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Development and application of hysteresis steel dampers

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ABSTRACT: "The Honeycomb Damper System" and "the Joint Damper System" have been developed, based on excellent energy-absorbing capabilities to reduce the seismic response of highrise buildings and large scale structures. The Honeycomb Damper System, chiefly for highrise buildings is formed by steel plates with honeycomb-shaped openings installed between stories to function only for loads acting within its own plane. This system, which is applied to a 29 story SRC building, can realize the well-balanced framing structure having the highly seismic safeness and can improve its human comfort. The Joint Damper System connects two or more adjacent structures with different vibration characteristics by the specially devised dampers capable of functioning with loads acting in any direction. This system, using the newly devised hourglass-shaped dampers, is applied to an indoor ski slope structure, reducing the response of several steel space frames simultaneousely and their relative displacements at the expansion joints.

1. INTRODUCTION

We have carried out research and developed passive seismic control systems which reduce the response of structures using hysteretic steel dampers. The purpose of applying these systems is saving the steel weight and the reinforced concrete volume accompanied with keeping the highly seismic safeness and the human comfort of structures during earthquake.

For highrise buildings, "the Honeycomb Damper System" using the newly deviced steel plates with honeycomb-shaped openings, as shown in Photo.1, has been developed. In this system, the excellent energy-absorbing capability is best utilized by installing these steel plates between stories continuously through a structure's height. This system is capable only of functionig when loads act in their own plane. This system is designed to apply to a 29 story building.

For large scale structures and a group of highrise buildings, "the Joint Damper System", which controls the response of structures by inserting dampers in-between two or more adjacent structures with different natural frequencys, has been developed. In this system, the excellent energy-absorbing capability is best utilized by concentrating installation of the dampers at the point where large relative displacements occur. This system can operate for loads acting in any direction. In a former application of this system, bell-shaped steel dampers were applied to an actual building, confirming

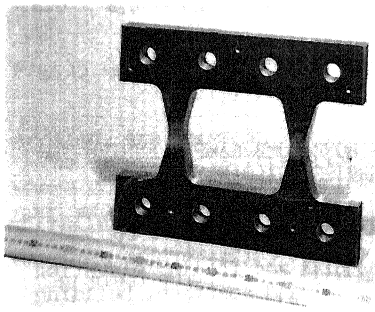

Photo.1 honeycomb damper plate

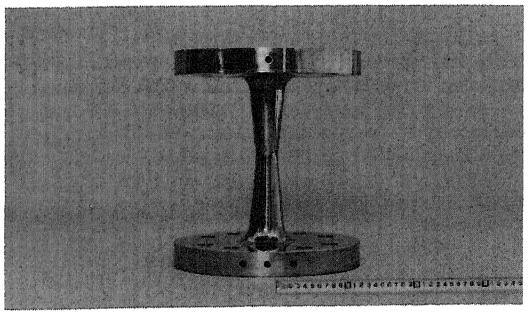

Photo.2 hourglass-shaped damper

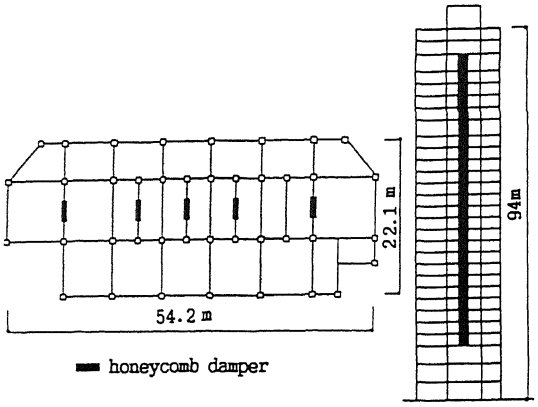

Fig.3 Typical floor plan and framing elevation of a 29 story building

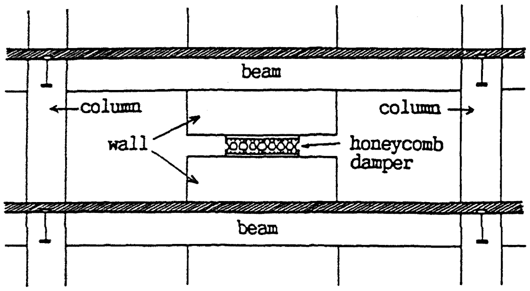

Fig.4 Installation of honeycomb damper

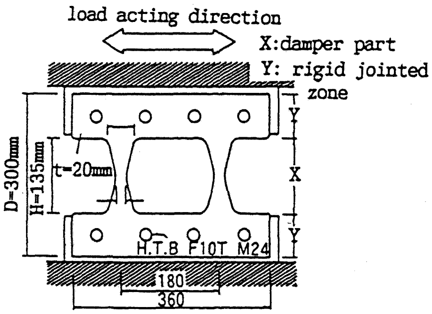

Fig.5 Test specimen of honeycomb damper

their effectiveness in reducing seismic response.[1] We now apply "the Joint Damper System" to the large scale structure of the indoor ski slope, using the newly deviced hourglass-shaped dampers, as shown in Photo 2

This paper presents the development and application of the two systems within actual structures.

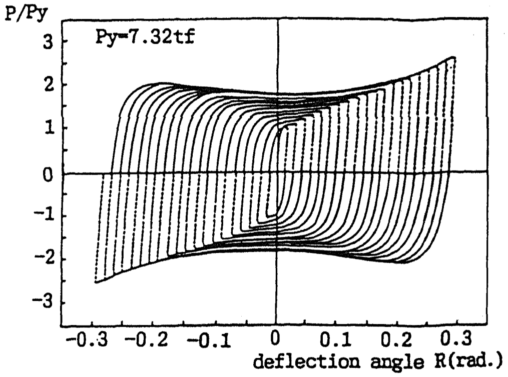

Fig.6 Load-deformation curve of honeycomb damper for cyclic loading

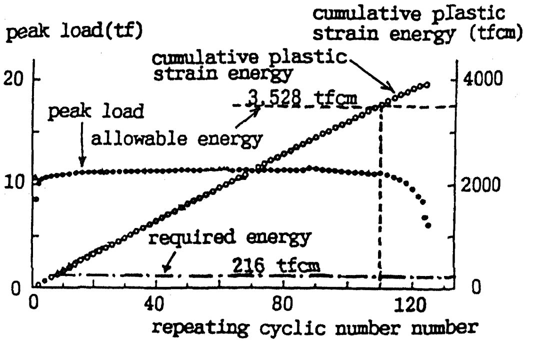

Fig.7 Energy absorption and repeating cyclic number in honeycomb damper

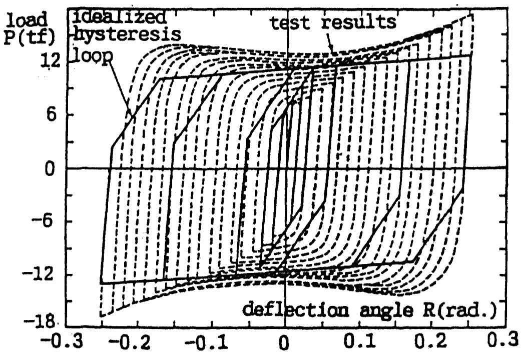

Fig.8 Modelling of restoring force characteristics

2.APPLICATION OF HONEYCOMB DAMPER SYSTEM TO A 29 STORY HIGHRISE BUILDING

2.1 Structural outline and damper installation

The typical floor plan and framing elevation of the 29 story hotel and apartment building to which "the Honeycomb Damper System" is

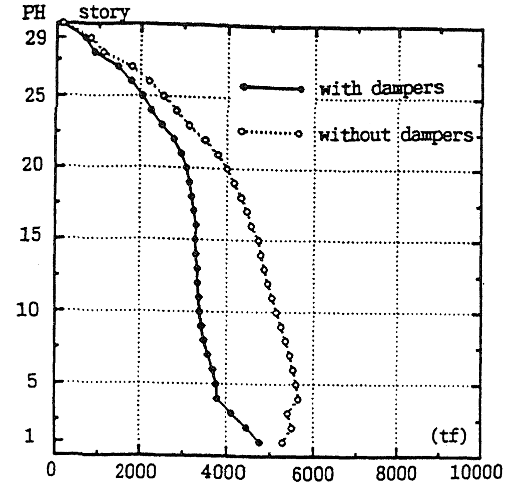

Fig.9 Max.response story shear forces

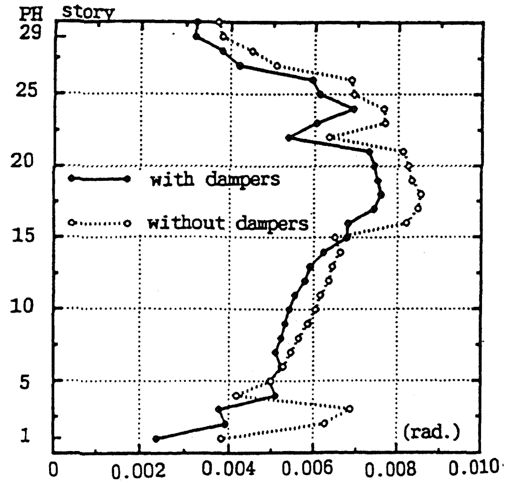

Fig.10 Max.response story drift angles

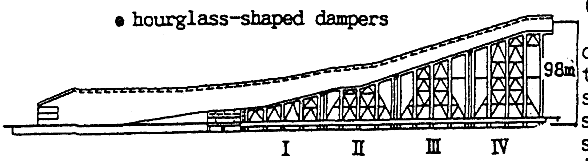

Fig.11 General view of the indoor ski slope

applied, are shown in Fig.3.

The structure is formed by open frames where columns are steel encased reinforced concrete and beams are steel or steel encased reinforced concrete. The building height to eaves is approximately 94 m.

The steel plates with honeycomb-shaped openings are installed from the 4th floor to the 27th floor. Their installation is as shown in Fig.4. Walls extending from the mid-span of upper and lower story beams are spliced at mid-story using the honeycomb damper plates, connected by high tension bolts through gusset plates. Thus, story shear drift is concentrated in the damper plates. In this building, 8 steel plates are installed in each wall at each story with the total number of plates used being 960.

2.2 Structural tests of honeycomb damper

The test specimen, which is a full-scale model, is shown in Fig.5. The honeycomb steel plate consists of the damper parts and rigid jointed zones. The damper part, whose height is 135 mm, is shaped so as to yield uniformly against the bending stress caused by the lateral drift. The specified steel whose elongation ratio (40%) is larger and yield ratio (70%) is higher than usual is adopted.

The two cyclic load cases adopted were increasing amplitude and constant amplitude corresponding to the max.deflection angles of 1/15 - 1/4.

The load- deformation curve under the cyclic loading with increasing amplitude is shown in Fig.6. Here, the vertical axis indicates the normalized load divided by the yielding load. The shape of the hysteresis loop is almost square from small to large amplitudes.

Fig.7 shows the relation between the repeating cyclic number and the peak load, the cumulative plastic energy under the constant amplitude corresponding to the the max. deflection angles of 1/15. This figure indicates the stable energy-absorbing capability of the damper system until the peak load reduces. Hence, the design allowable cumulative plastic strain energy per one piece of the damper plate can be determined to be 3,528 tfcm.

2.3 Earthquake response analyses

(1) Analytical model and analysis method

First, a unit of the damper wall which consists of the honeycomb damper plates and the walls for them to be built into is substituted by an equivalent column with the same bending rigidity. Then, the nonlinear static analysis against the gradually increasing earthquake lateral forces is conducted by introducing the aforementioned equivalent columns into the space frame model of this building. On this occasion the restoring force characteristic is assumed to be as the idealized trilinear curve based on the load-deformation curve obtained by the test, as shown in Fig.8.

Next, from the nonlinear static analysis, framed structures are substituted into equivalent bending-shearing elements with shearing nonlinearity in every framing line, and each unit of the damper wall is

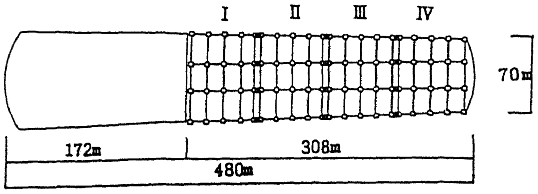

Fig.12 Plan of the indoor ski slope

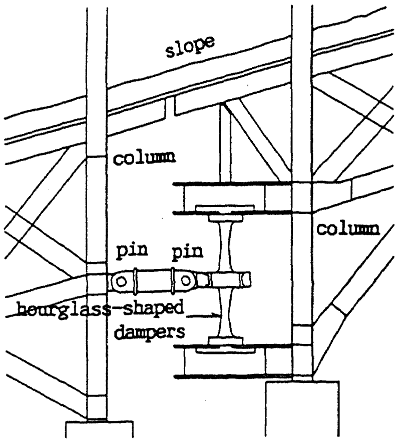

Fig.13 Installation detail of hourglass-shaped dampers

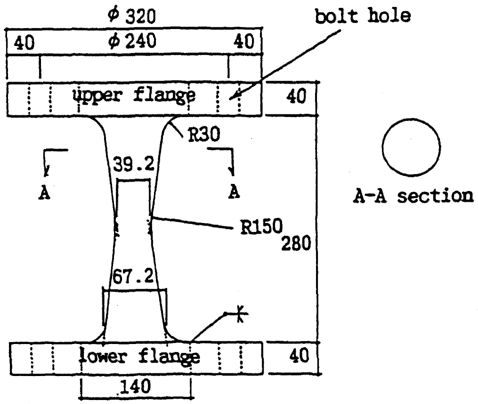

Fig.14 Test specimen of hourglass-shaped damper

substituted into equivalent nonlinear shearing elements. Consequently, the analytical model is the multi-lumped mass model where these elements are coupled by the rigid floor slabs in their horizontal planes.

The internal viscous damping system is adopted, and the damping factor is assumed to be 3 % for the fundamental natural vibration period of this building, 2.03sec.

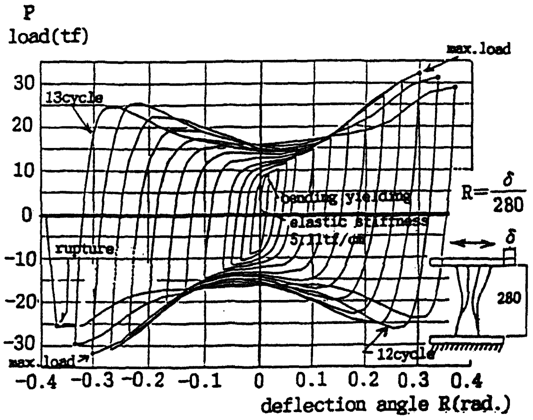

Fig.15 Load-deformation curve of hourglass-shaped damper for cyclic loading

(2) Design earthquakes and their intensities

Four strong earthquake ground motion records are adopted as the design input earthquake waves, and their max. input velocities are determined to be 25cm/sec. as Level 1 and 50cm/sec. as Level 2.

(3) Analysis results

Typical examples of analysis results, the max.response story shear forces and the max. story drift angles in the transverse direction against the severest earthquake of El Centro 1940 (NS), 50cm/sec. are shown in Fig.9 and 10, respectively. In these figures the response of the building without dampers having the same fundamental natural vibration period as that of the building with dampers are also shown for the confirmation of the damper effect.

Comparison between the two cases with and without dampers, indicates that the max. response story shear forces and the max. story drift angles with dampers are reduced to 65 - 90 % and to 55 - 122 % of those without dampers, respectively.

Calculating the cumulative plastic strain energy in the damper wall from the response results, the strain energy is maximum in the unit of the damper wall on the 4th floor. The cumulative plastic strain energy per one piece of the damper plate in this unit is obtained to be 216 tfcm.

Consequently, against the Level 2 earthquake, the safety factor to the design allowable strain energy, which is determined a stated above from the test, is obtained to be approximately 16.

3.APPLICATION OF JOINT DAMPER SYSTEM TO INDOOR SKI SLOPE STRUCTURE

3.1 Structural outline and damper installation

The structure composing the indoor ski slope

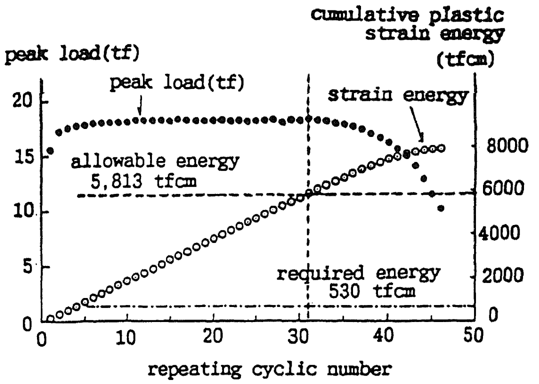

Fig.16 Energy absorption and repeating cyclic number in hourglass-shaped damper

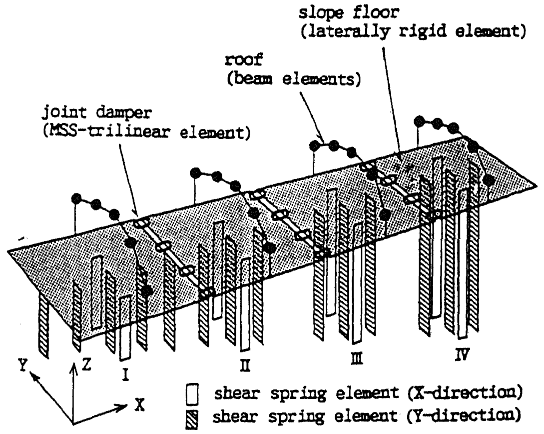

Fig.17 Vibration model of indoor ski slope space structure

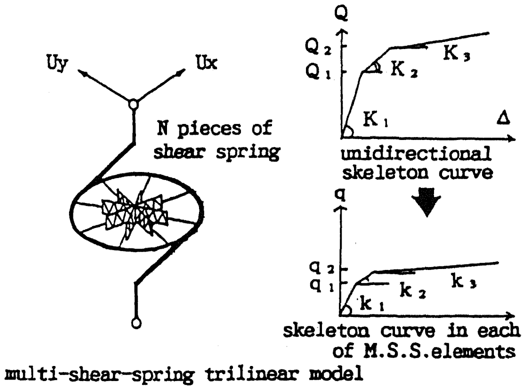

Fig.18 Nonlinear biaxial modelling of restoring force characteristic for damper

area is long and large steel space frames whose floor is rigid in each sub-structure and descends steeply, as shown in Fig.11 and 12. Its maximum height is approximately 98 m, and its length approximately 480 m. The width of the slope increases from 70 m to 100 m, as its height decreases from 80 m to 0 m.

The whole structure composing the downhill slope is designed to be divided into two large blocks, the lower part and the higher part. The lower part is designed to be reinforced concrete frames with shear walls, and the higher part to be braced steel framed construction, divided into the 4 sections by expansion joints with dampers.

At each of the three expansion joints within the higher part, four pairs of hourglass-shaped dampers are installed using a total number of 24 dampers. A pair of hourglass-shaped dampers is stacked vertically on top of one another, as shown in Fig.13. The connection between the adjacent sections is through a connecting bar with pin joints at both ends. One end connects to a column, while the other attaches to the center flanges of a pair of hourglass-shaped dampers in the adjacent section.

3.2 Structural test of hourglass-shaped damper

The test specimen of an hourglass-shaped damper which is approximately an one-third scale model is shown in Fig.14.

This hourglass-shaped damper is fastened rigidly to each structure at the upper and the lower flanges with its middle portion as the plastic range of the damper. The shape of the middle portion is such that it will yield uniformly over its length due to the relative displacement between the upper and the lower flanges.

The steel material is SS41 stipulated in the Japanese Industrial Standard.

The same cyclic loading processes as for the tests on the honeycomb damper are adopted.

The load-deformation curve under the cyclic loading with increasing amplitude is shown in Fig.15. The shape of the hysteresis loop is almost square at small amplitudes indicating excellent energy-absorbing capability, but is like a butterfly due to the hardening at larger amplitudes.

Fig.16 shows the relation between the repeating cyclic number and the cumulative plastic energy under the constant amplitude corresponding to the Level 2 earthquake. From this figure, the design allowable cumulative plastic energy until strength reduction can be determined as 5,813 tfcm.

3.3 Earthquake response analyses

(1) Analytical model and analysis method

The analytical model of the higher part steel structure is shown in Fig.17. In each of the 4 sections, the concentrated mass points are considered in the slope floor to be laterally rigid (1 mass point) and in the

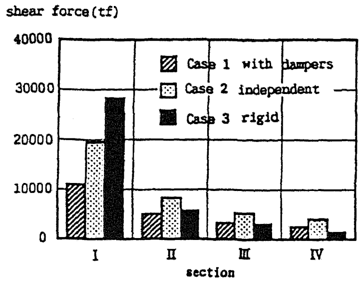

Fig.19 The totals of the max. response shear forces in each block on X-direction

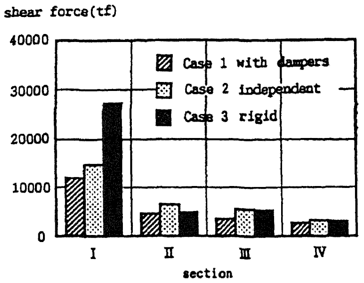

Fig.20 The totals of max. response shear forces in each block on Y-direction

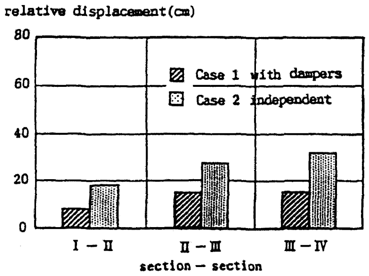

Fig.21 Max. response relative displacements at each expansion joint

roof arch (5 mass points). Next, each steel structure composing the slope floor is represented by the equivalent shear spring models, and the roof arch is represented by beam element models. Then, the 4 models which are set up in order are jointed by nonlinear spring models representing the joint dampers.

The nonlinear spring of the joint damper is first assumed to be the idealized trilinear hysteresis loop with unidirectional characteristics based on the load-deformation curve from the test. Since the damper may be subjected to forces simultaneously in 2 directions X and Y within a plane, the multi-shear-spring model representing biaxial nonlinear characteristics obtained by expanding the unidirectional characteristics to the bidirectional are adopted, as shown in Fig.18.

Damping system of Rayleigh type is assumed, and damping factors are assumed to be 0.02.

(2) Analysis cases

In order to confirm the effect of the joint Damper, the analysis cases considered are as follows:

Case 1: The Joint Dampers are installed as in the actual design.
Case 2: The structure is designed to be completely divided into 4 sections without dampers.
Case 3: The whole structure is designed to be jointed rigidly.

(3) Design Earthquakes and their intensities

They are chosen to be the same as the aforementioned case of a 29 story building.

(4) Analysis results

As typical examples of analysis results, the totals of the max. response story shear forces of the frames composing each section in the earthquake applied direction where the severest earthquake of Taft 1952 EW, 50cm/sec. is applied in the X and Y direction independently are shown in Fig.19 and 20 respectively. Also, the max. response relative displacements at the expansion joints are shown in Fig.21.

In Case 2 (independent), the max. shear forces in every sections and the relative displacements at each expansion joint become larger. In Case 3 (rigid), the shear forces concentrate excessively on the lowest section. While, in Case 1 (with dampers) the max. shear forces are almost all smaller than those in Case 2 and 3. Namely, they are 57-83% of those in Case 2 due to the effect of dampers.

Also the relative displacements at each expansion joint are 45- 55% of those in Case 2.

Calculating the cumulative plastic strain energy of the damper corresponding to the test specimen against Level 2 earthquake from the response results in Case 2, it can be obtained to be 530 tfcm which is much smaller than for the aforementioned allowable value, 5,813 tm.

4.CONCLUSION

"The Honeycomb Damper System" and "the Joint Damper System" not only can realize the well-balanced structures with the highly seismic safeness and can improve their human comfort by the superior hysteretic energy-absorbing capability, but also can save the steel weight and the reinforced concrete volume of structures.

REFERENCE

1)Kobori, T., et al. 1988.: Study on elasto-plastic joint damper. 9WCEE, Japan.