Council on Tall Buildings and Urban Habitat Korea

International Journal of
High-Rise Buildings
www.ctbuh-korea.org/ijhrb/index.php

International Journal of High-Rise Buildings
September 2024, Vol 13, No 3, 273-280
https://doi.org/10.21022/IJHRB.2024.13.3.273

Application of a Joint Damper System to two High Rise Buildings of Different Heights

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Abstract This paper presents an overview of the structural design of the Shibuya Sakura Stage, a complex consisting of two high-rise buildings in Shibuya, Japan. The 39-storey, 180 m high SHIBUYA Tower and the 17-storey, 90 m high Central Building are connected at the top of the Central Building by a joint damper system. This makes the lower floors structurally integrated and achieves a structural plan that is consistent with the design and planning of the lower floors. The building also incorporates other structural control devices, such as high performance passive hydraulic dampers with semi-active characteristics, which provide high additional damping effect from small to large deformations, improving safety in the event of a major earthquake as well as habitability.

Keywords Joint damper system, Structural Control

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1. Introduction

This paper presents an overview of the structural design, including the application of a joint damper system to two high rise buildings of different heights on a site adjacent to Shibuya Station, one of Japan's most famous terminal stations.

2. Building Outline

Adjacent to Shibuya Station, the "Shibuya Sakura Stage" offers a variety of office, commercial, residential, educational, and cultural facilities. It consists of two zones: the urban "SHIBUYA Side" on the side of JR Shibuya Station, and the leafy "SAKURA Side" connected to residential areas. The "SHIBUYA Side" covered in this report consists of two buildings, the 39-storey "SHIBUYA Tower" to the north and the 17-storey "Central Building" to the south, with commercial floors on the lower floors and office floors on the upper floors. SHIBUYA Tower is an attractive workplace near Shibuya Station, with an international medical facility with global capabilities and enterprise

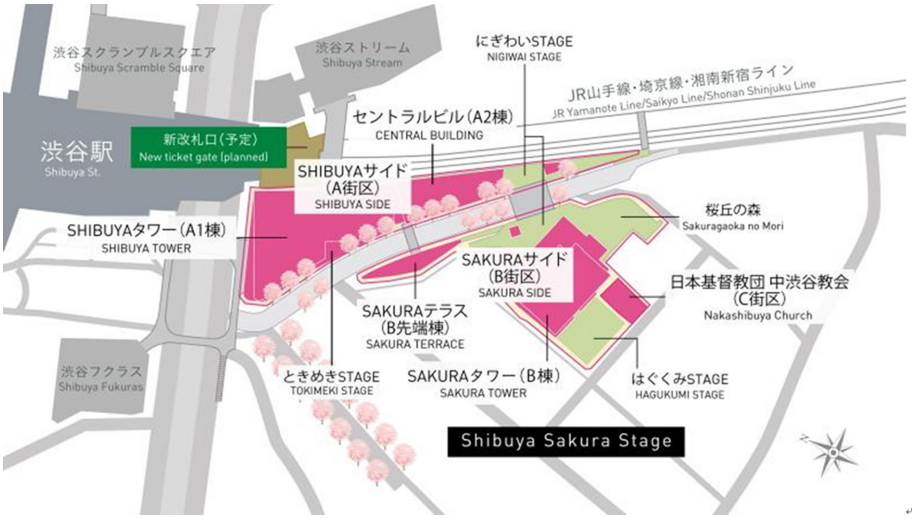


Figure 1. Plan of the "Shibuya Sakura Stage" area.



Figure 2. Panoramic view of the "Shibuya Sakura Stage".

support facilities. The lower floors are also an area that gives a sense of urban Shibuya and is memorable to visitors, with the "Urban Core" running from the second basement to the third floor above ground, directly connected to the deck crossing Route 246, the "Tokimeki STAGE" connected to the new ticket gate of JR Shibuya Station, the "Nigiwai STAGE" connected to the SAKURA side via the crossing deck, and the new symbol "Shibu S".

The offices are arranged in an L-shape in the "SHIBUYA Tower" and an I-shape in the "Central Building", making rational use of the triangular site. Each building has a different standard floor area, and the office layouts encourage companies of all sizes and in a variety of industries to move in, from large corporations to start-ups.

3. Outline of the Structural Design

On the SHIBUYA side, two high-rise buildings of different heights are arranged side by side: the SHIBUYA Tower (Building A1), which is 39 stories above ground and 180 m high, with a large office area, and the Central Building (Building A2), which is 17 stories above ground and 90 m high, with a small office area. Normally, for the sake of seismic design clarity, Exp. J. is provided up to the ninth floor, excluding the basement, and each building is structurally designed as a separate building, but by adopting a joint damper system that connects Building A1 and Building A2 at the top of Building A2, the number of seismic control devices on the A2 side is minimized so that the rental floor area is maximized. The relative displacement between the two buildings during earthquakes is reduced, and the structural plan also takes into account economic efficiency by reducing the total amount of steel and eliminating Exp. J., which reduces potential water leakage problems and installation costs.

3.1. 'SHIBUYA Tower' (Building A1)

The office floor of Building A1 is a trapezoidal plan of approximately 90 m × 65 m, combining an L-shaped rental space with a triangular common space. The office floor is 21.6 m column-free. The structure is a moment resisting steel frame consisting of square steel tubes filled with high strength concrete (CFT) columns and steel beams, with high performance oil dampers (HiDAX-e) and buckling restraint braces (BRBs) as structural control devices. HiDAX-e, the variable damping coefficient oil damper, which provides high additional damping from small deformations during wind loading and small to medium earthquakes to large deformations during large earthquakes, is used to improve occupant comfort and safety during large earthquakes. To suppress the misalignment between the center of gravity and the center of rotation of the building caused by the plane shape, the rigidity of the plane is balanced by adjusting the arrangement of the BRBs and the beam strength of the perimeter beams so that torsional vibrations are not excited by the eccentricity caused by the trapezoidal plane.

HiDAX-e is a passive damper with semi-active

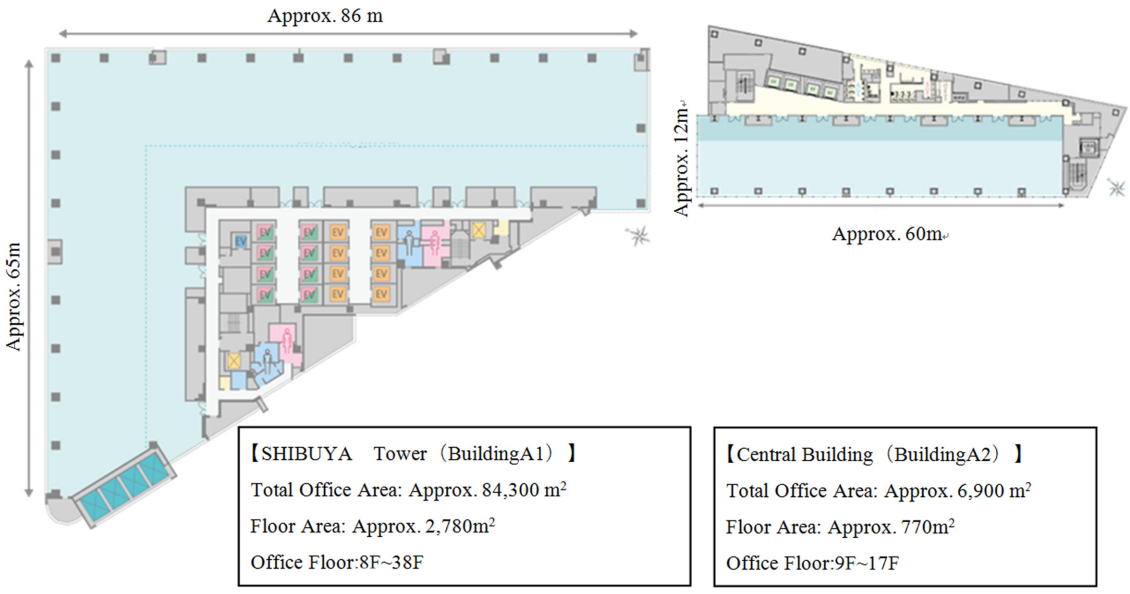


Figure 3. Office floor plans.

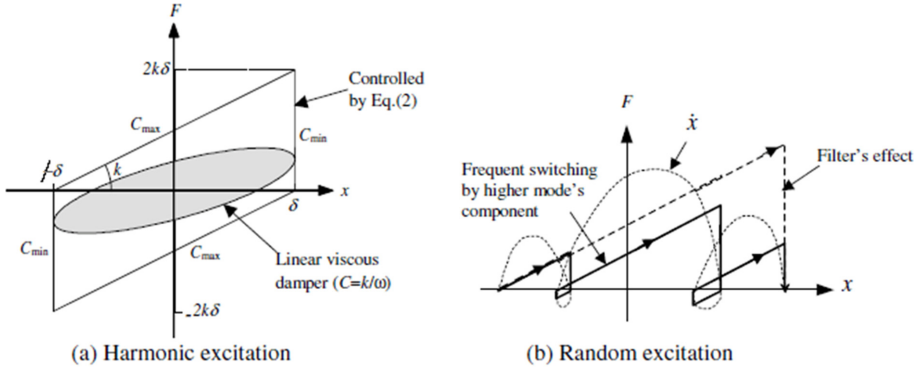


Figure 4. Relations between damper force and displacement.

characteristics. The control algorithm for maximizing the energy dissipation of a semi-active device is expressed by the following on/off or bang-bang control formula (Kurino, 2003).

$$\begin{cases} F\dot{x} \geq 0 \quad or\ |F| \leq F_0 \quad :C(t) = C_{max} \\ F\dot{x} < 0 \quad and\ |F| > F_0 \quad :C(t) = C_{min} \end{cases} \tag{1}$$

the semi-active damper controlled by Equation (1) realizes much more energy dissipation than a conventional passive damper.

HiDAX-e has been applied to about 50 buildings, and its control effect has been confirmed by observation in actual buildings. (Kurino, 2010)

Figure 5 shows the relations between damper force and displacement of the results of the seismic response analysis.

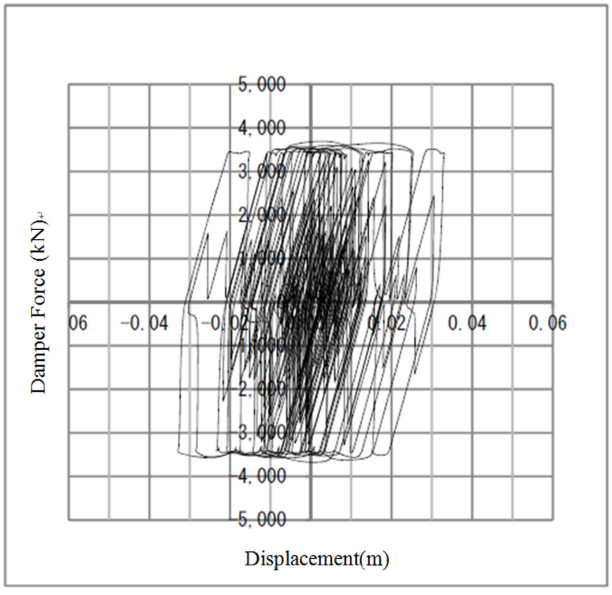


Figure 5. Relations between damper force and displacement of HiDAX-e.

3.2. 'Central Building' (Building A2)

The office floor of Building A2 has an elongated

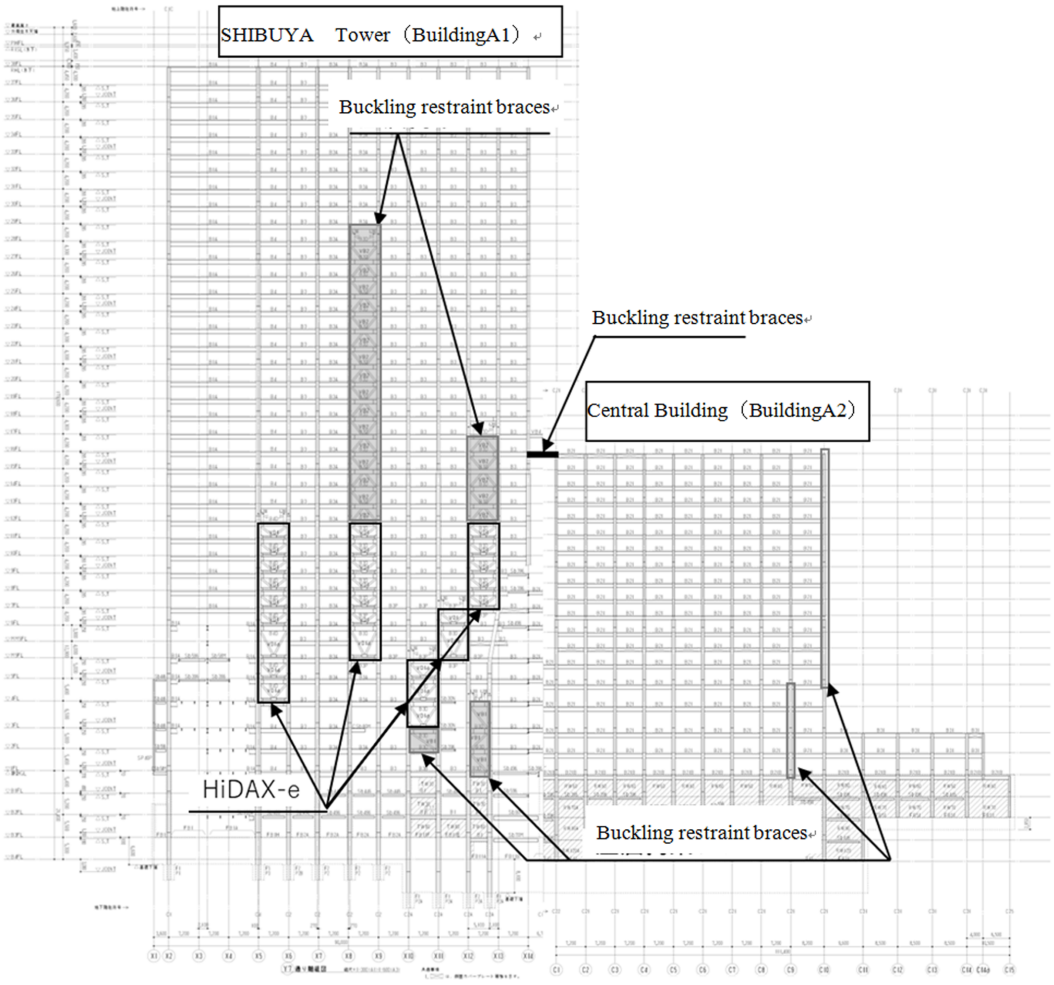


Figure 6. Conceptual diagram of the structural plan.

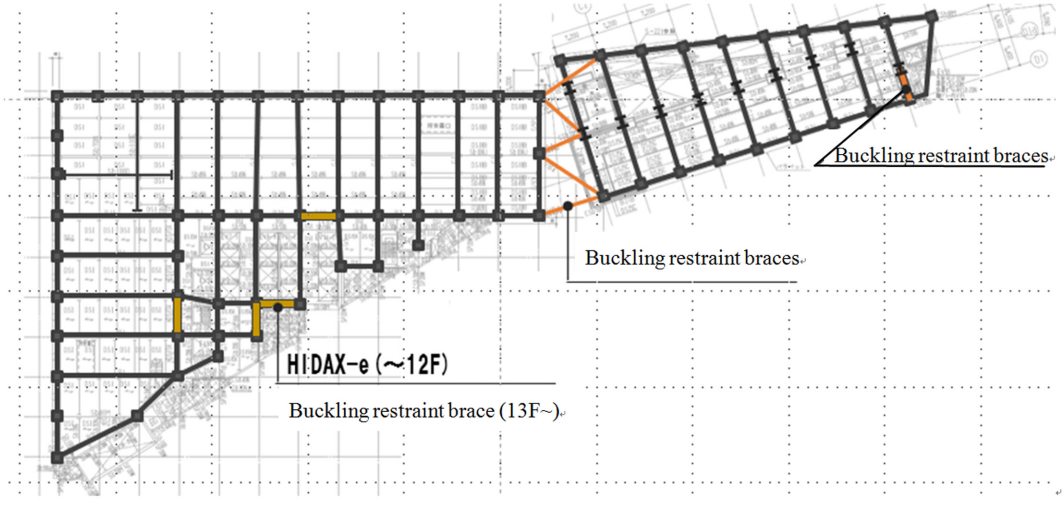


Figure 7. The location of the seismic control system on the typical floor.

trapezoidal shape, as it is located at the apex of the wedge-shaped site, with a footprint of approximately 66 m × 27 m. The office floor is 12 m column-free. The structure type is a moment resisting steel frame consisting of CFT columns and steel beams, similar to Building A1. Building A2 is connected to Building A1, which is larger in scale, so that horizontal forces during an earthquake are partially absorbed by Building A1. This minimizes the

use of structural control devices in the office area of Building A2, and the rental area is designed so as not to reduce the rental area. Since torsional vibrations are generated in the short side direction during earthquakes due to the effect of the linkages, buckling-restrained braces are provided at the outermost point.

3.3. Outline of Underground and Foundation

The A1 and A2 buildings in the basement are integrated: the slab on the ground floor is at least 300 mm thick to allow access for construction vehicles, but also to ensure smooth transmission from the superstructure to the basement retaining walls which resist all seismic forces.

As Building A1 has a large number of above-ground stories and uses a top-down method to build the basement at the same time as the ground level, temporary columns are needed to support the superstructure until the basement is built - usually these are also used as structural elements - and between the basement and ground floors, the CFT is switched to steel reinforced concrete with built-in cross-shaped steel columns.

The foundation sits on a layer of consolidated silt at a depth of 15 m below the design GL. The foundation type is being rationalized in accordance with the construction plan each for Building A1 and A2. Since the overall construction period is determined by Building A1, the top-down method is used for Building A1 because the construction of the underground frame from the ground level to the basement in parallel with the above-ground steel framework is effective in shortening the construction period. The top-down method requires a temporary foundation piles and columns to precede the superstructure. By using these for the main structure and combine with raft foundation, the foundation and underground construction is rationalized. Pile diameters and lengths in the ground are adjusted by column location to control bearing capacity and settlement corresponding to the axial forces that occur as steel construction progresses. On the other hand, building A2 is not critical in the overall process, so the raft foundation without piles was adopted by using a bottom-up method in which construction proceeds upward from the foundation after excavation.

Since the construction method differs between Building A1 and Building A2, construction joints are provided at the basement connection to absorb the difference in deformation caused by ground deformation and other factors during construction.

4. Seismic Response Analysis

Time history seismic response analysis is required by the Japanese Building Code as a seismic design method



Figure 8. Conceptual drawing of the foundation.

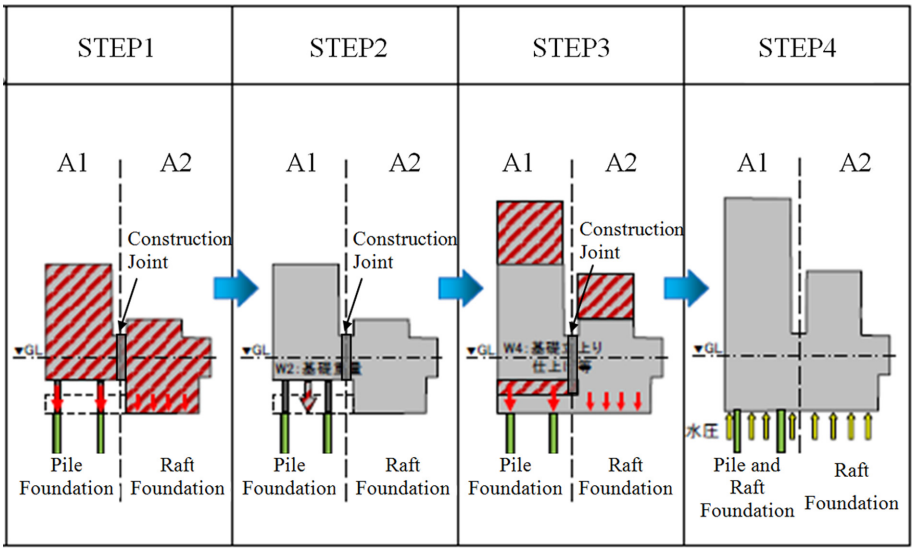


Figure 9. Construction Procedure.

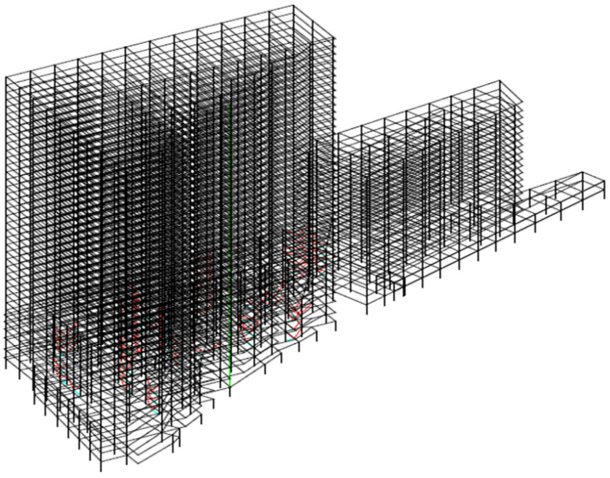


Figure 10. Seismic Response Analysis Model.

for buildings over 60 m in height. The analysis was performed using a three-dimensional framework analysis model with columns, beams and column-beam connections. The oil damper was modelled as a Maxwell model with a series of coupled spring and dashpot elements representing the oil damper characteristics.

The primary period of the horizontal shaking of the building is 4 to 5 seconds for Building A1 and approximately 2 seconds for Building A2. Two levels of design seismic motions have been adopted as seismic loads: infrequent (level 1) and very infrequent (level 2). Three observed waves recorded in the past and three synthetic waves are selected for seismic response analysis. For level 2, the KA1 long-period earthquake and the Taisho Kanto and North Tokyo Bay earthquakes are used as site waves. The seismic criteria for the superstructure are listed below.

For level 1, each structural member except for BRBs shall be less than the allowable stress. The story drift angle between floors shall be 1/200 or less.

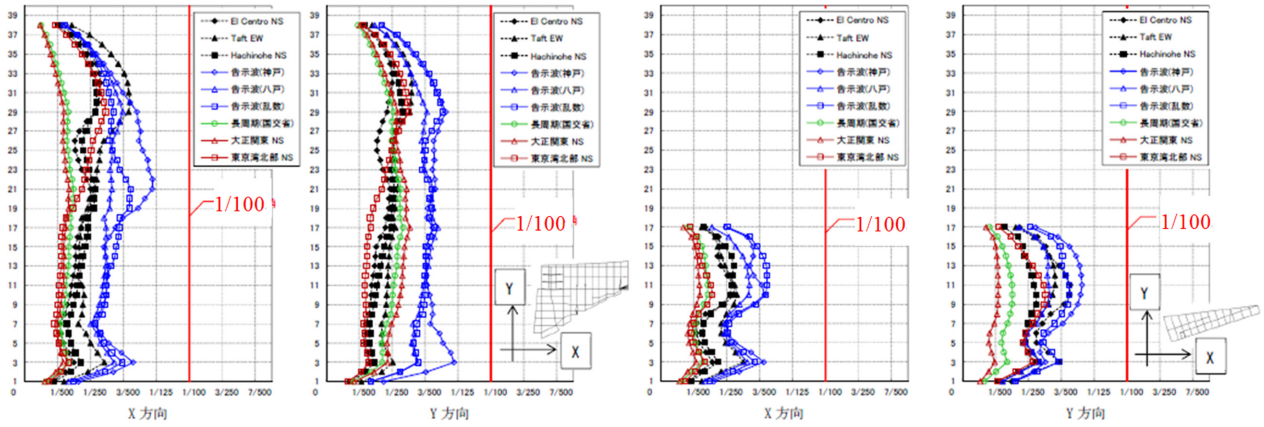
For level 2, the response plasticity factor of each member, except for BRBs, shall be less than 4.0. The story drift angle between floors shall be 1/100 or less.

For level 2, the oil dampers shall be less than or equal to the allowable value based on the design specifications.

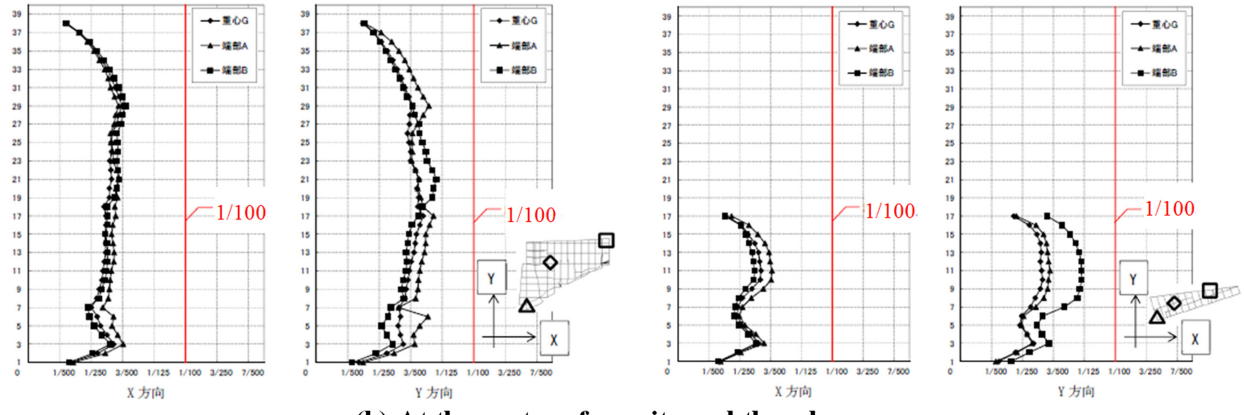
For level 2, fatigue damage of BRBs shall be less than the allowable value.

The results of the seismic response analysis are shown below.

Figure 11(a) shows the maximum story drift angle at the center of gravity for level 2.



(a) At the center of gravity



(b) At the center of gravity and the edge

Figure 11. Maximum Response Story Drift Angle.

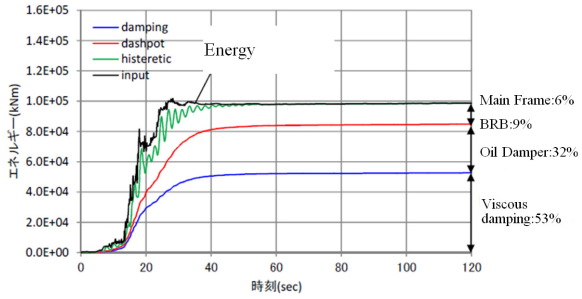


Figure 12. Energy absorption time history.

Figure 11(b) shows the maximum story drift angle at the center of gravity and at the edge.

Not only the response at the center of the gravity but also the response at the edges meets the criteria of the story drift angle.

Figure 12 shows the time history of energy absorption for level 2. The oil dampers absorb about 30% of the total energy, and the BRBs absorb about 10%, giving a total energy absorption of 40% by the seismic damping device. The amount of energy absorbed by the frame is about 6%, indicating that damage to the frame is reduced.

5. Design of Connections

5.1. Design of Dampers

The dampers used as a joint damper system are a feature of this building, and they are installed externally. Since they must function reliably throughout the service life of the building, it was considered essential that they should be completely maintenance free, and buckling restraint braces with hot-dip galvanized finish were selected.

For the BRBs, it was confirmed that the fatigue damage was less than the allowable value for level 2 based on the results of the seismic response analysis. The results of the study are shown below.

The fatigue damage factor D of the BRBs is calculated using equations (2) to (4).

$$D = N/N_f \tag{2}$$

D : fatigue damage factor

Where N is the number of times the absorbed energy is converted into the equivalent absorbed energy at the maximum axial strain, calculated from equation (3).

$$N = \Delta W / W_{max} \tag{3}$$

ΔW : energy absorbed during the earthquake

W_{max} : Energy absorbed in a loop at maximum axial strain

The calculation of N_f is based on equation (4).

$$\frac{D_p}{t}\frac{\sigma_y}{E}\frac{\Delta\varepsilon_t}{2} = 0.00541 N_f^{-0.333} \tag{4}$$

D_p : Steel tube diameter

t : Steel tube thickness

σ_y : Yield stress

E : Young's modulus

$\Delta\varepsilon_t$ (%) : total strain amplitude

Accuracy of steel erection was required for the on-site installation of the joint damper. As the bracing steel itself could not be re-manufactured in time, it was necessary to adjust the steel frame at a height of approximately 80m above the ground, which could only be achieved by adjusting the joints. As a result of the settlement of the two buildings with different foundation types, the fabrication

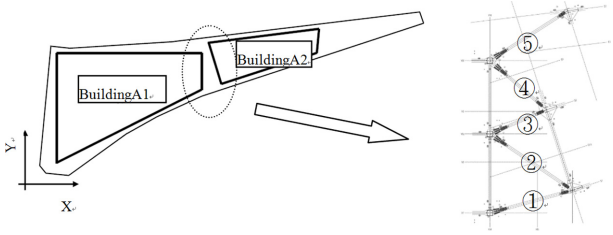


Figure 13. The location of BRBs.

Table1. Fatigue damage in X direction for level 2

Name	Yield Force P_y(kN)	Total strain amplitude $\Delta\varepsilon_t$ (%)	Energy ΔW(kNm)	Fatigue damage D	Location
Wave 1	4000	2.46	861	0.053	③
Wave 2	4000	1.59	1362	0.035	③
Wave 3	4000	1.88	2174	0.078	③
KA1	4000	0.31	0	0.000	④

Table 2. Fatigue damage in Y direction for level 2

Name	Yield Force P_y(kN)	Total strain amplitude $\Delta\varepsilon_t$ (%)	Energy ΔW(kNm)	Fatigue damage factor D	Location
Wave 1	4000	2.55	1736	0.089	④
Wave 2	4000	1.42	2493	0.040	④
Wave 3	4000	1.84	7563	0.143	①
KA1	4000	0.74	846	0.002	①

accuracy of the steel frames from different fabricators, the erection accuracy on site, and the thorough quality control of all these factors, the installation was successfully completed by adjusting the position of the holes in the splice plate by up to 20 mm in the longitudinal direction.

5.2. Design of the connection slab

Buildings A1 and A2 are connected at floors 2 to 9. During an earthquake, a portion of the shear force generated in each building is transferred to the other through the slab at the connected floors. It is confirmed that the shear stress in the slab due to the transfer force at the connection is within the allowable shear stress.

6. Conclusions

The Shibuya Sakura Stage, which integrates the low-rise sections of two buildings of different heights (180 m and 90 m) with a joint damper seismic control system, was introduced.

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