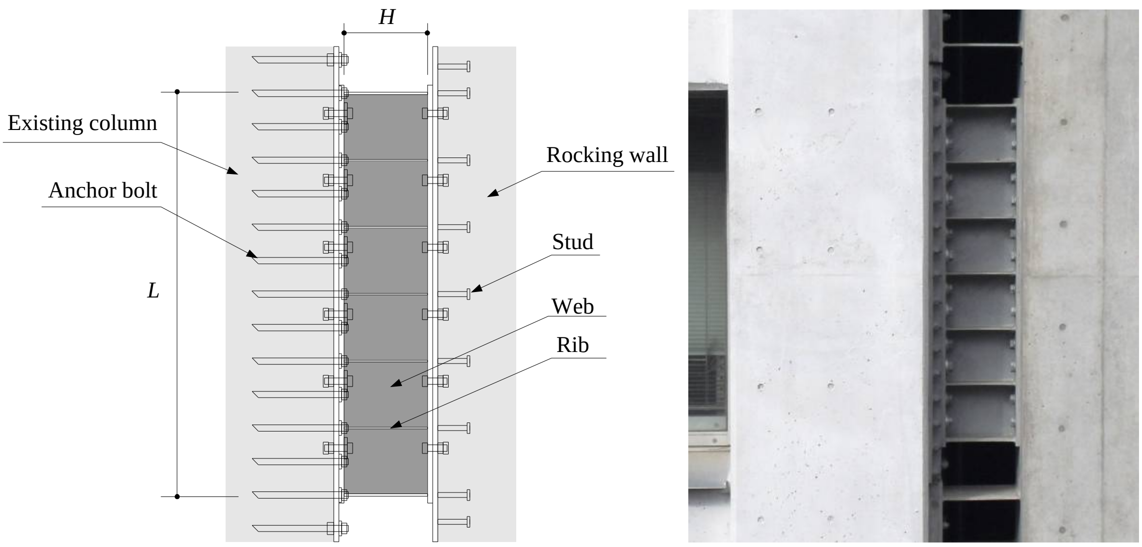


Figure 8 Installed steel damper with a length of 1500mm

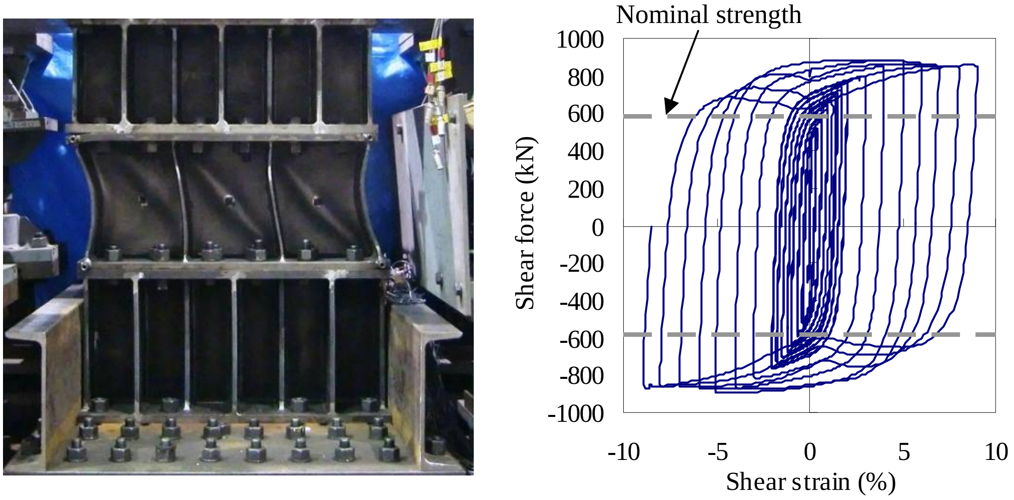


Figure 9 Steel damper deformations and its hysteretic response to cyclic loading [15]

3 Seismic performance assessment

Nonlinear time history analysis was carried out to assess the seismic performance of the structure before and after the retrofit. Two ground motion records, NGT-NS and JMA Kobe-NS, are used and their acceleration time histories are depicted in Figure 10. The peak ground accelerations (PGA) and peak ground velocities (PGV) are listed in Table 1. They generally represent a Level II earthquake ground motion in the seismic design practice in Japan, i.e. PGV=50cm/s.

Table 1 Peak ground acceleration and velocity of the selected records

ID	PGA (gal)	PGV (cm/s)
JMA Kobe – NS	381.15	55.61
NGT - NS	224.33	42.36

The pseudo acceleration response spectra of the two records are shown in Figure 11, where the fundamental periods of the structural models before and after the retrofit were 0.68s and 0.58s, respectively.

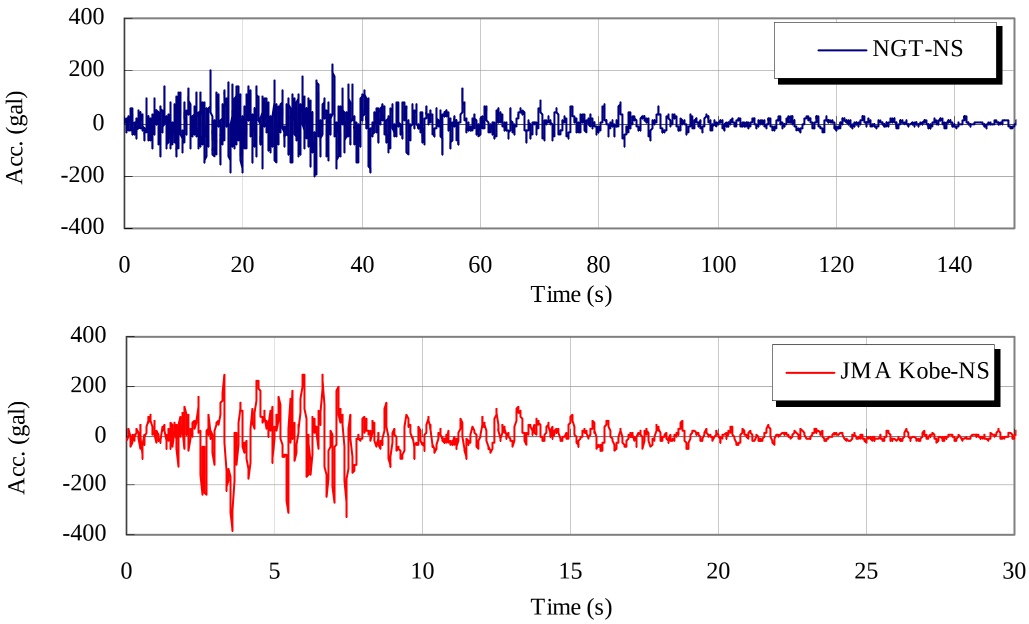


Figure 10 Ground motion records used in the analysis

Two-dimensional member-by-member finite element models are built in ABAQUS 6.8. A fiber-based beam element is used to model the existing SRC frame, and user-defined uniaxial materials were used for concrete fibers and steel fibers. The behavior of the steel dampers was idealized as an elastic-perfectly plastic model, and the rocking walls were assumed to remain elastic through the analysis. The maximum story drift ratios of the structure before and after the retrofit under the above ground motions are shown in Figure 12. It is obvious that the deformation of the structure is significantly reduced and is below the 1/200 criteria under both ground motions after being retrofitted. Furthermore, the deformations in different stories are much more evenly distributed along the height of the structure, which indicates that the damage is spread throughout the structure so that excessive damage is not concentrated in a local part of the structure, which could cause premature failure of the whole structure.

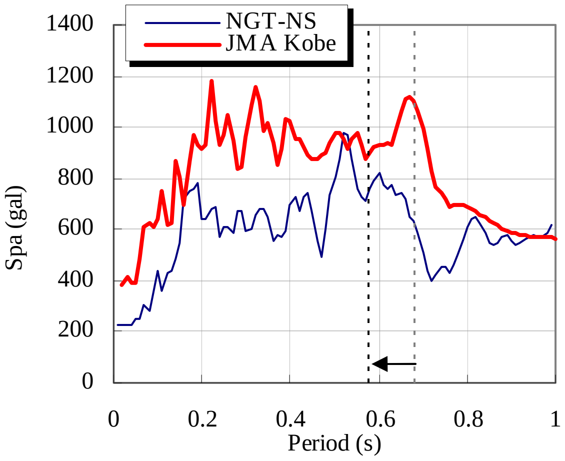


Figure 11 Response spectra of the selected ground motion records

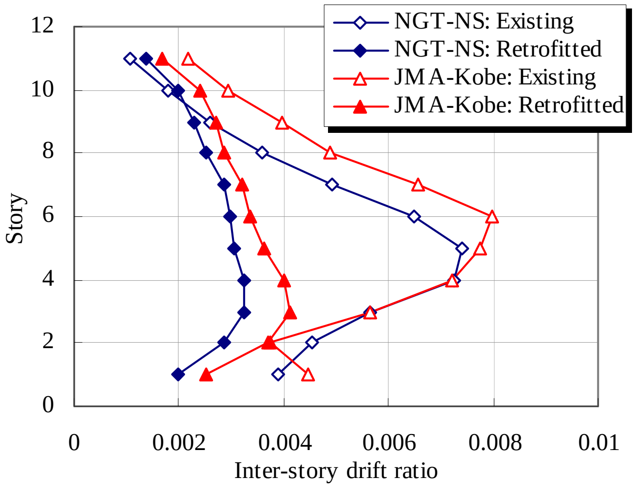


Figure 12 Story drift ratios of the structure before and after the retrofit

4 Conclusions

An innovative application of the continuous stiffness concept is presented to control the deformation pattern of a building structure over its height. Post-tensioned concrete rocking walls and shear steel dampers are featured. This structural system has the following notable characteristics:

(1) The structure integrity is increased by continuous rocking walls, and weak story failure can be effectively suppressed.

(2) With the well-controlled deformation pattern, steel dampers can take full advantage of the concentrated vertical deformation between the rocking walls and the existing structure to dissipate the earthquake input energy. As a result, the energy dissipation demand for the rest of the structure is minimized, and the damage to the rest of the structure is reduced.

In the retrofit project presented above, the construction is also economically feasible because the building remained occupied during the construction.

Notably, the rocking wall system is applicable not only to retrofitting of existing buildings. The arrangement of rocking walls and dampers might be easier to implement in newly designed buildings without the limitation of the structural layout of the existing structures.

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